PERFORMANCE ASSESSMENT OF URBAN DRAINAGE SYSTEMS OF ALETA-CHUKO TOWN

MASTER OF SCIENCE THESIS

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This is to certify that the thesis entitled "Performance Assessment of Urban Drainage Systems of Aleta-Chuko Town submitted in partial fulfillment of the requirements for the degree of Master's with specialization in Water Resource Engineering and Management, the Graduate Program of the Faculty of Bio-systems and Water Resource Engineering, Department of Water Resource and Irrigation Engineering, and has been carried out by Mebiratu Mengistu Id. No. GPWREMR/0009/12 under our supervision. Therefore, we recommend that the student has fulfilled the requirements and hence hereby can submit the thesis to the department.

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As members of the Examining Board of the Final MSc Open defense, we certify that we have read and evaluated the thesis prepared by Mebiratu Mengistu entitled "Performance assessment of urban drainage systems of Aleta-Chuko town" and we recommend that it can be accepted as fulfilling the thesis requirement for the degree of masters of science in Water Resources and Irrigation Engineering (specialization: Water Resource Engineering and Management)

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Final approval and acceptance of the thesis are contingent upon the submission of the final copy of the thesis to the School of Graduate Studies (SGS) through the Department/School Graduate Committee (DGC/SGC) of the candidate"s department.

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DECLARATION

I declare that this thesis is my effort work and all sources of materials used for this thesis have been duly acknowledged. I formally declare that this thesis is not submitted to any other institution anywhere for the award of any academic degree, diploma or certificate.

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LIST OF ABBREVIATIONS AND ACRONYMS

CAD	Computer-Aided Design
CW	Weighted Runoff Coefficient
DDM	Drainage Design Manual
DEM	Digital Elevation Map
EPA	Environmental Protection Agency
ERA	Ethiopian Road Authority
FHWA	Federal Highway Administration
FUPCOB	Federal Urban Planning Coordinating Bureau
GIS	Geographical Information Systems
GPS	Global Position System
GTZ	German Technical Cooperation Agency
HA	Hectare
HEC-RAS	Hydrologic Engineering Center – River Analysis System
IDF	Intensity-Duration-Frequency
IDFC	Intensity Duration Frequency Curve
km^2	Square kilometer
MOUDC	Ministry of Urban Development and Construction
n	Manning's Roughness Coefficient
NMSE	National Meteorology Service Agency
NRCS	Natural Resources Conservation Service
SUDS	Sustainable Urban Drainage System
SWMM	Storm Water Management Model
Tc	Time of concentration
USWD	Urban Storm Water Drainage

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ABSTRACT

As the process of urbanization accelerates, drains become increasingly overloaded and unable to cope with heavy rainfall. The main objective of this study was to assess the performance of the urban drainage system in Aleta-Chuko Town. Both primary and secondary data were collected by field survey, Resident community, agency, Municipal administration, books, and articles. Rainfall data were used from Ethiopian Meteorology Agency from 1991 to 2019 (30 years) data. To develop the Intensity Duration Frequency curve (IDF) the rainfall intensities for the different duration were selected as the best-fit probability distribution based on Easyfit professional 5.6 software and correlation coefficient. From the analysis result, log-Pearson Type III was the best fit probability distribution for this study area as confirmed goodness of fit tests statistics for different return periods. Considering the current land use, rainfall intensity, and catchment area, the peak discharge was estimated using the rational formula. The adequacy of the existing drainage systems were checked by comparing the estimated runoff with the existing drainage capacity . The Stormwater Management Model (SWMM 5.1) was applied to simulate the water level in the links and junctions by considering the current land use condition.The model allows the catchment area to be subdivided into sub-catchments. For this study area is subdivided into 32 sub-catchments. From model simulation result's a number of the existing drainage lines are undersized, and therefore not able to handle the required capacity of a 10-year storm occurrence. Typical flooding during the rainy season sub-catchments are: S-C-02, S-C-03, S-C-13, S-C-14, S-C-27, S-C-28, S-C-30, and S-C-31. The storm overtops the drainage system. In another hand, some stormwater drainage systems were oversized. Based on the GTZ standard, For instance total drains about 37.93% is severely degraded, and 24.2% is light around existing market and Aleta- Chuko Primary Hospital. Finally, lack of community awareness, drainage systems are not well connected,and improper construction alignment problem for the existing system were investigated, and to avoid this problem creating awareness for the community, repair degraided channel either fully or partially, providing drainage channels without drainage system's, periodic cleaning and modification of slope is recommended.

Keywords*:* Storm drainage systems, Capacity of existing drains, Runoff, LID

1 INTRODUCTION

1.1. General Background

As the process of urbanization accelerates, drains become increasingly overloaded and unable to cope with heavy rainfall. the loss of permeable soil due to urban development and improper design are the major causes of flooding in most developing urban areas in Ethiopia (Belete, 2009), this urbanization affects the performance of the drainage lines. Drainage systems seek to manage rainfall in a way similar to natural processes, by using the landscape to control the flow and volume of surface water and reduce pollution of downstream development (Yidnekachew, 2019).

The major causes of flooding as the blockage of urban storm water drainage lines along with inadequate/poor integration between road and urban storm water drainage infrastructures. In addition, with urbanization, impermeability increases with the increase in impervious surfaces (i.e. residential houses, commercial buildings and paved roads), the overland flow gets faster flooding, and environmental problems like land degradation increase. It is a crucial problem facing the existing and future road infrastructure (Belete, 2011).

In addition, drainage facilities in most urban centers of Ethiopia are at lower coverage. Planning and design rarely guide the construction of such facilities as well as their management leading a significant number of recently constructed roads to be affected and damaged by floods. The construction of improperly designed roads and lack of integration with the master plan frequently creates a considerable adverse impact on the environment and socio-economic activity making it difficult to achieve the required service from a given infrastructure (Meraf, 2015).

In general road construction without adequate provision of drainage is a major cause of flooding. Lack of urban stormwater drainage (USWD) management represents one of the most common sources of complaint from the residents in many urban centers of the country, and this problem gets worse and worse with the rate of urbanization (Wagari, 2019). Currently, Aleta-Chuko Town is the one facing a drainage problem due to inadequate provision of drainage structure in conveying the runoff to the outlet points properly and road drainage structures are not properly functioning. As a result, change in the run-off characteristics within the town increased runoff and greater susceptibility to flooding.

Therefore, this study aimed to investigate urban stormwater drainage network systems and road flooding risk management in Aleta-Chuko Town.

1.2. Statement of the Problem

Essentially, drains are structures that collect, convey and discharge runoff from the surface of road pavements and adjacent catchment areas to artificial or natural waterways like ditches, channels, rivers, streams, ponds, lakes, etc. Drainage problems occur either due to design, usage, increased runoff, or construction defects.

The drainage problem, Aleta-Chuko Town is currently facing the drainage problem due to the expansion of urbanization, as a result, a change in the run-off characteristics within the town increased runoff and is susceptible to flooding due to a change of land use land cover. During the intensive rainfall there is flooding through the study area at most of the drainage systems, this is due to the inadequacy of drainage structures, the increasing imperviousness in the overall catchment or urbanization in a town, improper management of a waste system like illegal dumping of household solid and liquid wastes.

In addition, some roadside ditch is loaded or silted by debris, silt, waste materials, and disposal of solid and liquid waste in the stormwater drainage line, the road becomes flooded. Those problems created the road to malfunction during the rainy season every year. Flooding over asphalts, walkways, and near the residences has been a big problem in Town.

1.3. Objectives

1.3.1. General Objective

The main objective of this study was the assessment of the performance of the urban drainage system in Aleta-Chuko Town.

1.3.2. Specific Objectives

The specific objectives of the study were to:

1. identify the current situation of the Aleta-Chuko Town drainage system.
2. evaluate the hydraulic capacity of the existing drainage structures of the Town.
3. estimate peak discharge by SWMM5.1 and indentify flooded junctions (flood prone area)
4. determine the impact of best management practices in terms of runoff reduction.

1.4. Research Questions

1. What is the current situation of the Aleta-Chuko Town drainage system?
2. What is the hydraulic capacity of the existing drainage structures of the Town?
3. What is the peak discharge by SWMM and where was flood prone area?
4. What are the possible recommendations to alleviate the problems taking best management systems (BMS) in to account?

1.5. Significance of the Study

The benefit that draws from this study may contribute to current efforts by governments and other concerning bodies to solve the problem of drainage schemes that contribute to better service coverage. To understand problems of damage and preserve the structures by avoiding further deterioration by taking corrective measures as well as to reduce any inconvenience and disruption to travel due to the overflow of water in the main road due to flooding. Also may help in filling the gaps by identifying problems to Sustainability, taking proper designing of the Stormwater drainage system, and proper functioning of drainage schemes in the town.

This study aims at improving the situation of the Aleta-Chuko Drainage system problems, so further investigation will contribute to the solution for the stormwater drainage problem and sustainable drainage system for future use in the area. It is beneficial for researchers who conduct similar research on road drainage structures, Ethiopia Road Authority (ERA), Aleta-Chuko Town Administration, and other stakeholders who will conduct similar research on other road drainage structures.

1.6. Scope and limitation of this Study

This study will address issues related to urban stormwater drainage. The specific focus of this study includes: evaluating the extent and performance of the existing drainage system and proposing best management practice for the existing problem.

Some representative major flood prone areas were selected. According to the residents these areas have been flooded most of the rainy season and based on field observation two major flood prone Kebeles were selected. Evaluating the whole catchment is not necessary to come up with solution for the current storm water problem. Therefore, some representative major flood prone areas are selected. Therefore, this study was geographically limited at Kebele-01 and Kebele-02.

It is not common in our country measuring the water level in the junction points and outfalls what do developed countries like the USA and Canada even by the installation of the sensor. For instance, there is no recorded data of runoff at different junction points and outfalls to calibrate the simulated result of the model at the junctions as well as outfalls. So, the model results are simulations without calibrations.

2 LITERATURE REVIEW

2.1. Historical Perspectives of Urban Storm Drainage System

The drainage systems of the Roman Empire were the first documented systems that systems have been viewed from various perspectives. During different periods and in different locations, it has been considered a vital natural resource, a convenient cleansing mechanism, an efficient waste transport medium, a flooding concern, and a transmitter of disease. In general, climate, topography, geology, scientific knowledge, engineering and construction capabilities, societal values, religious beliefs, and other factors have influenced the local perspective of urban drainage (Burian and Edwards, 2002).

Before giving an account of the important transformation which occurred in the 19th Century related to sanitary waste systems and rainwater drainage, it is worthwhile to go back some years in history when, according to Silveira (2002), a period of elimination of flooded areas began, the burying of septic tanks and then their replacement by underground channeling systems. This process for removal of waste and rainwater began in Italy as a result of observing a correlation between mortality rates of people and animals and the sanitation system; after that, it was adopted in innumerable European cities as a public health measure.

The perspective of urban drainage also changed from a design standpoint during the nineteenth century. Most sewers constructed before the nineteenth century were not planned or designed by an engineer using numerical calculations. Instead, a trial-and-error process was executed, which in some cases eventually produced well-functioning systems (Belete, 2009).

2.2. Current Urban Storm Drainage Systems Perspectives

Urban drainage in the early parts of the twentieth century was firmly established as a vital public works system. Engineers continued to improve design concepts and methods. During the second half of the twentieth-century regulatory elements were promulgated in the United States, Europe, and other locations addressing urban drainage issues. Extensive monitoring efforts vastly improved the understanding of urban drainage quantity and quality characteristics. Computer modeling tools advanced the methods used to design and analyze urban drainage systems. Regulations, monitoring, computer modeling, and environmental concerns have altered the perspective of urban drainage from a public health and nuisance flooding concern during the first half of the twentieth century into a public health and nuisance flooding with additional concerns for ecosystem protection and urban sustainability(Wagari, 2019).

Methods to design and construct sustainable urban drainage systems are currently being researched and tested. Alternative development concepts (e.g., low-impact development) are influencing development practices to minimize the impacts of development on stormwater drainage (Graham, 2012). Communities worldwide are yet searching for innovative techniques to capture, detain, and use rainwater within the watershed instead of constructing massive drainage structures. Many communities are developing watershed-wide storm water quality management plans to meet the dual objectives of flood prevention and water quality control. Urban drainage has indeed expanded significantly during the past few decades beyond a technical challenge to drain the urban area rapidly to include the consideration of social, economic, political, environmental, and regulatory factors (David Butler and John W. Davies, 2000).

2.3. Urban Storm Drainage System Problems

The main causes of stormwater drainage system problems are the inadequacy of drainage structures during the rainy season to pass the flood, poor quality construction, inappropriate site selection, and improper alignment of some drainage structures for road alignment. These shortcomings cause damage to superstructures of drainage structures and stream crosscurrents.

The practice of urban drainage in developing countries encounters more serious problems than those in developed countries because urban development occurs under more difficult socioeconomic, technological, and climatic conditions. Developing counties experience accelerated urbanization without adequate investment in infrastructure, and against a background of deficient public services for water treatment, collection and treatment of foul sewage, garbage collection, urban drainage, transport, and health. Urban concentrations have environmental consequences in the form of urban flooding and pollution of watercourses, soil, and air. Settlements are established in inappropriate areas such as those originally set aside for environmental preservation and on steep hillsides and areas liable to flooding (Hassen, 2016).

2.4. Urban Storm Drainage Systems Practice in World

Drainage systems are needed in developed urban areas because of the interaction between human activity and the natural water cycle. This interaction has two main forms: the abstraction of water from the natural cycle to provide a water supply for human life, and the covering of land with impermeable surfaces that divert rainwater away from the local natural system of drainage (Butler and Davies, 2000).

According to United Kingdom's Department for Environment Flood and Rural Affairs (DEFRA) research framework for implementing integrated urban drainage, it is defined as the management of the risk arising from drainage and flooding in urban areas. A worldwide number of different countries are working toward integrated drainage, either on its own or as part of a broader approach to managing the water cycle. In the United Kingdom, the term Sustainable Urban Drainage System (SUDS) is used to mean the integrated urban drainage system, as the integration of the drainage system with the environment ultimately leads to sustainability (Balmforth, et al., 2009).

2.5. Urban Storm Water Drainage Practice in Africa

Different types of structures are employed in the drainage systems are open channels whether artificial or natural convey the flows of water surface and sub-surface drainage systems (Kalantari, 2011). In addition noted that the main challenge in developing countries has to do with the lack of an adequate budget and number of skilled personnel who can plan and implement urban drainage system (Armitage, 2010).

Sustainable Urban Drainage Systems (SUDS) is a philosophy used around the world to help reduce the excess flow of stormwater from spreading into unwanted areas. The major goals of the SUDS philosophy are to manage stormwater runoff, rid the water of any pollutants, and encourage community involvement. When local community members participate in implementing and managing stormwater solutions, it increases the likelihood that community members will take care of the stormwater management systems, making said solutions more successful in the long run (SUDS Background, 2005).

2.6. Urban Storm Water Drainage Practice in Ethiopia

In Ethiopia's context, where watersheds of many urban centers receive a significant amount of annual rainfall and where rainfall intensity is generally high, control of runoff at the source, flood protection, and safe disposal of excess water/runoff through proper drainage facilities are very essential (MoWUD,2008).

During continued urbanization, more roads, houses, and buildings of both commercial and industrial kind are added to the area. More people can now live together on a closer area, consequently increasing wastewater amounts being discharged into local water bodies. The impermeable surfaces in form of pavements and constructions hinder the infiltration of water into ground water reserves, giving less water for groundwater recharge. Furthermore, an increase in stormwater runoff can also be noted. Water that was to be infiltrated, will instead reach waterbodies with the risk of causing flooding (USGS, 2016a).

In addition, as FUPCoB (2008), the drainage problems in urban areas include flooding, deterioration of roads, land degradation, sedimentation, blockage of drainage facilities, waterlogging, etc. With urbanization, impermeability increases with the increase in impervious surfaces (i.e. residential houses, commercial buildings, paved roads, parking lots, etc.), drainage pattern changes, the overland flow gets faster, and flooding and environmental problems such as land degradation increase. It is a crucial problem facing the existing and future road infrastructure. Despite these problems, drainage facilities in most urban centers of the country are nearly absent or at lower coverage. Planning and design rarely guide the construction or provision of such facilities and management.

Despite these problems, drainage facilities in most urban centers of Ethiopia are nearly absent or at lower coverage. Planning and design rarely guide the construction or provision of such facilities and management (MoWUD, 2008). Similarly, Mahari and G/Mariam, (2015) noted that whereas in Ethiopia there is relatively good progress in the expansion of roads but many of these roads are not functioning well to the desired lifetime and quality mainly due to failure of the cross drainage structure.

Adequate drainage facility is an important factor that should be given paramount importance in the design of roads since it greatly determines the road's serviceability and useable life. Drainage problems in urban areas include flooding residential areas deterioration of roads, land degradation, sedimentation and blockage of drainage facilities as well as waterlogging (Mulualem, 2016).

2.7. Development of IDF Curve

The design of hydraulic structures for the conveyance of stormwater requires knowing the magnitude of the incoming peak flood in a certain return period. This is mainly conducted to determine the discharge capacity of stormwater and sewerage systems, drainage systems like channels, and culverts, and the design of bridges (Fasikaw and Tsegamlak, 2017).

2.8. Hydraulics of Storm Drainage Systems

2.8.1. Flow Type Assumptions

The design procedures presented here assume that flow within each storm drain segment is steady and uniform. This means that the discharge and flow depth in each segment is assumed to be constant for time and distance. Also, since storm drain conduits are typically prismatic, the average velocity throughout a segment is considered to be constant. In actual storm drainage systems, the flow at each inlet is variable, and flow conditions are not truly steady or uniform. However, since the usual hydrologic methods employed in storm drain design are based on computed peak discharges at the beginning of each run, it is a conservative practice to design using the steady uniform flow assumption (Debo and Reese, 2003).

2.8.2. Hydraulic Capacity

The hydraulic capacity of a storm drain is controlled by its size, shape, slope, and friction resistance. Several flow friction formulas have been advanced which define the relationship between flow capacity and these parameters. The most widely used formula for gravity and pressure flow in storm drains is Manning's Equation (ERA Drainage Design Manual, 2002).

2.9. Urban Drainage System Modelling and Selection criteria

As a level of software availability, accuracy, cost, and time of the modeling should be selected to investigate flooding problems. Many stormwater management models are used in different parts of the world, like Australia, Victoria and the United State of America (USA) are (WP Software, 1991), HEC-RAS(Hydrologic Engineering Centre, 2000), ILSAX (O'Loughlin, 1993), CIVILCAD (Surveying and Engineering Software, 1997), RORB (Laurenson and Mein, 1990), SWMM (U.S. Environmental Protection Agency, 1992), WBNM (Boyd et al., 2000) and

Rational Formula method (Aitken,1975) were used as Urban drainage computer models for design and analysis.

The most widely used urban drainage models in Australia and overseas in modeling event hydrographs are AUSQUAL (White and Cattell, 1992), STORM (U.S. Army Corps of Engineers, 1977), and MOUSE (Danish Hydraulic Institute, 1988). some are commercial and some are open-source. The open-source models require a nominal cost; however, they provide very little technical support for users. In contrast, commercial models support beginners well, but their cost is often too high for widespread use (Zoppou, 2001).

SWMM is a widely accepted model and is currently applicable in Ethiopia for planning, analysis, and design related to the water system in urban areas (Getachew, et al, 2015). For this study, the stormwater management model(SWMM) was selected due to availability, good degree of precision of the estimation and easily understandable use of both hydrological and hydraulic characteristics of catchment, and also is mainly used for flood plain mapping of channel system, design, and sizing of drainage system components(Niyonkuru et al., 2018).

2.9.1. EPA's Stormwater Management Model (SWMM)

Stormwater modeling has significant consequences for urban hydrology, water quality, and flood risk, and has changed substantially over history, but it is unknown how these paradigm shifts play out at the local scale and whether local changes in stormwater infrastructure use follow similar trajectories across cities drainage systems (Rebecca, 2016). Stormwater management practices, when properly selected, modeled, and implemented, can be utilized to mitigate the adverse hydrologic and hydraulic impacts caused by drainage facilities, thereby protecting downstream areas from increased flooding, erosion, and water quality degradation (Nejib, 2016).

2.9.1.1. Modeling Capabilities of the SWMM

SWMM accounts for various hydrologic processes that produce runoff from urban areas. These include:

- ✓ Time-varying rainfall
- ✓ Evaporation of standing surface water
- ✓ Rainfall interception from depression storage
- ✓ Infiltration of rainfall into unsaturated soil layers

- ✓ Percolation of infiltrated water into groundwater layers
- ✓ Interflow between groundwater and the drainage system
- ✓ Capture and retention of rainfall/runoff with various types of low-impact development (LID) practices.

SWMM contains a flexible set of hydraulic modeling capabilities used to route runoff and external inflows through the drainage system network of pipes, channels, storage or treatment units, and diversion structures. These include the ability to handle drainage networks of unlimited size, use a wide variety of standard closed and open conduit shapes as well as natural channels model special elements such as storage or treatment units, flow dividers, pumps, weirs, and orifices. The runoff component operates on a collection of sub-catchment areas that receive precipitation and generate runoff and pollutant loads. SWMM tracks the quantity and quality of runoff made within each sub-catchment (Rossman, 2004).

2.9.1.2. Application of SWMM

SWMM applications have been used in sewer and stormwater studies throughout the world. Its typical applications include:- designing and sizing drainage system components for flood control, sizing detention facilities and their appurtenances for flood control and water quality protection, mapping flood plains of natural channel systems, designing control strategies for minimizing combined sewer overflows, evaluating the impact of inflow and infiltration on sanitary sewer overflows, generating non-point source pollutant loadings for waste load allocation studies, controlling site runoff and evaluating the effectiveness of BMPs for reducing wet weather pollutant loadings (Rossman, 2004).

2.9.1.3. Computational Methods of the SWMM

Since SWMM is a physically-based, discrete-time simulation model and it employs principles of conservation of mass, energy, and momentum wherever appropriate. The methods SWMM uses to model stormwater runoff quantity and quality through the following physical processes.

- **Surface Runoff**

Inflow comes from precipitation and any designated upstream sub-catchments. Each sub-catchment surface is treated as a nonlinear reservoir. There are several outflows, including infiltration, evaporation, and surface runoff. The capacity of this "reservoir" is the maximum depression storage, which is the maximum surface storage provided by ponding, surface wetting, and an interception.

- **Infiltration**

Infiltration is the process of rainfall penetrating the ground surface into the unsaturated soil zone of pervious sub-catchment areas. For modeling infiltration SWMM offers:

a) Horton's Method

This method is based on empirical observations showing that infiltration decreases exponentially from an initial maximum rate to some minimum rate throughout a long rainfall event. The maximum and minimum infiltration rates, a decay coefficient that describes how fast the rate decreases over time, and the time it takes a fully saturated soil to completely dry are some of the input parameters required by this method (Rossman, 2004).

b) Modified Horton Method

This is a modified version of the classical Horton Method that uses the cumulative infiltration above the minimum rate as its state variable (instead of time along the Horton curve), providing a more accurate infiltration estimate when low rainfall intensities occur. It uses the same input parameters as does the traditional Horton Method (Rossman, 2004).

c) Green-Ampt Method

This method for modeling infiltration assumes that a sharp wetting front exists in the soil column, separating soil with some initial moisture content below from saturated soil above. The initial moisture deficit of the soil, the soil's hydraulic conductivity, and the suction head at the wetting front are input parameters required in this method. The recovery rate of moisture deficit during dry periods is empirically related to the hydraulic conductivity (Rossman, 2004).

d) Modified Green-Ampt Method

This method modifies the original Green-Ampt procedure by not depleting moisture deficit in the top surface layer of soil during initial periods of low rainfall as was done in the original method. This change can produce more realistic infiltration behavior for storms with long initial periods where the rainfall intensity is below the soil's saturated hydraulic conductivity.

e) Curve Number Method

This approach is adopted from the NRCS (SCS) Curve Number method for estimating runoff. It assumes that the total infiltration capacity of a soil can be found from the soil's tabulated Curve Number. During a rain event, this capacity is depleted as a function of cumulative rainfall and remaining capacity. The input parameters for this method are the curve number and the time it takes a fully saturated soil to completely dry (Rossman, 2004).

Flow Routing

Flow routing within a conduit link in SWMM is governed by the conservation of mass and momentum equations for gradually varied, unsteady flow (i.e. the Saint-Venant flow equations). We have a choice on the SWMM for flow routing. They are Steady Flow Routing, Kinematic Wave Routing, and Dynamic Wave Routing. Each of these routing methods employs the Manning equation to relate the flow rate to flow depth and bed (friction) slope. For user-designated Force Main conduits, either the Hazen-Williams or Darcy-Weisbach equation can be used when pressurized flow occurs (Rossman, 2004).

a) Steady Flow Routing

Steady Flow routing represents the simplest type of routing possible (actually no routing) by assuming that within each computational time step flow is uniform and steady. Thus, it simply translates inflow hydrographs at the upstream end of the conduit to the downstream end, with no delay or change in shape. The normal flow equation is used to relate the flow rate to flow area or depth. This type of routing cannot account for channel storage, backwater effects, entrance/exit losses, flow reversal, or pressurized flow. It can only be used with dendritic conveyance networks, where each node has only a single outflow link (unless the node is a divider in which case two outflow links are required). This form of routing is insensitive to the time step employed and is only appropriate for preliminary analysis using long-term continuous simulations.

b) Kinematic Wave Routing

This routing method solves the continuity and momentum equation in each conduit. The maximum flow that can be conveyed through a conduit is the full normal flow value. And allows flow and area to vary both spatially and temporally within a conduit. However, this form of routing cannot account for backwater effects, entrance/exit losses, flow reversal, or pressurized flow, and is also restricted to dendritic network layouts. It can usually maintain numerical stability with moderately large time steps, on the order of 1 to 5 minutes (Rossman, 2004).

C) Dynamic Wave Routing

It solves the complete one-dimensional Saint-Venant flow equations and therefore produces the most theoretically accurate results. These equations consist of the continuity and momentum equations for conduits and a volume continuity equation at nodes. With this form of routing it is possible to represent pressurized flow when a closed conduit becomes full, such that flows can exceed the full normal flow value. Flooding occurs when the water depth at a node exceeds the maximum available depth and the excess flow is either lost from the system or can pond atop the node and re-enter the drainage system. Dynamic wave routing can account for channel storage, backwater, entrance/exit losses, flow reversal, and pressurized flow. Because it couples together the solution for both water levels at nodes and flows in conduits it can be applied to any general network layout and with much smaller time steps, on the order of thirty seconds or less (Rossman, 2004).

2.10. Low Impact Development(LID)

Due to climate change urban flooding frequently occurs in the worldwide and also highspeed urbanization has caused rapid changes to the underlying surfaces, resulting in fundamental changes in urban runoff. Urban water logging brings a series of socio-economic losses such as traffic paralysis, loss of property and even human losses.

The effective control of urban water logging is both crucial and difficult to manage for urban storm water runoff. The evaluations of the effectiveness of the various management measuresare more important. Various runoff reduction measures have been implemented, including LID, and widely applied throughout the world(Asghar and Garg, 2018). LID is an innovative urban storm water management system that was jointly introduced by the storm water management experts from Programs and Planning Division of Prince George‟s County Department of Environmental Resources during the mid-1990s.

At the end of the 1990LID was developed by the United States Environmental Protection Agency (USEPA) with encouragement. It has been generally recognized and adopted by countries all over the world. A series of related storm water management regulations have been formulated in Florida, Chicago, and other locales, and remarkable achievements have been made (Wang, 2015). The idea behind LID is to depress the negative influence of water quantity as well as the quality of the runoff process caused by urbanization to allow regional runoff processes to return to a natural undeveloped state to the largest degree possible. Its meaning has been extended in many countries or regions that have similar ideas regarding storm water management, such as Water-Sensitive Urban Design (WSUD) in Australia, Sustainable Urban Drainage Systems(SUDS) in the UK, and the natural drainage systems inSeattle (Nasrin, 2018).

2.10.1. Commonly Used LID Techniques

Effective low impact development includes the use of both non-structural and structural storm water management measures that are a subset of a larger group of practices and facilities known as Best Management Practices (BMP). As noted above, the BMPs utilized in low impact development, known as LID-BMPs, focus first on minimizing both the quantitative and qualitative changes to a site‟s pre-developed hydrology through nonstructural practices and then providing treatment as necessary through a network of structural facilities distributed throughout the site. In doing so, low impact development places an emphasis on non-structural stormwater management measures, seeking to maximize their use prior to utilizing structural BMPs. Non-structural BMPs used in low impact development seek to reduce storm water runoff impacts Structural BMPs used to control and treat runoff are also considered LID-BMPs if they perform these functions close to the runoff‟s source.

LID techniques are not restricted to land development sites with limited impervious cover but can also be applied to virtually any development site regardless of the impervious coverage, to produce improved site designs and lesser storm water impacts. The common LID practices are bio-retention, green roofs, permeable pavements, rain gardens, vegetative swales, and rain cisterns (Rain Barrel)that are used to create a functionally equivalent hydrologic landscape (Enis Baltaci, 2016). These LID practices play an important role because of their ability to store water, allowing it to infiltrate or releasing it to receiving streams. They also have the benefit of lengthening the flow path and runoff time (Asghar and Garg, 2018).

The two popular LIDs currently used in residential areas are bio-retention cells and rain barrels. An analysis of LID for runoff reduction obtained the benefits of optimized LID implementation in reducing runoff and peak flow rates because LID reduces the need for expensive channel systems such as pipes, channels, and combined sewer systems (Seema Bardhipur, 2017).

2.10.1.1. Bio Retention Cells

Bio-retention cell is the most widely applied LID practice throughout the U.S., which restores the natural system function by using design techniques that infiltrate, filter, store, evaporate, and detain runoff close to its source (Trowsdale and Simcock, 2011). Bioretention cell consists of a grass buffer strip, a sand bed, a pond area, an organic layer of mulch, planting soil, and plants. Runoff water passes across the length of the pond area which consists of organic mulch. Later, water infiltrates into planting soil and sand beds (USEPA, 2000). Some of the bio-retention facilities have under drains which convey the excess water to the storm drain system. Bio retention cell reduce the peak flow depending on site conditions such as soil and basin slope with substantially delayed time-to-peak (Trowsdale and Simcock, 2011).

2.10.1.2. Rain Barrels

Rainwater tanks are one of the widely-used WSUD approaches for non-potable reuses oroutdoor uses (Mogen felt, 2017). These are popular on-site storm water rainwater collection method which store water during a storm event. These storage tanks are usuallyplaced beneath roof downspouts, which capture roof runoff and thus prevent storm waterinflow entering the sewer network(Nasrin, 2018).

2.10.1.3. Permeable Pavements

Permeable pavements are excavated areas where gravel is used to fill the area. Here, porous concrete or asphalt mix is used for paving the surface. Storm water runoff can pass through the permeable surface, filter by the soil layer and then enter the gravel storage zone beneath the pavement. After that, runoff can easily infiltrate the natural soil or convey to storm drain through optional drainage system. They are effective reducing peak runoff and improving groundwater recharge (Patwardhan et al., 2005). They can improve water quality as well by reducing sediments, nutrients and metals (Sample et al., 2014).

2.10.1.4. Detention ponds

They are used to retain stormwater runoff from impervious area during storm event and then, completely release through some specific outlets within few hours. They store storm water runoff temporarily and thus, reduce runoff volume and peak flows (Pennino et al., 2016). They have varying styles in terms of manicured or natural appearing vegetation.

2.10.1.5. Infiltration Trenches

These are narrow ditches filled with gravel to the ground level. They provide storage and capture stormwater runoff from the impervious areas. The captured runoff then infiltrates into the natural soil (Rossman and Huber, 2016). They can significantly reduce runoff volumes that enter sewer system. They can also improve landscape and aesthetic by providing green space (Sample et al., 2014).

2.10.1.6. Swales

These are depressed areas which act as channels to route the surface runoff. Grass or vegetation is used to cover the sliding slopes of the depression areas (Rossman and Huber, 2016). Vegetative swales help to reduce the conveyance capacity of stormwater runoff andprovide sufficient time to infiltrate the storm water into the natural soil.

2.10.1.7. Green Roofs

They are also known as vegetated roof covers. Green roofs have a surface layer of living plants that grow on the top of a roof a thin soil layer and a special drainage mat below the soil layer. They can retain significant amount of rainfall and roof runoff, then filter through soil layer and drain excess percolated water off the roof (Trowsdale and Simcock, 2011). They have multitude of benefits other than retarding storm water runoff and decreasing flows to sewer network during intense rainfall. This include reducing direct energy uses and urban heat island effects through evaporative cooling, removing sound pollution and improving airquality and biodiversity (Wise et al., 2010). They can also provide green space in dense urban zones and thus, improve community aesthetic.

2.10.2. Implementations of LID to SWMM5

Within LID simulation, SWMM is one of the earliest hydrological models to be supplemented with LID module. The LID module of SWMM provides five single measures for storm runoff control. The hydrological process and its key factors, such as regional surface runoff and peak discharge, can be simulated by applying the LID module combined with the hydraulic module. Hence, the runoff reduction effect of LID can be evaluated through these key factors. The SWMM for different LID measures has also been extensively used in different regions and countries, especially in Germany. Because the SWMM was well applied in both storm runoff simulations and LID evaluations and was appropriate for different types of areas, it was selected as the simulation model.

3 MATERIAL AND METHODS

3.1. Description of the Study Area

3.1.1. Location

Aleta-Chuko is one of the towns in the Sidama National Regional State (SNPRS), at a distance of 64 km from the SNPRS capital, Hawasssa, 339 km from the national capital, Addis Ababa. 33 km north of Dilla, west of Aleta Wendo at a distance of 11 kilometers. The coordination of town extends from 06^{0}33’00’’N to 06^{0}37’30’’ N latitude and 38^{0}18'00" E to 38^{0}23'00" E longitude. And the study area is indicated below in (Figure 1).

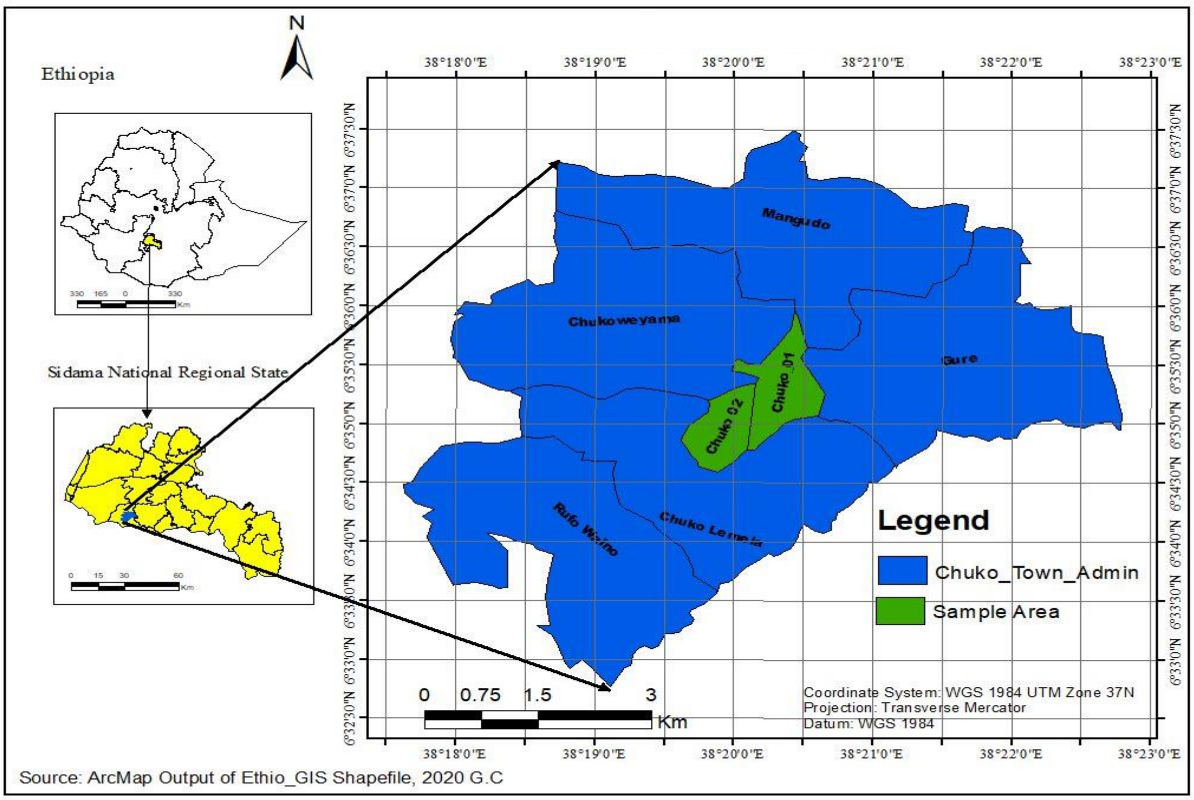

Figure 1: Location map of the Study area

3.1.2. Topography

The relief of the town is classified into plain and small portions of the undulating surface. As one moves from the center to the northeast, west further south the altitude generally decreases. Likewise, the altitude increases as one moves from the central part of the towns towards the east direction. Almost all part of the town is characterized by plain topography. The altitude of the town ranges from 1890 meters around the eastern part towards Senteria road Area to 1816 meters above sea level towards the southern peripheries.

The average altitude of the town is 1850 meters above sea level. Generally, from the topographic point of view, the town is best suited for settlement.

3.1.3. Climate

Climatic factors considered for the study include wind, rainfall, temperature, humidity, etc... Concerning the study town, traditionally is classified into Woina-dega type.

3.1.4. Catchment Shape

Study town catchment areas that contribute to runoff in the main discharging direction have oval kinds of shapes. That runoff from the upper land drain to a central part of the town contributes much runoff to the lower stream.

3.1.5. Soil Group

Soil properties influence the relationship between runoff and rainfall since soils have differing rates of infiltration. Permeability and infiltration are principal data required to classify soils into hydrologic soil groups (HSG).

Most of the soil in the study area is grouped under the Soil Group: - Clay loam, silty clay loam, silty clay, or clay. Soils have a high runoff potential due to a very slow infiltration rate. These soils primarily consist of clays with high swelling potential, soils with permanently-high water tables, soils with a claypan or clay layer at or near the surface, and shallow soils over nearly impervious parent material.

3.1.6. Slope of the Terrain

The area which is flat to the rolling slope has a high time of concentration. The runoff takes more time to reach a certain point. In the town which has flat to rolling terrain, the runoff and flood are expected to continue for some time after the rainfall stops.

3.2. Research Methods and Procedures

This flow chart shows the steps and methods which were followed to conduct this study.

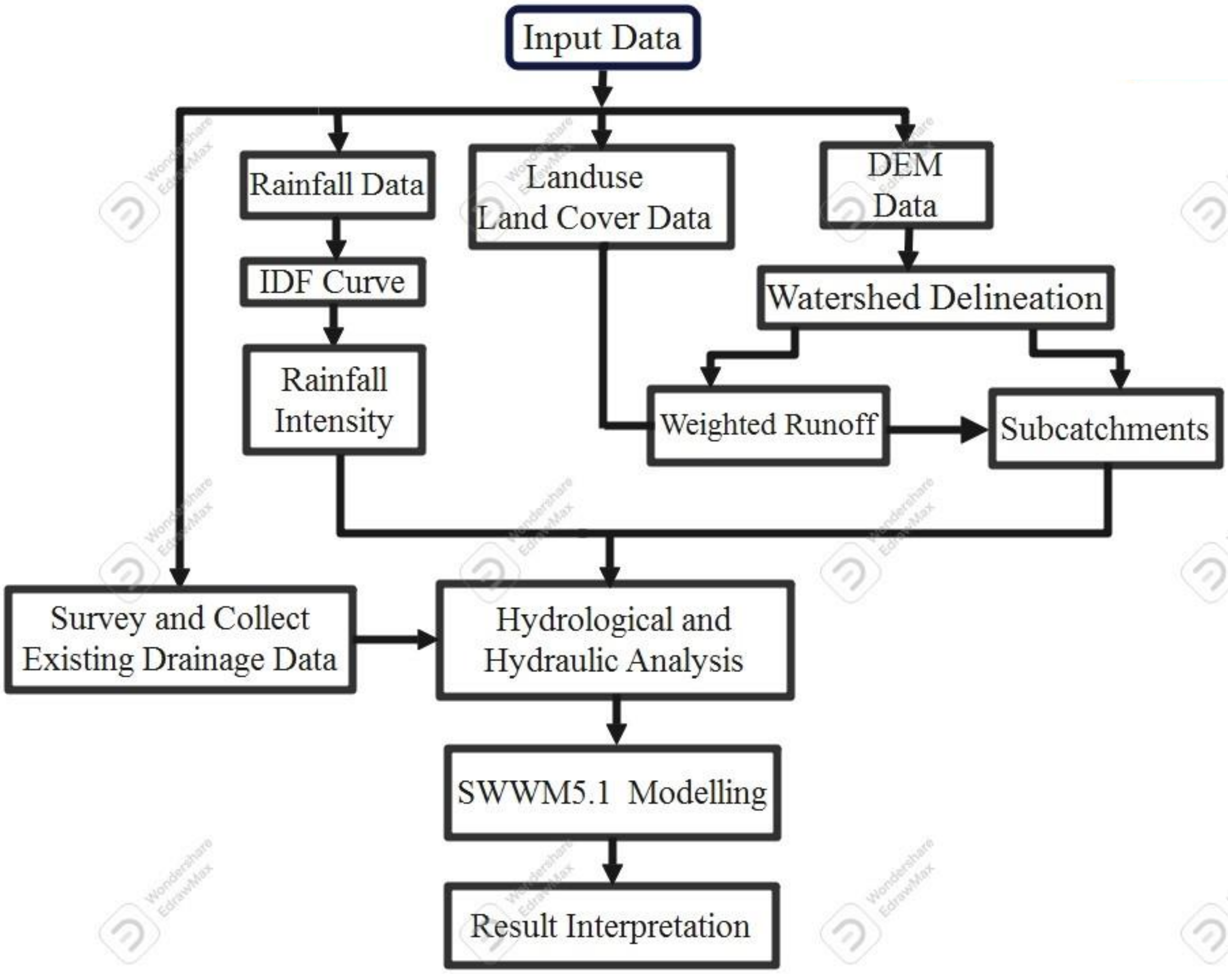

Figure 2: Conceptual framework

3.2.1. Assessment of Current Situation Existing Drainage Systems

For this study, information and data collection were obtained from primary and secondary sources.

Primary Sources

i) Field Survey

A field survey was employed to measure the dimensions of drainage lines located in the study area, and to gather information about the current condition of the drainage system with the help of a master plan and checklist as per the objective of this study. Observing flood marks, measuring the size of the existing drainage structures, measuring the elevation difference and flood marks as well as gathering information about the overall performance of drainage structures during the rainy season.

The materials used for the study of research are digital cameras, GPS devices, and measuring tape. All these materials were used during a field visit to the study area for getting my primary data.

Secondary Sources

Secondary data sources that were used were books, journals, manuals conference proceedings, Meteorological data (National Meteorological Agency of Ethiopia), contour maps, other findings/literature, reports, etc. Both quantitative and qualitative techniques in data collection and analysis were utilized as the main instruments.

3.2.2. Analysis of Intensity-Duration-Frequency (IDF) Curves

Rainfall intensities of various frequencies and durations are important parameters for the hydrologic design of storm sewers, culverts, and other hydraulic structures. This can be achieved by the rainfall Intensity–Duration–Frequency (IDF) relationship, which is determined through rainfall frequency analysis. The order statistics approach is applied for the determination of distributional parameters to estimate rainfall and develop IDF relationships for different return periods. (Vivekanandan, 2013).

3.2.2.1. Meteorology Data (Rainfall Data)

Rainfall data for the Study area can be obtained from five important stations: Aleta-Chuko, Aleta-Wendo, Tefer-Kella, Kebado, and Aposto. But, the Aleta-Chuko station is aged not more than ten years with a lot of missing data therefore the data are not complete enough to use for frequency analysis. This study used rainfall data from the Aleta-wendo station for it has thirty consecutive years(from 1991 to 2019) of daily rainfall depth record as well as the station is very close to the study area. Therefore the data at this station is used to develop an intensity-duration curve (IDF) for rainfall analysis.

The rainfall data is the basic input to many hydrological models as it affects the resulting runoff peak and volumes as well as sets in one rain gauge station but due to the incompleteness of rainfall data additional three rain gauge stations were used for this thesis to fill unrecorded rainfall data.

3.2.2.2. Missing rainfall data estimation and data quality

Due to the absence of observer or instrumental failure rainfall data records occasionally are incomplete. In such a case one can estimate the missing data by using the nearest station rainfall data. There are different approaches for estimating missing rainfall data varying with and based on the effect of orography on rainfall, the distance between the rainfall stations, and the variation of rainfall amount recorded on the stations Several methods have for estimating missing rainfall data were station average method, normal ratio method, and inverse-distance weighting method, and regression methods. (Dingman, 2002). From the methods, the normal ratio is used to fill in missed data for this study.

The normal ratio method is recommended to estimate missing data in regions where annual rainfall among stations differs by more than 10% (Dingman, 2002). For instance, if rainfall data on day 1 is missed from station x having a mean annual rainfall of Nx and there are other three surrounding stations with a mean annual rainfall of N1, N2, and N3 respectively, then the missed data P_x can be computed as follows (Garg, 2005).

$$Px = \frac{1}{3}\left(P1\frac{Nx}{N1} + P2\frac{Nx}{N2} + P3\frac{Nx}{N3}\right) \quad (3.1)$$

Where: P_x – Missing rainfall data (daily, monthly or yearly)

P_1, P_2, and P_3 – Rainfall data at different nearest stations (daily, monthly, or yearly)

Nx-Mean annual rainfall at missed station

N_1, N_2, and N_3 –Mean annual rainfall at nearest stations corresponding to P_1, P_2, and P_3.

3.2.2.3. Check the quality of the Data

An outlier is an observation that deviates significantly from the bulk of the data, which may be due to errors in data collection or recording, or due to natural causes (Ramachandra et al, 2000). The presence of outliers in the data causes difficulties when fitting a distribution to the data. Low and high outliers are both possible and have different effects on the analysis. Outliers in maximum daily rainfall can play a considerable role in the unreal analysis leading to unreal predictions. Therefore, accurate statistical determination of data to find outliers is very important. The following frequency equation can be used to detect high and low outliers.

$$Y_H = \text{antilog}(\bar{x} + k_N S) \quad \text{..} (3.2)$$

$$Y_L = \text{antilog}(\bar{x} - k_N S) \quad \text{..} (3.3)$$

Where: Y_H and Y_L -are the high and low outliers of the sample, respectively.

K_N- Constant given in appendix A, table A1 for sample size N

S- Standard deviation of N sample size

Therefore, the study area is analyzed since, from 1990 up to 2019 consecutively for 30 years of daily heaviest rainfall. Hence, the highest outlier is 2.015 in the log unit (103.556mm) and the lowest at 1.414 in the log unit (25.951 mm). (Appendix Table 1).

Hence, there is no data below and above the outlier values through this data, which means all the available data satisfy the given condition.

3.2.2.4. Check the Consistency of the Data

Consistency of time series was analyzed based on the theory that a plot of two cumulative quantities measured for the same period should be a straight line and their proportionality unchanged, which is represented by the slope. Sometimes a significant change may occur in and around a particular rain gauge station. Such a change occurring in a particular year will start affecting the rain gauge data, being reported from that particular station. After several years, it may be felt that the data of that station is not giving consistent rainfall values. Therefore, the inconsistency of the record data was corrected by double mass curve techniques (Garg, 2005).

This study was conducted for rainfall data sets of four stations indicated that the average cumulative precipitation of neighboring stations is serially arranged in the reverse chronological order (i.e. the latest year getting the first entry) and plotted with the yearly cumulative precipitation of the study area for the corresponding years (Appendix Table 2) as shown in (Figure 3) below.

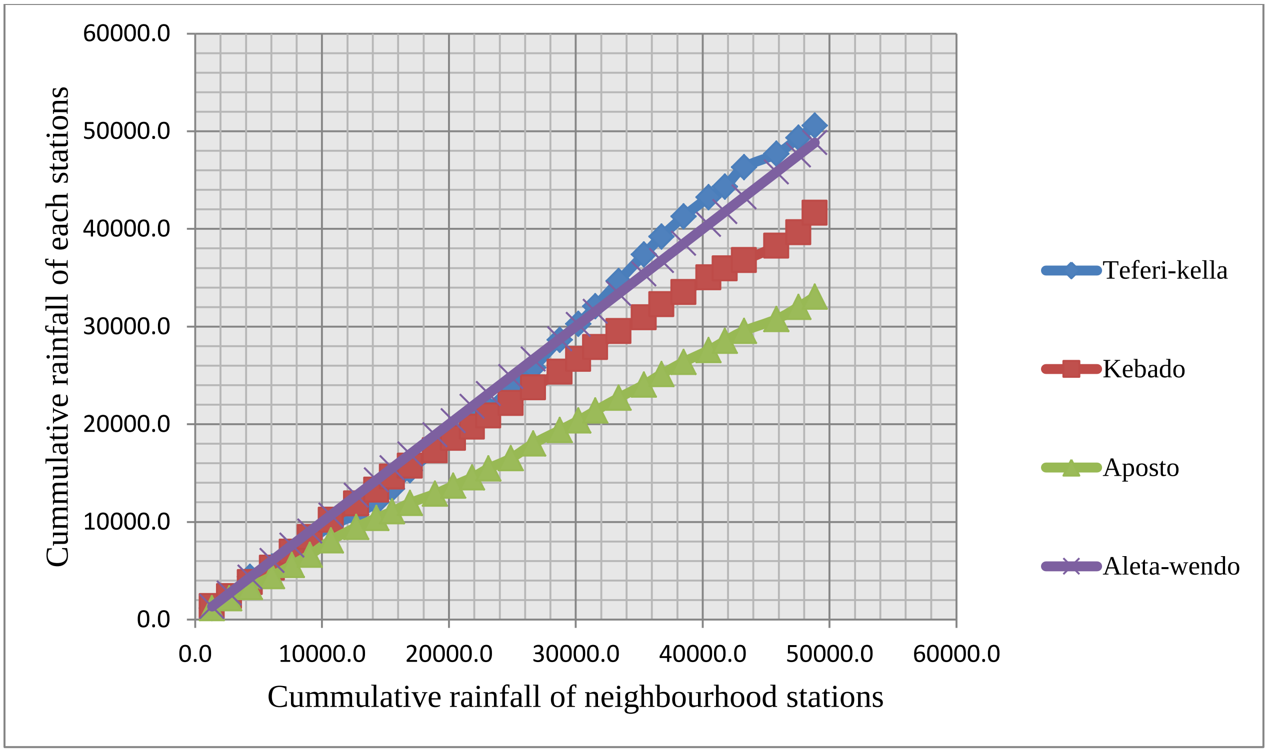

Figure 3: Uncorrected double mass curve

3.2.2.5. Adjust for the inconsistency of rainfall data

Therefore, inconsistency of the record was done by the double-mass curve technique. This technique is based on the principle that when each recorded data comes from the parent population, they are consistent. The double mass curve technique was used to adjust precipitation records to take account of non-representative factors such as a change in location or exposure of a rain gauge. The accumulated totals of the gauge in question are compared with the corresponding totals for a representative group of nearby gauges.

If a significant change is observed on the plot, it should be corrected by:

$$Px = Px * \frac{M}{M'} \quad \text{...(3.4)}$$

Where: - Px' = Corrected precipitation at station x

Px = Original recorded precipitation at station x

M' = Corrected slope of the double mass curve

M = Original slope of the double mass curve

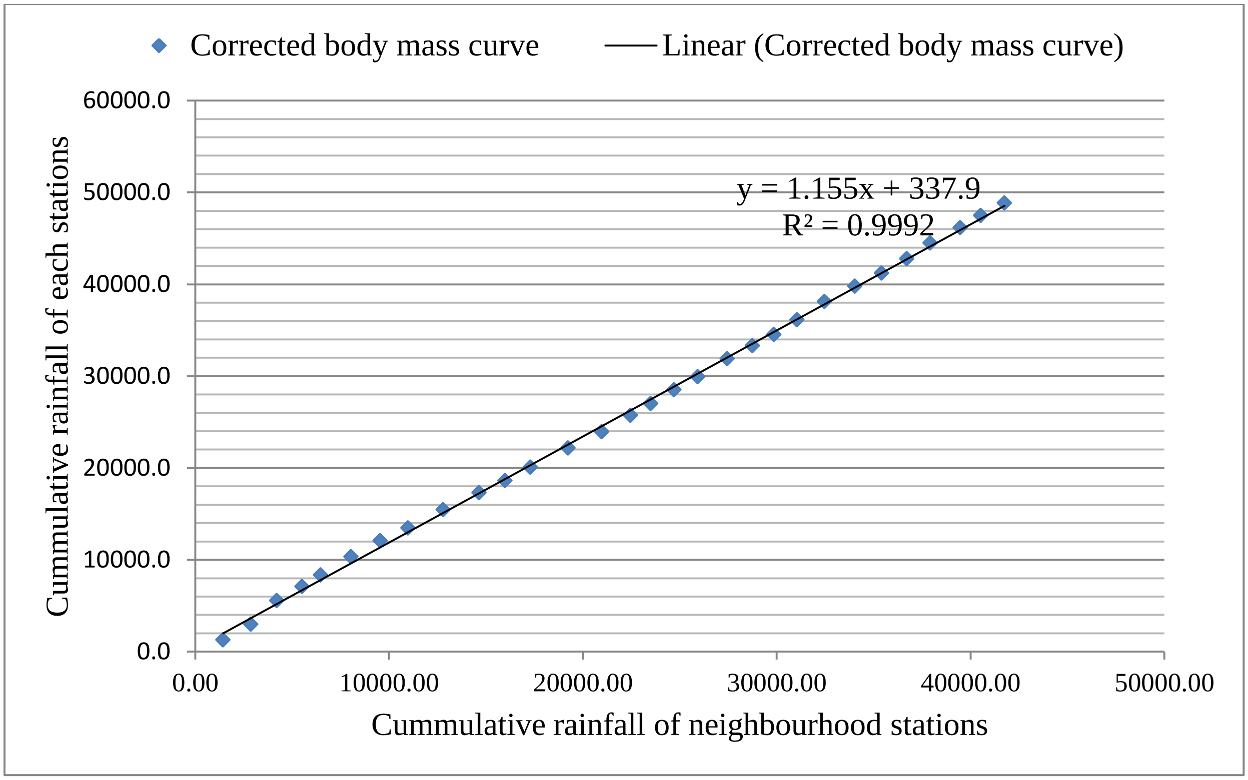

Figure 4: Corrected double mass curve

3.2.2.6. Analysis of rainfall frequency distribution

Hydrologic processes such as floods are complex natural events. To analyze the maximum discharge expected in T years we can use the frequency distribution function. Some of the commonly used frequency distribution functions for the prediction of extreme maximum values are; Normal distribution method, the Gumbel distribution method, the Log-Pearson type III distribution method, Log-normal distribution method (Subramanya, 1994). Two common and suggested by the Ethiopian Drainage Design Manual (ERA, 2013) Gumble theory of distribution and Log-Pearson type III frequency analysis techniques were used to develop the relationship between rainfall intensity, storm duration, and return periods from rainfall data for this study.

Log- Pearson Type III

Log- Pearson Type III probability model was selected to perform the flood probability analysis to develop IDF curves. It uses only extreme events (maximum values or peak rainfalls). This frequency analysis method calculates the 2, 5, 10, 25, 50, and 100- year return intervals for each duration period and requires several calculations.

In this method, the flow data is first transformed into logarithmic form (base ten) and the transformed data is then analyzed. If X is the variety of random flow series then the series of Z varieties where Z is obtained for this series for any recurrence interval T.

$$Z = \log X \quad Z_T = \bar{Z} + K_Z * \delta z \qquad (3.5)$$

Where, Kz = a frequency factor which is a function of T and the coefficient of skewness Cs,

Variation of the Kz = f (CS, T) is given in the table as a function of Cs and T

δz = standard deviation of Z variety sample.

$$\delta n\text{-}1 = \sqrt{\frac{\sum(Zi-\bar{Z})^2}{N-1}} \qquad (3.6)$$

$$Cs = \frac{N\Sigma(Z-Z)3}{(N-1)(N-2)(\delta z)} \qquad (3.7)$$

Finally, the antilog of the solution in X_T will provide the estimated extreme value for the given return period.

Gumbel Distribution Method

Gumbel Distribution Method is used to calculate the rainfall intensity at different rainfall durations and return periods to form the historical IDF curves for each station and expressed by this equation.

$$X_T = \bar{X} + K_T * \boldsymbol{\delta} n\text{-}1 \qquad (3.8)$$

Where; X_T=annual maximum of mean flow of T year return period

K_T=frequency factor and expressed as δn-1=standard deviation of the sample size

$K_T = (Y_T - \bar{y}n)/\bar{Sn}$

Y_T be a reduced variety, a function of T, and is given by: $Y_T = -\ln(\ln(T/(T-1)))$

Yn = reduced mean, it is a function of sample size.

Sn=reduced standard deviation which is also a function of the sample.

Yn and Sn are obtained from Gumbel table

3.2.2.7. Selection criteria of rainfall frequency analysis

For this study, to select the best frequency analysis of rainfall data based on;

- correlation coefficient

- Easyfit professional 5.6 software
- Recommendations of Ethiopian Road authority Drainage Design manual,(ERA, 2013).

3.2.2.8. Goodness of Fit Test

The goodness of fit (GOF) tests were used to identify which distribution fits theoretical probability distribution and conducted using Easy Fit 5.6 professional software. It measures the compatibility of a random sample with a theoretical probability distribution function. These tests show how well the selected distribution fits the data. There are three most commonly used GOF tests. These tests are the Anderson Darling, the Kolmogorov-Smirnov, and the Chi-Squared tests. In all three tests, a parameter or statistic unique to each method is calculated for the required distribution types and these distributions are ranked based on their parameter values.

i. Kolmogorov-Smirnov Test

This test is used to decide if a sample comes from a hypothesized continuous distribution. It is based on the empirical cumulative distribution function (ECDF). Assume that we have a random sample X_1, X_2, X_3,,,,,,,,,X_N from some continuous distribution with CDF F(x). The empirical CDF is denoted by

$F_n(X) = \frac{1}{n} * (Number\ of\ observations \leq X)$

The Kolmogorov-Smirnov statistic (D) is based on the largest vertical difference between F(x) and Fn(x). It is defined as

Dn = $Sup_x /Fn(X) - F(X)/$ When comparing different distributions lower statistics mean a better fit.

ii. Anderson-Darling Test

The Anderson-Darling procedure is a general test to compare the fit of an observed cumulative distribution function to an expected cumulative distribution function. This test gives more weight to the tails than the Kolmogorov-Smirnov test.

The Anderson-Darling statistic (A2) is defined as

$$A^2 = -n - \frac{1}{n}\sum_{i=1}^{n}(2i-1)\cdot[\ln F(X_i) + \ln(1 - F(X_{n-i+1}))]$$

The hypothesis regarding the distributional form is rejected at the chosen significance level (alpha) if the test statistic, A^2, is greater than the critical value obtained from a table. When comparing different distributions lower statistics mean a better fit.

iii. Chi-Squared Test

The Chi-Squared test is used to determine if a sample comes from a population with a specific distribution. This test is applied to binned data, so the value of the test statistic depends on how the data is binned. Although there is no optimal choice for the number of bins(k), there are several formulas that can be used to calculate this number based on the sample size(N). For example, EasyFit employs the following empirical formula:

$$K = 1 + \log_2 N$$

The data can be grouped into intervals of equal probability of equal width. The first approach is generally more acceptable since it handles peaked data much better. The Chi-Squared statistic is defined as,

$$X^2 = \sum_{i=1}^{k} \left(\frac{Oi - Ei}{Ei}\right)^2$$

Where; Oi = is the observed frequency for bin i

Ei = is the expected frequency for the bin I calculated by Ei = F(x2)-F(x1), Where F is the CDF of the probability distribution being tested, and x1, x2 are the limits for bin i. When comparing different distributions lower statistics mean a better fit.

Easy Fit 5.6 Professional software is used for testing the goodness of the recommended Log Pearson type-III and Gumble Methods in all three tests parameter to each method ranked based on their parameter values.

3.2.2.9. Correlation Coefficient (R2)

The Correlation Coefficient is a Statistic showing the degree of relation between two variables. It measures the nature and strength of two quantitative variables, i.e. the sign of r denotes the nature of association while the value of r denotes the strength of association.

3.2.2.10. Design rainfall of shorter duration

The rainfall depths obtained from the gauging station were 24hr duration depth. However, when short time duration rainfall data were not available, the intensity of a short time rainfall of longtime rainfall would be calculated using the reduction formula. Design and analysis of drainage structures require a rainfall intensity-duration relationship of shorter duration. Because rainfall data of a shorter duration is unavailable, appropriate IDF derivation for the shorter duration is required. Ethiopian Road Authority (ERA, 2013) Drainage Design Manual of 2013 suggests the following equation for calculation of shorter duration rainfall from 24hour duration rainfall were used.

$$R_{Rt} = \frac{t}{24}\frac{(b+24)^n}{(b+t)^n} \qquad (3.9)$$

Where:

R_{Rt} = Rainfall depth ratio Rt: R24

R_t = Rainfall depth in a given duration t

R_{24} = 24hr rainfall depth

Coefficients b = 0.3 and n = 0.78 - 1.09

The methods employed to develop the IDF curve for the shorter duration events using the above equations for this thesis, Log-Pearson type III distribution were selected and R24 was calculated for 2, 5, 10, 25,50, and 100 year return periods. Rearranging the above equation:

$$Rt= \frac{t(b+24)^n}{24(b+t)^n} R24 \qquad (3.10)$$

Substituting Intensity (mm/hr) in the above equation

$$It = \frac{Rt}{t} \qquad (3.11)$$

$$It = \frac{R24(b+24)^n}{24(b+t)^n} \qquad (3.15)$$

Using b = 0.3 and n = 0.92 as suggested by ERA manual results are tabulated for rainfall durations 10, 20, 30 … 180 minutes.

The maximum peak flood was computed using the return period recommend in ERA Drainage Design Manual 2013 considering the road standard and the design life span of the structure,10 years for design and a 25-year check (review) return period was adopted (design storm frequency (yrs) by Geometric Design Criteria (ERA DDM, 2011).

3.2.3. Hydraulic Capacity and Design of the Existing Drainage Systems

The hydrological analysis is the most important step before the hydraulic design of any drainage structure. Assessment of hydrological conditions involves the study of landscape Characteristics of the watershed area including topographic conditions, soil characteristics, land use, and rainfall. Hydrological parameters calculation should be completed by making it familiar with the local conditions and stream flows. The calculation of hydrology was carried out using a rational equation and Hydraulic parameters calculations are carried out using Manning's equation.

3.2.3.1. Design flood Estimation using Rational Method

This method estimates the peak runoff rate for small urban and rural watersheds of less than 50ha (0.5 km^2) as recommended by ERA Drainage Design Manual (EDDM) 2013. The equation of the rational formula is a function of the catchment area, runoff coefficient, and time of concentration. The equation is expressed as:

$$Q_P = 0.00278CIA \qquad (3.16)$$

Where: Q_P - Peak run-off (m3/sec)

C=Runoff coefficient

I= rainfall intensity (mm/hr.)

A = catchment area in (ha).

A. Runoff Coefficient (C)

The runoff coefficient C is the function of the land use land covers of the study area and is used as input for peak discharge estimation. The more the surface is impervious the higher the runoff would be as the infiltration decreases. The rainfall intensity directly affects the runoff coefficient. Vegetation cover reduces the impact of a raindrop on the ground and intercepts some of the rain on its leaves and branches letting them evaporate. This directly decreases the runoff coefficient. A weighting method is employed to obtain the representative runoff coefficient i.e. the individual areas multiplied by their specific runoff coefficient and their values added together and divided by the cumulative area (Ven Te Chow, *et al*, 2012).

$$Cw = \sum_{i=1}^{n} \frac{(Ai+Ci)}{At} \qquad (3.17)$$

Where: Ci= Weighted Runoff Coefficient for a given hydrologic soil group area

Ai= Catchment Area under each hydrologic soil group.

At= Total catchment area considered of Town

B. Time of Concentration (Tc)

The time of concentration is the time required for runoff to flow from the most remote point of the basin to the point of interest. In a rational method, it was used to determine the rainfall duration which would result in maximum runoff. The required design rainfall intensity was established from the IDF curve for the required recurrence interval.

$$\mathrm{Tc} = \mathrm{To} + \mathrm{Tt} \quad \text{……..……..………………………………………..………..………..(3.18)}$$

Where: Tc = time of the concentration (minute)

To = time of concentration of overland flow (hr) and

Tt = the travel time of channel flow(min)

- **Time of concentration of overland flow(To)**

The time of concentration of overland flow is estimated using the Kirpich equation.

$$\mathrm{T_o} = \left(\frac{0.87\mathrm{L}^2}{1000\mathrm{s_{av}}}\right)^{0.385} \quad \text{……...………………………………………...………..……..……...(3.19)}$$

Where: To = time of the concentration(hr)

L = hydraulic length of catchments measured along the flow path from the catchment boundary to the point where the flood needs to be determined (km)

Sav = average slope (m/m).

- **Travel Time**

Water moves through a catchment area as sheet flow, shallow concentrated flow, open channel flow, or some combination of these. The type that occurs is a function of the conveyance system and is best determined by field inspection. Travel time of the channel is the ratio of flow length to flow velocity for the channel flow as equation 3.21.

$$\mathrm{Tt} = \frac{L}{3600V} \quad \text{………....………………………..……………………………………………(3.20)}$$

Where: T_t = channel travel time, hr

L = length of the main drainage canal (m)

V = flow velocity in the channel (m/s)

3600 = conversion factor from seconds to hours.

The flow velocity is computed using the Manning formula

$$\mathrm{V} = \frac{1}{n} * R^{\frac{2}{3}} * S^{1/2} \quad \text{……….……………..…………………………………………............(3.21)}$$

Where: V = the flow velocity (m/s), R = the hydraulic radius (m), S = the channel slope (m/m) and n = the Manning roughness coefficient.

C. Rainfall Intensity (I)

The rainfall intensity (I) is the average rainfall rate in millimeters per hour for a duration equal to the time of concentration for a selected return period. Once a particular return period has been selected for design and a time of concentration calculated for the catchment area, the rainfall intensity can be determined from the Rainfall-Intensity-Duration curves of the study area.

D. Catchment Area (A)

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

3.2.3.2. Hydraulic Capacity Analysis using Manning Formula

Manning's formula was used for calculating the cross-sectional area, wetted perimeter, and hydraulic radius for the flow of a specified depth in a canal of known diameter. The formula was applicable for a constant flow rate through the channel with a constant slope, size, shape, roughness, also geometry, and canal type. The hydraulic design of the existing drainage system would be checked using the Manning formula as equation 3.21

$$Q = \frac{1}{n} * AR^{2/3} * S^{1/2} \quad \text{...}(3.22)$$

Where, Q = the volumetric flow rate passing through the channel reach in m^3/sec.

A = the cross-sectional area of flow normal to the flow direction in m^2

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

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

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

R = the hydraulic radius, which is the cross-sectional area of flow normal to the flow direction in m^2 divided by the wetted perimeter of the cross-sectional area of flow in m.

3.2.4. Storm Water Management Model (SWMM)

SWMM is a dynamic rainfall-runoff simulation model used for a single event or long-term (continuous) simulation of runoff quantity and quality from primarily urban areas. The runoff component of SWMM operates on a collection of sub-catchment areas that receive precipitation and generate runoff. In this study, the rational method and Stormwater management model (SWMM) was used for the design of flood computation and its analysis.

3.2.4.1. Model setup procedure

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

3.2.4.2. Model preparation area

Figure 5 shown below shaded by a blue polygon was selected to model the area for this study because of these area more exposed for the flooding.

Fig 5: Selected study area for model preparation

3.4.2.3 SWMM model input parameters

i. **Sub-catchment properties**

Sub-catchments are usually divided into previous and impervious sub-regions. The following infiltration models such as Horton Infiltration, Green Ampt Infiltration, and SCS-curve number Infiltration are described for the analysis of a previous zone. In SWMM sub-catchment include assigned rain-gage, outlet node or sub-catchment, assigned land uses, tributary surface area, imperviousness, slope, characteristic width of overland flow, Manning‟s n for overland flow on both pervious and impervious area, depression storage in both pervious and impervious areas, percent of the impervious area with no depression storage.

Table 1: Input parameters for SWWM

Name	User Assigned sub-catchment name
Area	Area of the sub-catchment in acres or hectares
Width	Characteristics of the overland flow path for Sheet flow runoff (feet or meters). These parameters are important in the modeling of flood peaks.
% Slope	The average percent of the slope of sub-catchment
% Imperviousness	Percent of land area which is impervious. It is measured from an Arial photo or land use map. The runoff volume and flow rates are strongly sensitive to estimate to estimates of imperviousness (Appendix Table 17)
N-Imperviousness	Manning's n for an impervious portion of sub-catchment (Appendix Table 14)
N-Perviousness	Manning's n for a previous portion of sub-catchment
Dstore-Imperviousness	Depth of the depression storage on the impervious of the sub-catchment (inch or mm).
Dstore-perviousness	Depth of the depression storage on the pervious of the sub-catchment (inch or mm).
%zero – Imper	% of the impervious area with no depression
Infiltration	Used to edit infiltration parameters for the sub-catchment. The green-Ampt equation is used for this study because the method reserved considerable attention to calculating the infiltration of urban catchments (Rossman, 2016). It is also a physically-based model which can give a good description

	of the infiltration process.

ii. Rain gauge

Rain-gage provides the rainfall data type, recording time interval, source of rainfall data, and name of rainfall data sources. Rain gauges must be located within and adjacent to the catchment.

Precipitation is supplied from the rain gauge in more than one catchment, but the rainfall data for this study is time series data and one rain gauge record.

iii. Junction nodes

Junction nodes represent the convergence of natural surface channels, manholes in a series system, or pipe fittings. The primary input parameters for a junction are an invert elevation, height to the ground surface, ponded surface area when flooded, and external inflow data.

iv. Outfall Nodes

Outfalls Nodes are terminal nodes that define the final downstream boundaries under dynamic wave flow routing. It behaves as a junction for another flow routing. The input parameters for outfall nodes include invert elevation, boundary condition type, stage description, and the presence of a flap gate to prevent backflow through the outfall.

v. Conduits

Conduits are pipes or channels that move water from one node to another node. The common shape of conduits define in SWMM are rectangular, trapezoidal, or user-defined irregular cross-section shapes. Using manning's Formula Equation 21.

3.4.2.4. Model parameterization

EPA SWMM requires three major parameter categories for runoff quantity modeling including physical catchment characteristics, rainfall, and infiltration. The physical catchment data are total catchment area(A), percentage of impervious area(%Imp), flow width(W), average slope(So), surface depression storage, and surface roughness. Other input parameters for catchment properties were adopted from the range provided in SWMM User's Manual.

Parameters included in sub-catchments, junctions, links, and outfalls were used for the simulation of the model.

Table 2: sensitivity parameter for SWWM

Parameter	Description	Allowed range
N-Imperv	Manning,s roughness coefficient for impervious areas	0.005-0.05
N-Perv	Manning,s roughness coefficient for pervious areas	0.05-0.5
Dstore- Imperv	Depth of surface storage in impervious areas(mm)	1.3-2.5
Dstore- Perv	Depth of surface storage in pervious areas(mm)	2.5-7.6

3.4.2.5. Selected Flow Routing and infiltration Option for Model Simulation

Dynamic Wave routing

Dynamic Wave routing solves the complete one-dimensional Saint-Venant flow equations and therefore produces the most theoretically accurate results. These equations consist of the continuity and momentum equations for conduits and a volume continuity equation at nodes. Dynamic wave routing can account for channel storage, backwater, entrance/exit losses and flow reversal (W.James et al., 2010). This methodwas selected for the model due to the fact that it is eligible for nonuniform unsteady flow moreover it allows to simulate flow parameters within shorter period of time.

Routing step of 30 seconds as well as Hazen Williams equation for pressure drop due to friction were employed for model simulation.

Green-Ampt Method

SWMM offers three methods these are Curve Number method, Horton’s method and the Green-Ampt method for infiltration computation which discussed in literature part. Green-Ampt method input parameter’s required are the initial moisture deficit of the soil, the soil’s hydraulic conductivity, and suction head at the wetting front. The recovery rate of moisture deficit during dry periods is imperically related to the hydraulic conductivity. It is physically based model which can give a good description of infiltration process. In this study Green-ampt method was used.

3.5. Best Management Practices Low Impact Development (LID) Control Scenarios in SWMM5

The LID control in SWMM5 are different practices from these bio-retention, permeable pavement, rain garden, rain barrel, infiltration trench, rooftop disconnection, vegetative swale, and green roofs.

Each LID in SWMM5 has a variety of process layers such as: surface, soil, storage, and drain. Each sub catchment can have multiple LID controls (Seema Bardhipur, 2017b). In Bio retention cell in residential communities is used to treat runoff from roads whereas infiltration trench was used to improve landscape and aesthetic by providing green space. This study explains the two modeling techniques of Bio-Retention cell and infiltration trench can be practice and reduce surface runoff.

3.5.1 Bio Retention Stucture

Bio retention areas or rain gardens are depressed areas in the landscape that are designed to accept stormwater. They can be used in residential and commercial settings and are typically planted withshrubs, perennials, or trees, and covered with shredded hardwood bark mulch. The benefits of bio-retention areas include decreased surface runoff, increased groundwater recharge, and pollutant treatment through a variety of processes (Prince George Country, 1999).

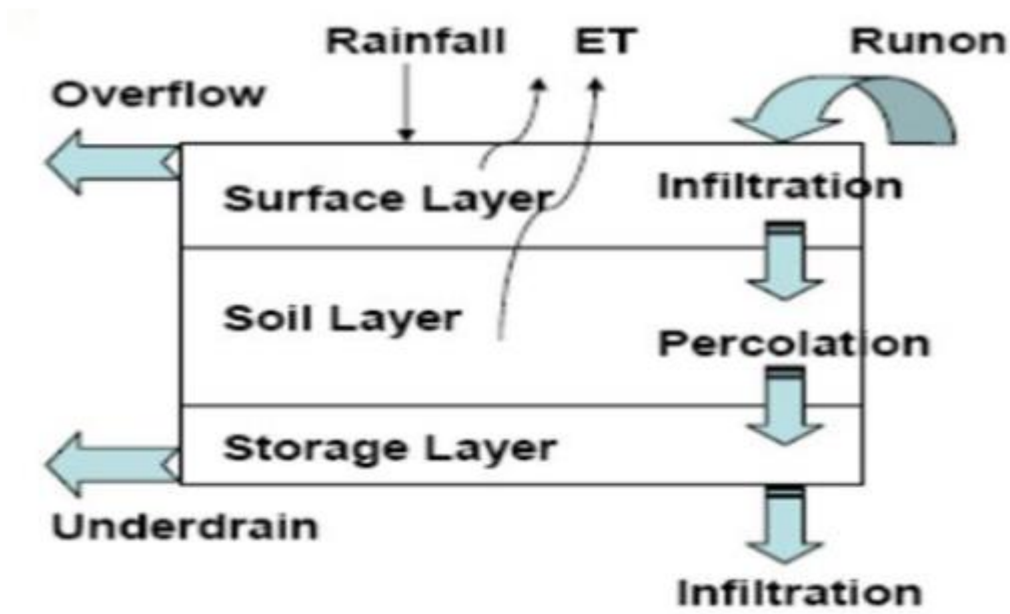

Figure 6: Conceptual Model of a bio retention cell in SWMM (L.A. Rossman, 2015)

Table 3: Parameters of bio retention cells for the Model

Layer	Parameters	Dimension
Surface	Berm Height (mm) Vegetative Volume Fraction Surface Roughness Surface Slope (%)	300 0.20 0.13 0.04
Soil	Thickness (mm) Porosity (volume fraction) Field capacity (volume fraction) Wilting Point (volume fraction) Conductivity (mm/hour) Conductivity slope Suction head (mm)	700 0.475 0.378 0.265 0.254 45 12.6
Storage	Thickness (height) (mm) Void ratio (voids/solids) Seepage rate (mm/hour) Clogging Factor	350 0.60 0.254 474
Drain	Flow coefficient Flow exponent Offset height (mm)	5 0.5 500

3.5.2 Infiltration trench

An infiltration trench is an excavated trench that has been back-filled with stone to form a subsurface basin. Stormwater runoff is diverted into the trench and is stored until it can be infiltratedin to the soil (Fuss & O'Neill, 2013).One infiltration trench with an area of 4000 m2 have been modelled with the following design elements.

Table 4: Parameters of infiltration trench for the Model

Layer	Parameters	Dimension
Surface	Berm Height (mm) Vegetative Volume Fraction Surface Roughness Surface Slope (%)	500 0.20 0.13 0.04
Storage	Thickness (height) (mm) Void ratio (voids/solids) Seepage rate (mm/hour) Clogging Factor	700 0.6 0.254 474
Drain	Flow coefficient Flow exponent Offset height (mm)	34 0.5 0.5

4 RESULTS AND DISCUSSIONS

4.1. Current Situation of Drainage Systems in the Aleta Chuko Town

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

Currently, the stormwater drainage management of the town of Aleta-Chuko is not efficient; as a result design construction and managing problem. Therefore, careful design of the stormwater drainage master plan is crucial.

Figure 7: Areas Exposed to flooding problem around Loko-ber (Source: - Field Survey)

The main problems of the drainage system in the study area during the site visit were:

- Drainage systems are not well connected;
- Most ditches do not have a proper slope to let water pass through them;
- In some areas, there are no drainage systems provided at all
- Some of the existing drainage ditches have been silted by sand and other rubbish materials
- Ponds or other spaces are not properly allocated to accommodate an overflow of flooding;

Figure 8: Deterioration of asphalt road around existing market and stagnation of water on the rightway in front of Aleta-Chuko Primary Hospital respectively

As a whole drainage facility in the Aleta-Chuko town requires the construction of a large number of the drainage system for existing and new unconstructed areas. In addition, the current situation of the drainage system of the study area is presented below. During the Field survey, the existing conditions of the storm drainage system were evaluated by using GTZ standards(Appendix Table 16)

Table 5: Drainage Line From existing cattle market to the Kale-Hwot cemeteries around outlet-01

Drainage Type	Drainage Pavement	Existing Condition	Length(m)	Percent (%)
Open	Masonory	Light	475	26.9
Open	Masonory	Light	400	22.65
Open	Masonory	Good	231	13.08
Closed	Concrete	V-good	660	37.37
Total	***Light =49.55%***	***Good =13.08%***	***V-Good=37.37%***	***100%***

As shown in Table 5, out of the total drains 49.55% is light, 13.08% is good, and 37.37% is very good drains. So that the major part of the drainage systems around outlet-01 is inadequate because most of the drainage system is very old and there is no proper maintenance.

Table 6: Drainage Line From existing market to the Chuko Lemala Kale-Hwot churches around outlet-01

Drainage Type	Drainage Pavement	Existing Condition	Length(m)	Percent (%)
Open	Masonory	Light	234	13.55
Open	Masonory	Light	200	5.92
Open	Masonory	Severe	115	9.64
Open	Masonory	Severe	215	20.30
Open	Masonory	V-good	100	15.15
Open	Masonory	Severe	150	7.99
Open	Masonory	V-good	450	22.72
Open	Masonory	Light	80	4.73
Total	***Severe=37.93%***	***Light =24.2%***	***V-Good=37.87%***	***100%***

Table 6 reveals that of the total drains about 37.93% is severely degraded, and 24.2% is light. That means the major percentage of drainage system from existing market to the Aleta- Chuko Primary Hospital was underperformance. And results in the flood generated around this location cannot safely be discharged into the nearby outlet and run over road surfaces shown in (figure 8 right) above

Table 7: Drainage Line From Loko-ber to the slaughter house around Outlet -04

Drainage Type	Drainage Pavement	Existing Condition	Length(m)	Percent (%)
Open	Masonory	Severe	229	13.55
Open	Masonory	Light	100	5.92
Open	Masonory	Severe	163	9.64
Open	Masonory	Severe	343	20.30
Open	Masonory	Good	256	15.15
Open	Masonory	Severe	135	7.99
Open	Masonory	Good	384	22.72
Open	Masonory	Light	80	4.73
Total	***Severe=31.18%***	***Light =48.52%***	***Good=20.30%***	***100%***

From the above table conditions of the urban drainage system around Loko-ber were 31.18% with total collapse, 48.52% with shape deteriorate, and 20.30% with good condition. Hence, the major drainage systems were underperformance to passe the runoff i.e. needs reconstruction severe conditions and maintenance for light and good conditions case.

Table 8: Drainage Line From Wofcho-tera to the Bus station around Outlet -05

Drainage Type	Drainage Pavement	Existing Condition	Length(m)	Percent (%)
Open	Masonory	Severe	297	13.55
Open	Masonory	Severe	103	5.92

Open	Masonory	Light	215	9.64
Open	Masonory	Light	226	20.30
Open	Masonory	Good	148	15.15
Open	Masonory	Good	214	7.99
Open	Masonory	Light	110	22.72
Open	Masonory	Light	80	4.73
Total	***Severe=44.15%***	***Light =29.86%***	***Good=25.99%***	***100%***

As presented in Table-8, of the total drains about 44.15% is severely degraded, 29.86% is light, 25.99% is good. As a result of the drainage condition, the majority of drainage canals were with severe and light conditions, which resulted in inadequate performance.

4.1.1. Causes of the Drainage Structure Failure and Associated Problems of Study area

4.1.1.1. Topography and Soil Type of Study Area

Study town catchment areas that contribute to runoff in the main discharging direction have oval kinds of shapes. That runoff from the upper land drain to the central part of the town contributes much runoff to the lower stream. And also Most of the soil in the study area is grouped under the Soil Group: - Clay loam, silty clay loam, silty clay, or clay. Soils of the study area have a high runoff potential due to a very slow infiltration rate. These soils primarily consist of clays with high swelling potential, soils with permanently-high water tables, soils with a claypan or clay layer at or near the surface, and shallow soils over nearly impervious parent material.

According to site observation, the drainage system provided along the local streets of the study area doesn't consider the sloping character of the area. Additionally, in the town which has flat to rolling terrain, the runoff and flood are expected to continue for some time after the rainfall stops. Therefore, stormwater in this area creates different impacts like flooding the houses and overflowing on the local and main streets.

4.1.1.2. Lack of Community Awareness and Frequent Clearance of Drainage Lines

Community awareness is one of the best active measures for maintainable urban drainage management. Unfortunately, from the field survey was observed that the inhabitants thought that dumping wastes into stormwater drainage systems regardless of the effects that could cause to their environment.

Dumping solid waste materials into the drainage system was the challenge of the stormwater drainage system in Aleta-Chuko town. Typically it consists of manufacturing and building materials such as sand, stones, dumped soils, bottles, plastic, etc. As a result of dumping such solid wastes into drains, the drainage systems have been clogged and caused flooding over streets and drainage failures.

In general, it was realized that even though few people have the responsiveness, the local government bodies should create awareness among the communities and also should provide proper waste management techniques.

4.1.1.3. Poor Construction and Design of Storm Water Drainage Lines

Proper design and construction of drainage structures are vital components for road structures to function without traffic interruption. Appropriate hydrological analysis of the catchment area where the drainage structure will be constructed and appropriate hydraulic parameters should be determined. This could improve the design and sustainability of these drainage channels.

The existing conditions of the drainage system in the study area were surveyed in detail. The problems associated with drainage systems; are drainage systems have the inadequate capacity to carry stormwater and problems of slope in the ditches to convey runoff to the corresponding catchment area.

Generally, the performance of the current drainage system is poor because from the findings it is observed that the existing drainage systems of Aleta-Chuko Town are faced with a problem are above listed.

4.2. Intensity –Duration Frequency (IDF) Curve

4.2.1. Selecting the best-fit probability distribution

The rainfall frequency analysis is done using both Gumbel and log-Pearson type III methods as recommended by the ERA manual 2013. The result obtained is tabulated in (Appendix Table 9). Which distribution fits the theoretical probability distribution, the goodness of fit test using Easy-fit 5.6 professional software to test different methods was identified in the methodology part.

i. Easy fit 5.6 software

The GOF test conducted using Easy Fit 5.6 professional software and the Log-Pearson Type III distribution fits for the statistical value for all the three different test methods is lesser than that of the Gumbel values as tabulated below. That is, the Log-Pearson III method has proved to be a good fit in all three tests compared to Gumbel's Method. The statistics for both methods are calculated and the ranking is given below (Table 7).

Table 9: Goodness of Fit of Log-Pearson III and Gumbel Methods

Distribution	Kolmogorov-Smirnov		Anderson-Darling		Chi-Squared	
	Statistic	Rank	Statistic	rank	Statistic	rank
Log-Pearson III	0.09916	1	0.29132	1	0.63806	1
Gumbel min	0.18445	2	1.7152	2	1.477	2

ii. Correlation Coefficient

The coefficient of correlation suggested by the Ethiopian Drainage Design Manual (ERA, 2013), a distribution that has better R2 values was chosen as the goodness of fit tests. Accordingly, the Log-Pearson III is chosen for further analysis.

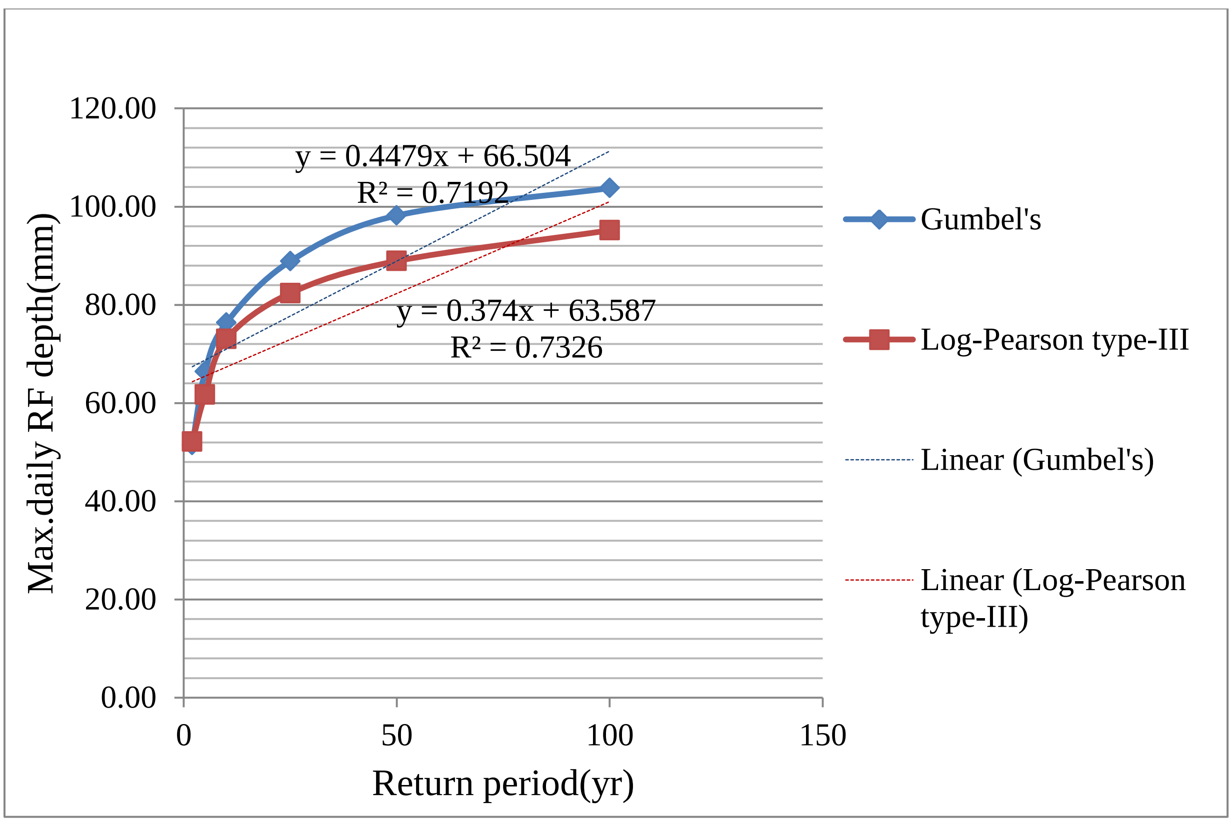

Figure 9: Correlation Coefficient of Probability distribution functions

A graph Return period Vs. Precipitation for both distributions is as shown above Log Person III fits for the distribution with $S2$=0.7326. Therefore, both methods show that the log Pearson type III distribution fits with the rainfall data used for this study. Accordingly, the Log-Pearson III is chosen for further analysis in Table 8 below.

Table 10: Log- Pearson III method for the study area

Return period(T)	Exceedance Probability(%)	Xmean	δ_z	Kz	δ_z* Kz	Z_T	X_T(mm)
2	50	1.715	0.117	0.017	0.002	1.72	52.12
5	20			0.644	0.075	1.79	61.70
10	10			1.27	0.149	1.86	73.04
25	4			1.72	0.201	1.92	82.37
50	2			2	0.234	1.95	88.92
100	1			2.25	0.263	1.98	95.17

4.2.2. Intensity –Duration Frequency (IDF) Curve of Study Area

The following IDF curve has been produced. The data obtained for the production of the IDF curve is the result of calculations using the reduction formula and it is tabulated in the (Appendix Table 13) . IDF curve of study area were formulated in figure 10 below.

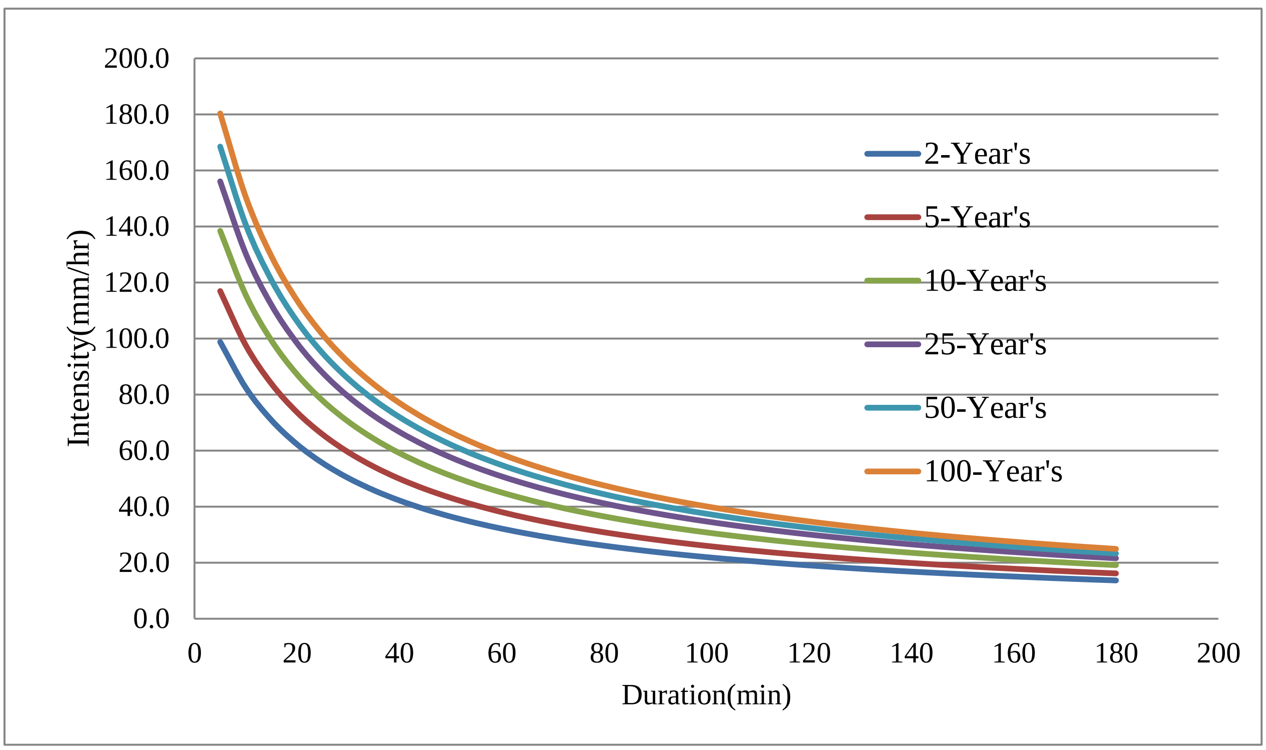

Figure 10: IDF Curve of Aleta-Chuko Town.

Ethiopian Road Authority (ERA) divides Ethiopian meteorological stations into different regions to develop Intensity-Duration-Frequency Curves. According to ERA divided into 9 (nine Regions) like A1, A2, A3, A4, B1, B2, C, and D. Aleta-Chuko Town found in Region B2 has been developed IDF curve by ERA and presented in its Drainage Design Manual, 2013. The IDF curve developed by ERA is presented here for comparison was shown below (Figure 11).

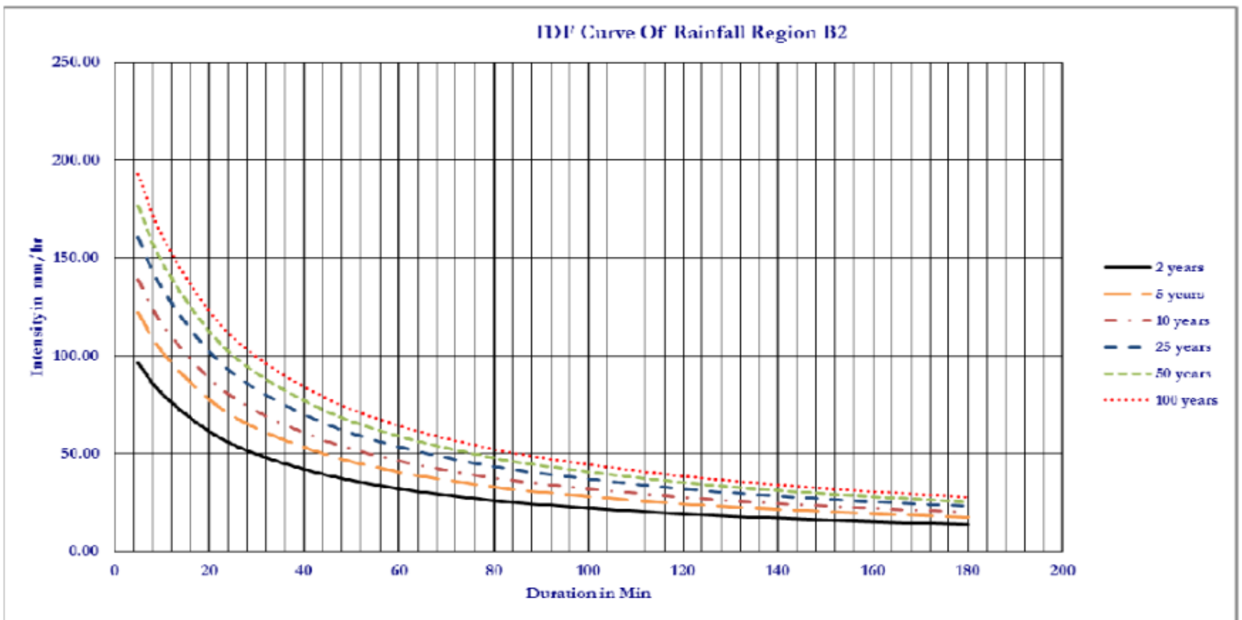

Figure 11: ERA, 2013 IDF Curve for Region B2

(Source; ERA, 2013 DDM).

Table 11: Comparison between IDF Curve developed for a Study area and ERA for Region B2

RF dur(min)	T-2		T-5		T-10		T-25		T-50		T-100	
	S-A	ERA	S-A	ERA	S-A	ERA	S-A	ERA	S-A	ERA	S-A	ERA
5	98.8	104.7	116.9	132.6	138.4	151.0	156.1	174.4	168.5	192.0	180.4	209.6
10	82.4	87.4	97.6	110.6	115.5	126.0	130.3	145.5	140.6	160.2	150.5	174.9
15	70.9	75.1	83.9	95.1	99.3	108.3	112.0	125.1	120.9	137.7	129.4	150.4
20	62.2	66.0	73.7	83.5	87.2	95.1	98.4	109.9	106.2	121.0	113.6	132.1
25	55.5	58.9	65.8	74.5	77.8	84.9	87.8	98.1	94.8	107.9	101.4	117.9
30	50.2	53.2	59.4	67.4	70.4	76.7	79.3	88.6	85.6	97.6	91.7	106.5
35	45.8	48.6	54.3	61.5	64.2	70.1	72.4	80.9	78.2	89.1	83.7	97.3

40	42.2	44.7	49.9	56.6	59.1	64.5	66.7	74.5	72.0	82.0	77.0	89.5
45	39.1	41.4	46.3	52.5	54.8	59.8	61.8	69.0	66.7	76.0	71.4	83.0
50	36.4	38.6	43.1	48.9	51.1	55.7	57.6	64.3	62.2	70.8	66.5	77.3
55	34.1	36.2	40.4	45.8	47.8	52.2	53.9	60.3	58.2	66.3	62.3	72.4
60	32.1	34.1	38.0	43.1	45.0	49.1	50.8	56.7	54.8	62.4	58.6	68.2
65	30.3	32.2	35.9	40.7	42.5	46.4	47.9	53.6	51.7	58.9	55.4	64.4
70	28.7	30.5	34.0	38.6	40.3	43.9	45.4	50.8	49.0	55.9	52.5	61.0
75	27.3	29.0	32.3	36.7	38.3	41.8	43.2	48.2	46.6	53.1	49.9	58.0
80	26.0	27.6	30.8	34.9	36.5	39.8	41.1	46.0	44.4	50.6	47.5	55.2
85	24.9	26.4	29.4	33.4	34.9	38.0	39.3	43.9	42.4	48.3	45.4	52.8
90	23.8	25.2	28.2	32.0	33.4	36.4	37.6	42.0	40.6	46.3	43.5	50.5
95	22.8	24.2	27.0	30.6	32.0	34.9	36.1	40.3	39.0	44.4	41.7	48.5
100	21.9	23.3	26.0	29.5	30.8	33.5	34.7	38.7	37.4	42.6	40.1	46.6
105	21.1	22.4	25.0	28.3	29.6	32.3	33.4	37.3	36.0	41.0	38.6	44.8
110	20.4	21.6	24.1	27.3	28.5	31.1	32.2	36.0	34.7	39.6	37.2	43.2
115	19.7	20.8	23.3	26.4	27.5	30.1	31.1	34.7	33.5	38.2	35.9	41.7
120	19.0	20.1	22.5	25.5	26.6	29.0	30.0	33.5	32.4	36.9	34.7	40.3
125	18.4	19.5	21.8	24.7	25.8	28.1	29.1	32.5	31.4	35.7	33.6	39.0
130	17.8	18.9	21.1	23.9	25.0	27.2	28.2	31.5	30.4	34.6	32.5	37.8
135	17.3	18.3	20.5	23.2	24.2	26.4	27.3	30.5	29.5	33.6	31.6	36.7
140	16.8	17.8	19.9	22.5	23.5	25.6	26.5	29.6	28.6	32.6	30.6	35.6
145	16.3	17.3	19.3	21.9	22.8	24.9	25.8	28.8	27.8	31.7	29.8	34.6
150	15.9	16.8	18.8	21.3	22.2	24.2	25.1	28.0	27.0	30.8	28.9	33.6
155	15.4	16.4	18.3	20.7	21.6	23.6	24.4	27.3	26.3	30.0	28.2	32.8
160	15.0	15.9	17.8	20.2	21.1	23.0	23.8	26.5	25.6	29.2	27.5	31.9
165	14.7	15.5	17.3	19.7	20.5	22.4	23.2	25.9	25.0	28.5	26.8	31.1
170	14.3	15.2	16.9	19.2	20.0	21.9	22.6	25.2	24.4	27.8	26.1	30.3
175	14.0	14.8	16.5	18.7	19.6	21.3	22.1	24.6	23.8	27.1	25.5	29.6
180	13.6	14.5	16.1	18.3	19.1	20.8	21.5	24.1	23.3	26.5	24.9	28.9

Note: RF dur = Rainfall duration, S-A= Study Area, ERA= Ethiopian Road Authority

It can be seen from the two IDF curves (the one developed in this study and that of the IDF curve given in the ERA drainage design manual) and the above table that the IDF curve developed by ERA for Region B2 and self-developed have differences in this study. The IDF curve from the road authority manual used rainfall data up to the year 2003 only. Whereas this study used 30-year daily rainfall data from 1991 to 2019. Notice that each value in Table-7 above for self-study is smaller than that of the correspondent values of ERA's IDF. It would be said that the use of ERA's IDF curve is safe for design purposes but it could be uneconomical.

4.3. Hydraulic Capacity of Existing Drainage Structure of Study Area

4.3.1. Delineation and Division of Catchment Area

ArcGIS is used to digitize the contributing land-use area to the drain line based on the principles of the direction of flow and the contour line. The flow of water follows the slope of the topography and can be diverted by an artificial canal. The common outlet point is one of the important parameters we should consider for the classification of our study area into sub-catchment. Generally, the topographic slope, contour line, and the position of major drainage lines were considered for the delineation of the catchments. For this study ArcGIS 10.3, was used for catchment delineation (Figure12) below.

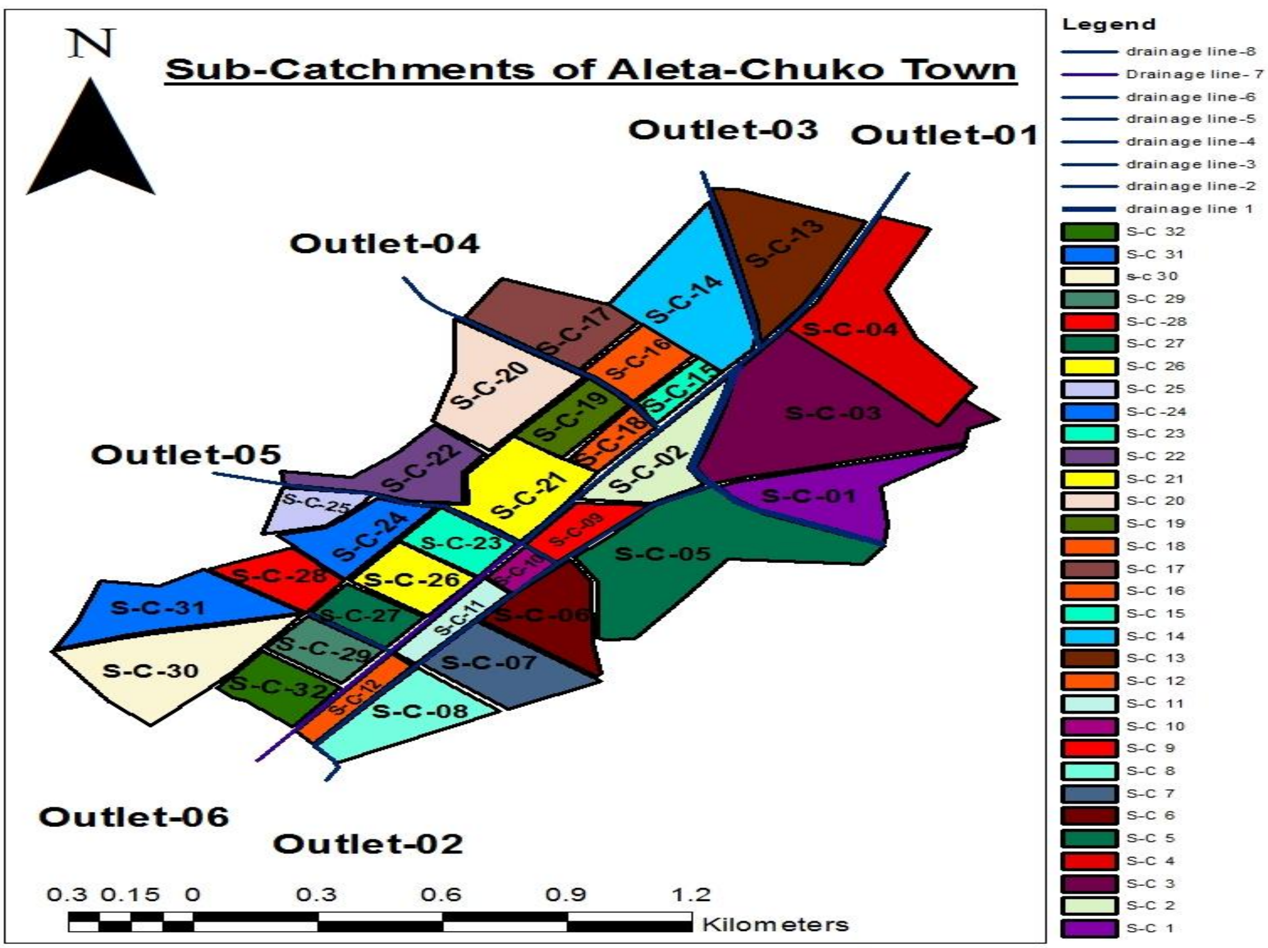

Figure 12: Selected sub-catchment area and drainage network

4.3.2. Rational Method

Rational Method was used for this study area to determine peak runoff on each sub-catchment which is normally only recommended by ERA Manual 2013 for catchments less than about 50 hectares. The weighted runoff coefficient (Cw), percentage of imperviousness, length of conduit/canals, and slope of land were required to estimate peak discharge.

The runoff coefficient (Cw) is the most important variable in the rational method of rainfall to runoff transformation. For this study area calculated runoff coefficient on each catchment with the respective land use composition and time of concentration is calculated which consists of an inlet time plus the time of flow in a closed conduit or open channel to the design point to know the intensity of the rainfall in the (Appendix table 17). and then peak flood discharge is calculated using equation (3.5). as presented in table 11 below.

Table 12: Runoff determination using a rational method for the study area

Sub-Catch No and Code	Area(Ha)	Weighted Coefficient(Cw)	Intensity 10yr	Intensity 25yr	Q(m3/s) 10yr	Q(m3/s) 25yr
S-C-01	14.5	0.66	99.3	112.0	1.33	1.6
S-C-02	21	0.63	99.3	112.0	1.1	1.3
S-C-03	8.5	0.63	85.32	96.3	2.9	3.6
S-C-04	6.1	0.75	99.3	112.0	2.6	3.3
S-C-05	18.2	0.64	99.3	112.0	2.0	2.5
S-C-06	6.5	0.64	99.3	112.0	1.1	1.4
S-C-07	7.20	0.63	99.3	112.0	1.1	1.4
S-C-08	7.0	0.56	99.3	112.0	1.0	1.2
S-C-09	3.0	0.75	99.3	112.0	0.5	0.7
S-C-10	1.40	0.75	99.3	112.0	0.3	0.4
S-C-11	2.50	0.75	99.3	112.0	0.3	0.4
S-C-12	2.5	0.65	99.3	112.0	0.4	0.6
S-C-13	11	0.63	99.3	112.0	1.7	2.1
S-C-14	11.7	0.63	99.3	112.0	1.9	2.3
S-C-15	2.0	0.75	99.3	112.0	0.4	0.5
S-C-16	3.9	0.65	99.3	112.0	0.6	0.7
S-C-17	7.8	0.49	99.3	112.0	1.1	1.3
S-C-18	2.2	0.75	99.3	112	0.5	0.6

S-C-19	4.0	0.58	99.3	112	0.6	0.8
S-C-20	9.6	0.57	99.3	112	1.3	1.6
S-C-21	7.5	0.72	99.3	112	1.0	1.3
S-C-22	6.60	0.70	99.3	112	1.0	1.2
S-C-23	3.60	0.72	99.3	112	0.7	0.8
S-C-24	5.10	0.71	99.3	112	0.9	1.1
S-C-25	2.90	0.64	99.3	112	0.4	0.5
S-C-26	4.30	0.70	99.3	112	0.8	0.9
S-C-27	3.30	0.70	99.3	112	0.5	0.6
S-C-28	3.80	0.64	99.3	112	0.5	0.6
S-C-29	3.70	0.69	99.3	112	0.6	0.7
S-C-30	11.5	0.65	96.9	109.3	1.4	1.7
S-C-31	8.5	0.62	96.9	109.3	1.3	1.6
S-C-32	4.20	0.72	99.3	112	0.8	1.0

4.3.3. Adequacy and Hydraulic Capacities of the Existing Drainage Structure

Based on the hydraulic calculation of the result drainage capacity of the existing system was checked by using Manning's equation and presented in Appendix Table 18 to compare with the estimated discharge by determining the peak discharge of 10 year return period for each existing sub-catchment from the measured depth and width of the stormwater drainage channel for all sample sub-catchments of the town.

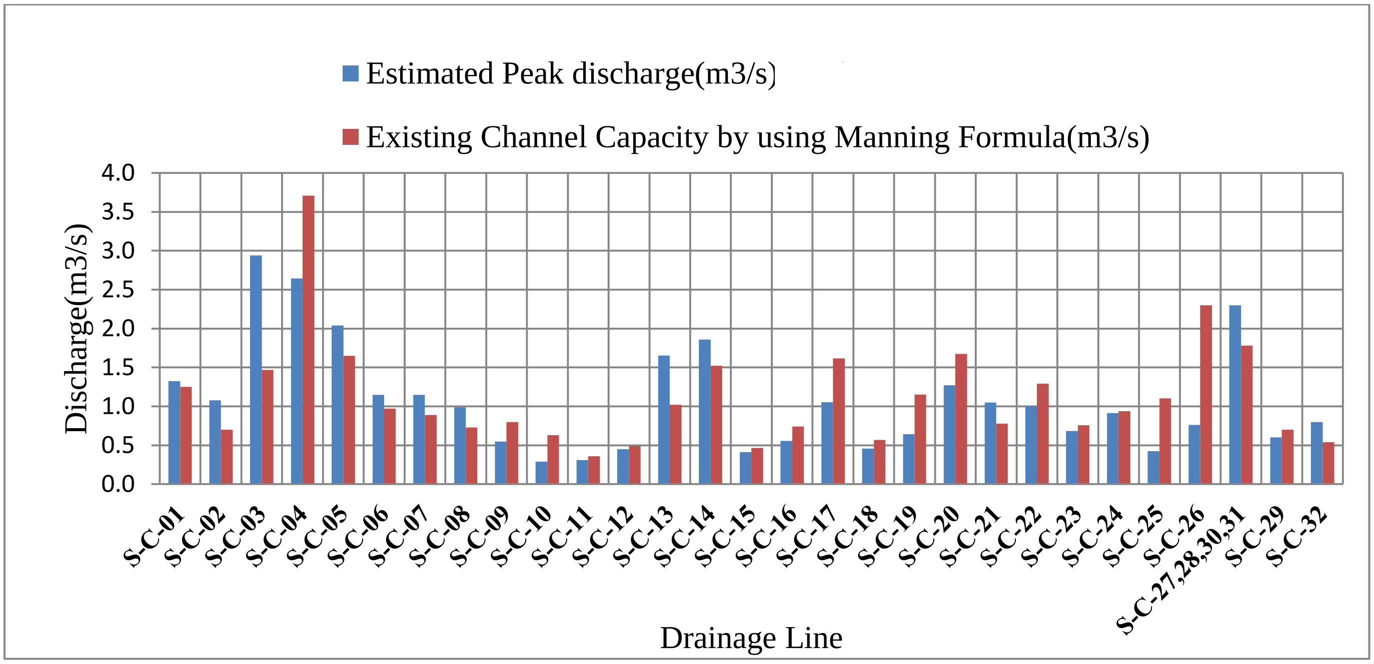

Figure 13: Comparison of existing channel capacity and estimated peak discharge.

From the above Figure, a number of the existing drainage lines are undersized, therefore not able to handle the required capacity of a 10-year storm occurrence. Typical flooding during the rainy season are: S-C-02, S-C-03, S-C-13, S-C-14, S-C-27, S-C-28, S-C-30, and S-C-31. The storm overtops the drainage system. In another hand, some stormwater drainage systems were oversized.

4.4. SWMM5.1 Modelling

As previously mentioned, the modeled area is divided into 32 sub-catchments and flows from asphaltic roads and walkways, and pavements. The network consists of 28 nodes(Junctions), 6 outfalls, and 28 links(Conduit), see Figure below.

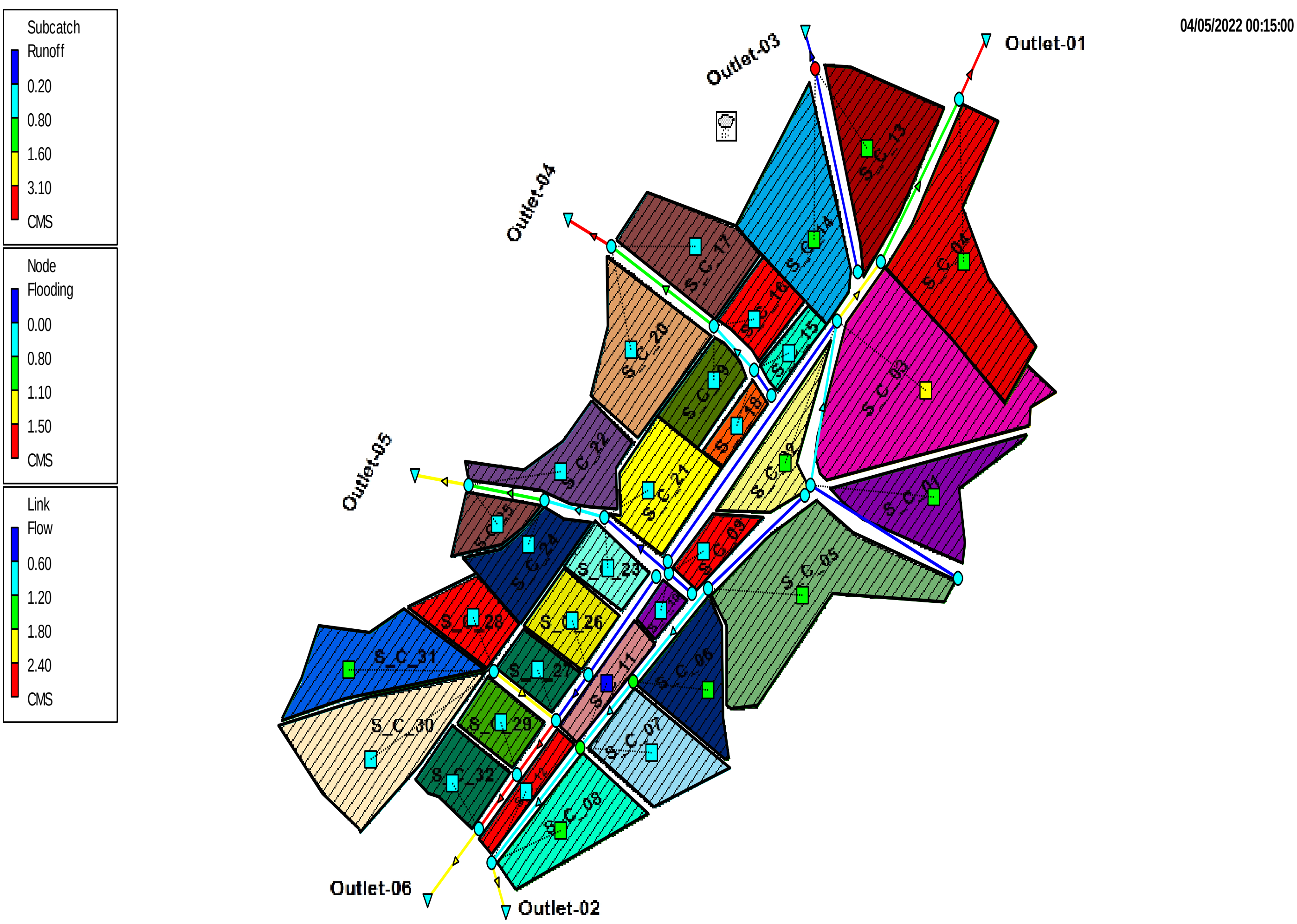

Figure 14: Map of Sub-Catchments by SWMM

4.4.1. Water Level at System Nodes(Junctions) and links(Conduits)

The Aleta-Chuko Town sample watersheds and their characteristics are presented by using the SWWM. The simulation results were obtained based on the duration of the 3hrs rainfall intensity. So that the simulation results at junctions and Links were presented below.

4.4.1.1. Water Level and Flood Level in the Junctions

I. Drainage Junction around Local market and Highschool

The designed Water level of the drainage canal of junctions 2 and 3 is 0.8 and 1.2m respectively. The flooding level attained with 3hrs rainfall intensity injunction 2 is 0.31 m and injunction 3, 1.4m. As can be read from the graph the maximum designed water level drainage canal of junction 2 was sufficient and there is no overflow. And, the maximum flooding level of junction 3 is greater than the designed water level of drainage canal depth. i.e this junction is incapable to carry entering stormwater as presented in the figure below.

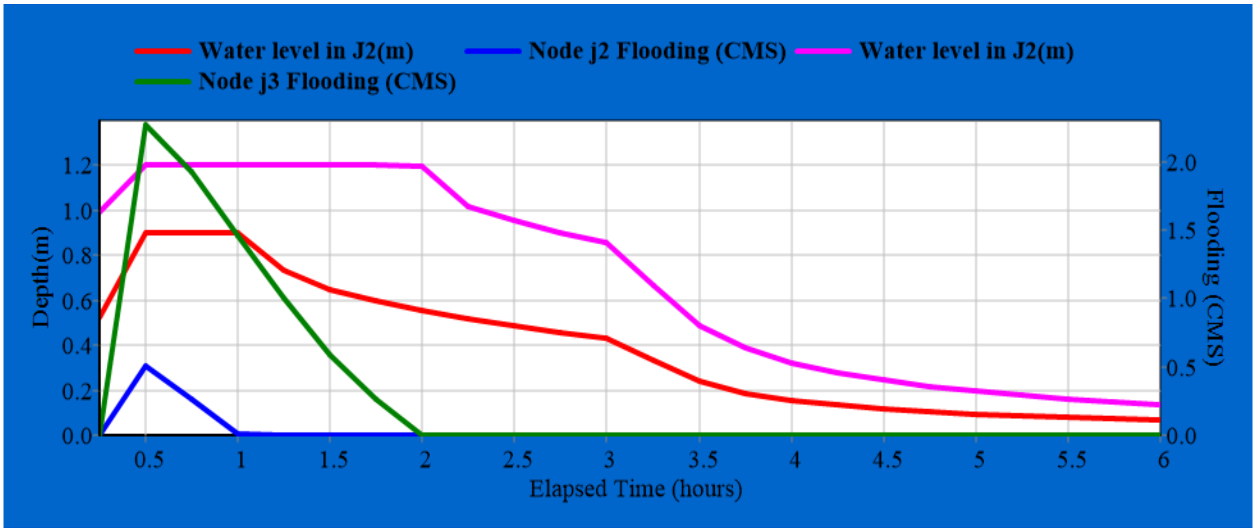

Figure 15: Water level and flood between Local market and Highschool.

II. Drainage Junction around Primary Hospital

At junctions 7 and 8 as shown (figure 16) below, the water level for both was 0.6. As a result of simulation at junctions, 7 and 8 are 0.7 and 0.5 respectively. These indicate that junction 7's designed canal depth was insufficient and there is an overflow of the flood around Aleta-Chuko Primary Hospital. But, around junction 8 designed drainage canal is sufficient and there is no overflow as a result of the model.

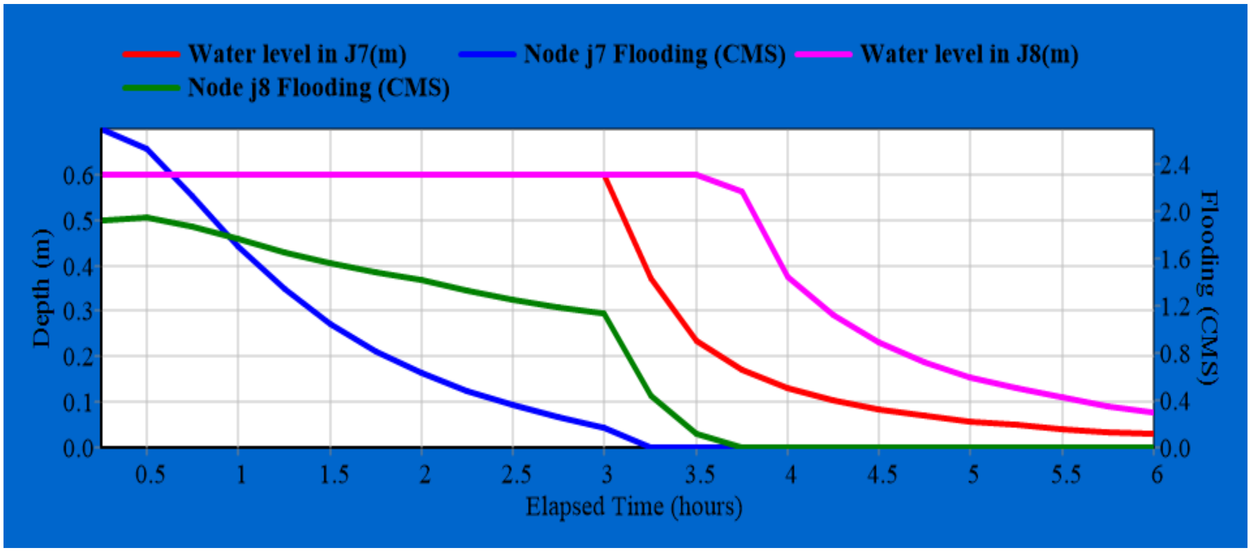

Figure 16: Water level and flood around the primary hospital

III. Drainage Junction around Lemela Kale-Hweyt Church and Stadium

Figure 17 below, the water level of the drainage canal at junctions 9 and 12 is 1m and 0.8m respectively. In front of Lemela Kale-Hweyt Church injunction 9, the maximum flooding level for the 3hrs of rainfall intensity is less than the maximum designed water level. But, around the stadium at junction 12 maximum flooding level is greater than the designed water level. i.e. maximum designed water level drainage canal of junction 12 was insufficient and the occurrence of overflow.

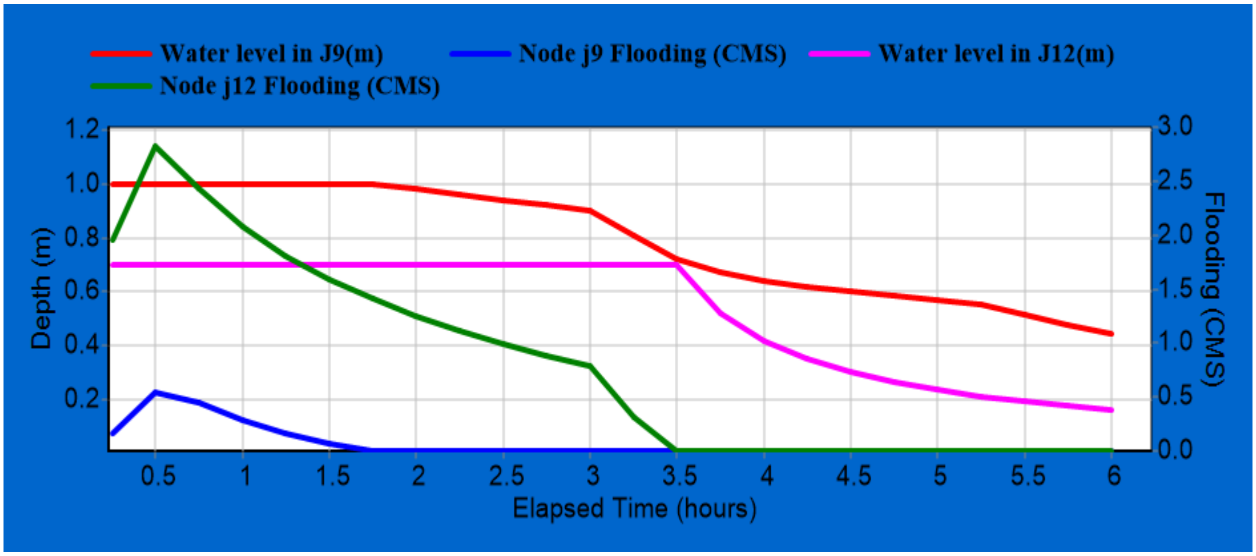

Figure 17: Water level and flood around Lemela Kale-Hweyt and Stadium

IV. Drainage Junction around Slaughter House and Coffee Site

The designed water level of drainage at junctions 16 and 25 is 0.9m and 0.7m respectively. The flooding level attained with 3hrs rainfall intensity at junction 16 is 0.18 m and at junction 25, 0.78m. Based on the simulation result of the model for the current land use condition, the designed canal depth was sufficient for junction 16 and insufficient for J25.

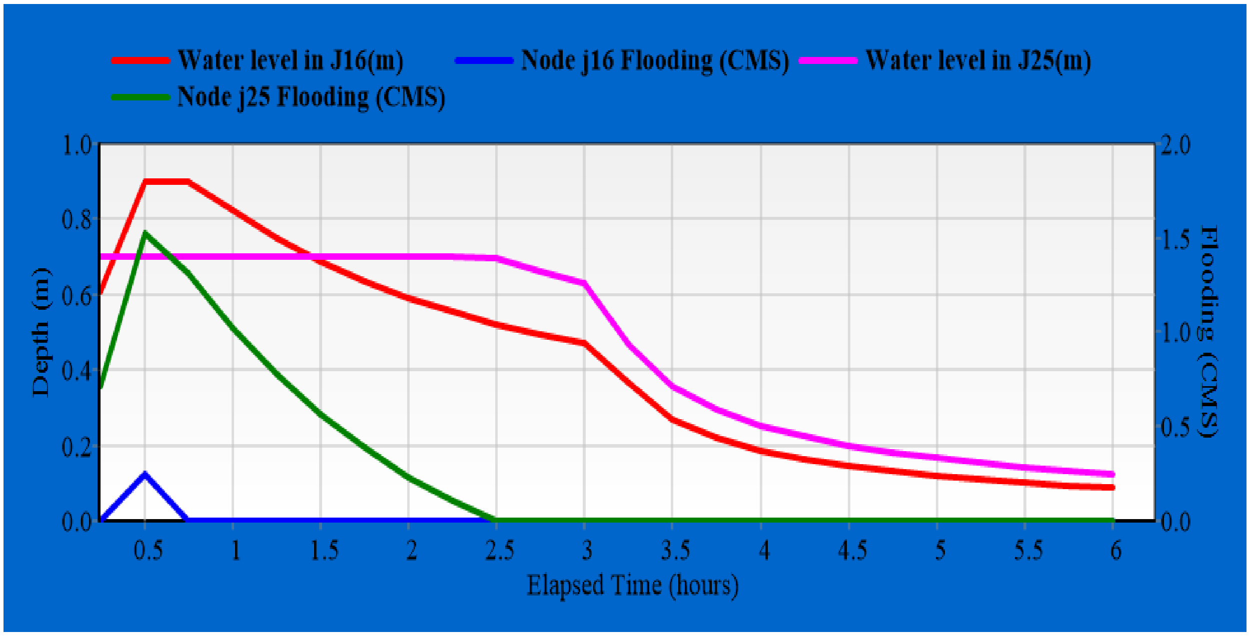

Figure 18: Water level and flood around Slaughter House and Coffee Site

4.4.1.2. Water depth and Flow in the Conduits

I. Drainage towards Outfall-01

As it was observed in the field survey and SWWM model simulation result indicates that junctions 2 and 3 were flooded due to insufficient design water depth. Consequently, the link (C2) is also insufficient to carry the generated runoff. But the drainage canal found downstream part of link Junction 1(link C1) is inadequate to convey runoff as shown in the figure below.

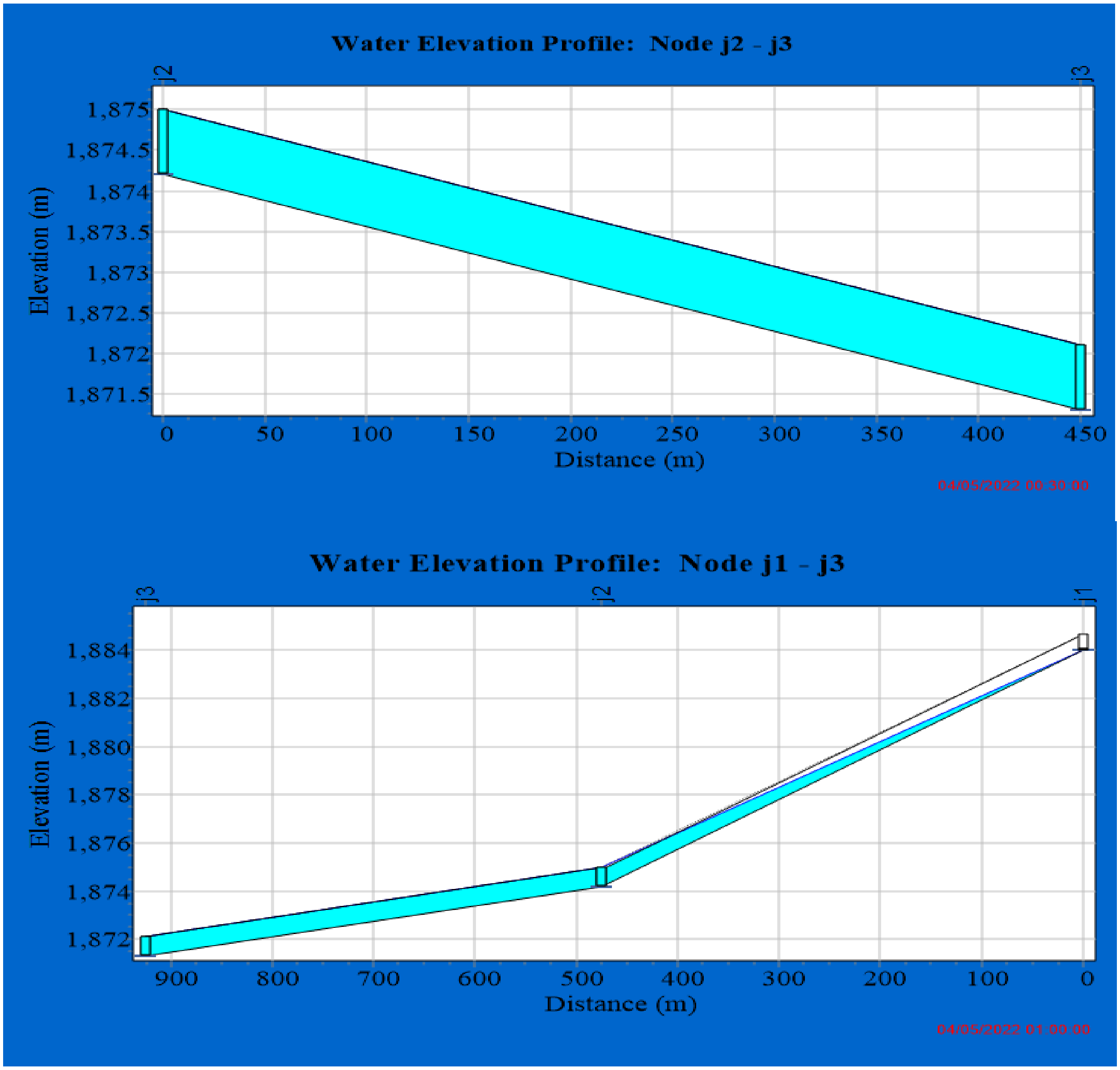

Figure 19: Water Elevation Profile towards outlet-01

II. Drainage towards Outfall-02

As shown in figure 20 below that a flow within the link and nodes towards Outfall-02.The simulation result indicates that both junctions (7 and 8) are flooded due to insufficient design water depth. the link between two junks (C21) is also busy at this time and the overflow of runoff has happened.

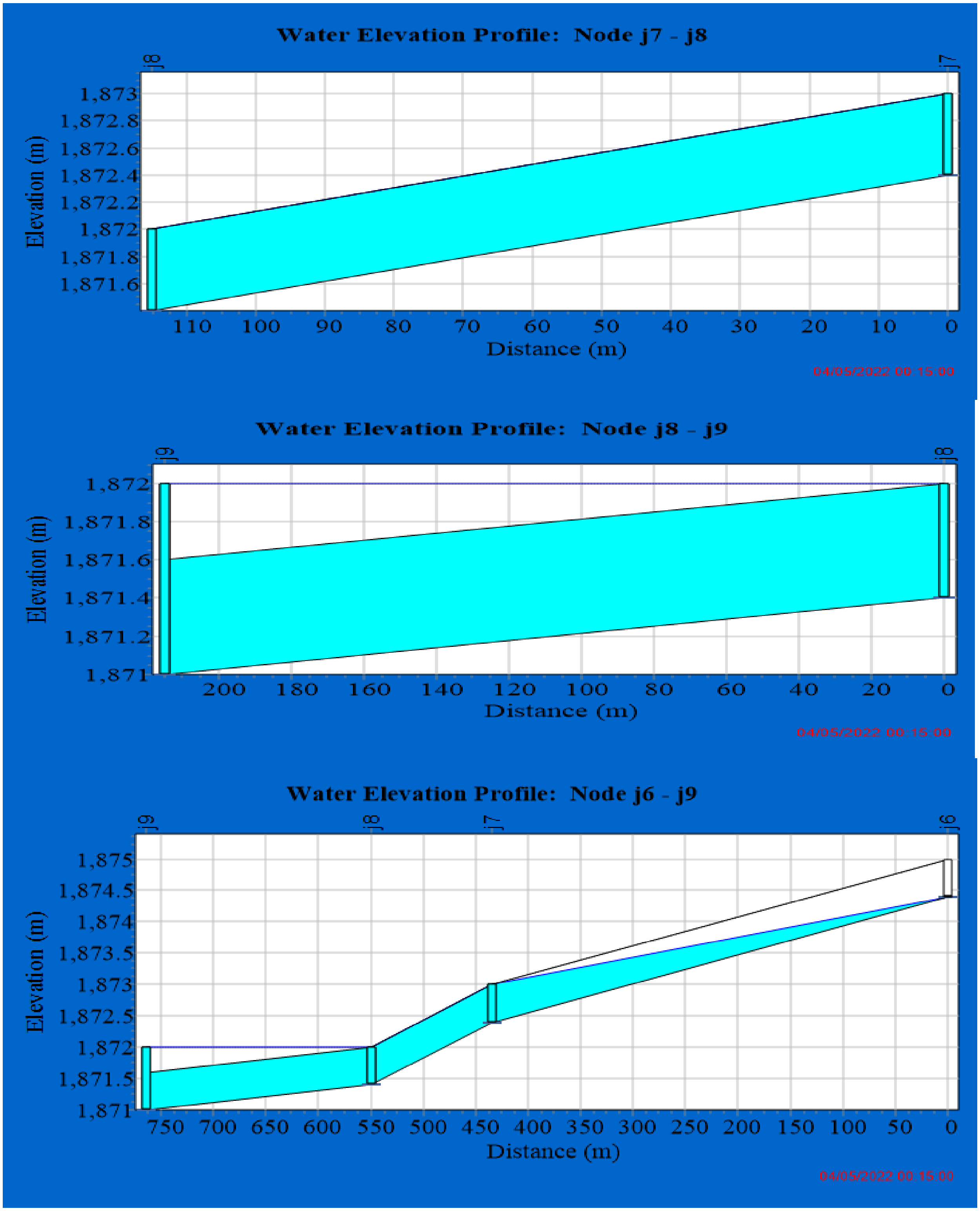

Figure 20: Water Elevation Profile towards outlet-02

III. Drainage around outlet-04

The drainage canal found an upstream part of the Outfall-04 outfall link(C16) as the simulation result indicates the junctions (17) are insufficient to carry the generated runoff and also there is flooding as shown in figure 21 below.

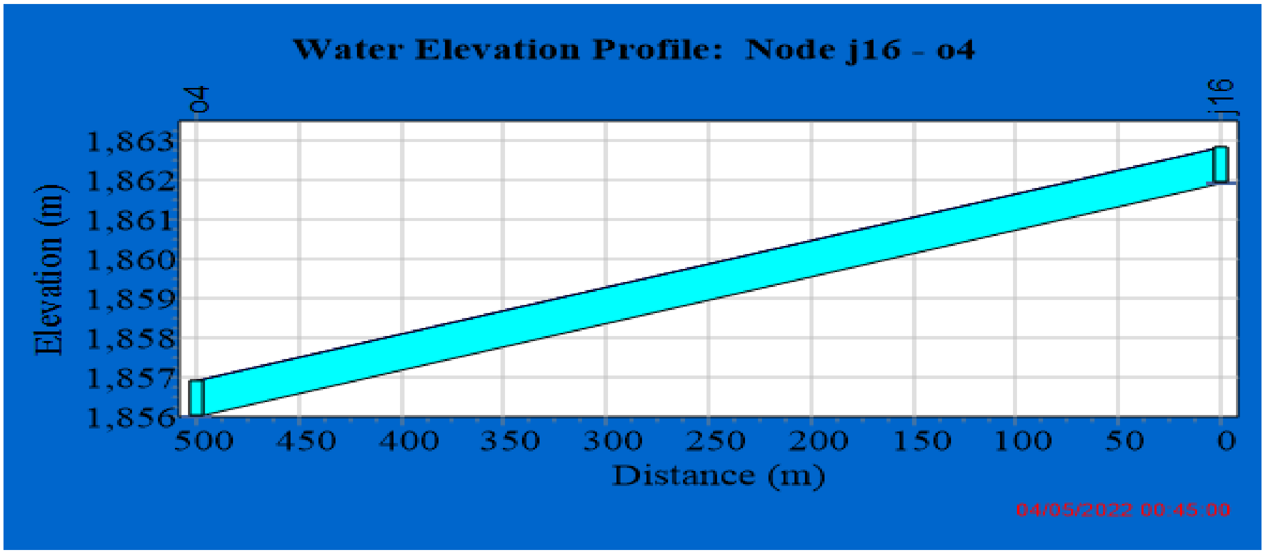

Figure 21: Water Elevation Profile towards outlet-04

IV. Drainage around Coffee Site

As the simulation result indicates, the flow level in the drainage canal at the upstream part of junction J25 (link C23) and through J5 was insufficient to carry the generated runoff (Figure 22).

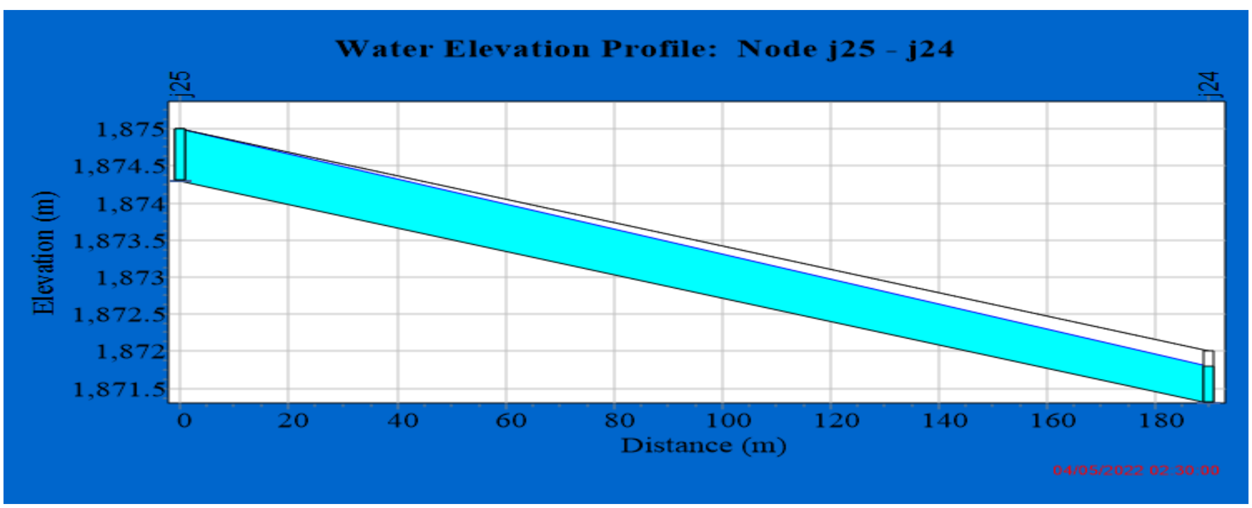

Figure 21: Water Elevation Profile towards outlet-06

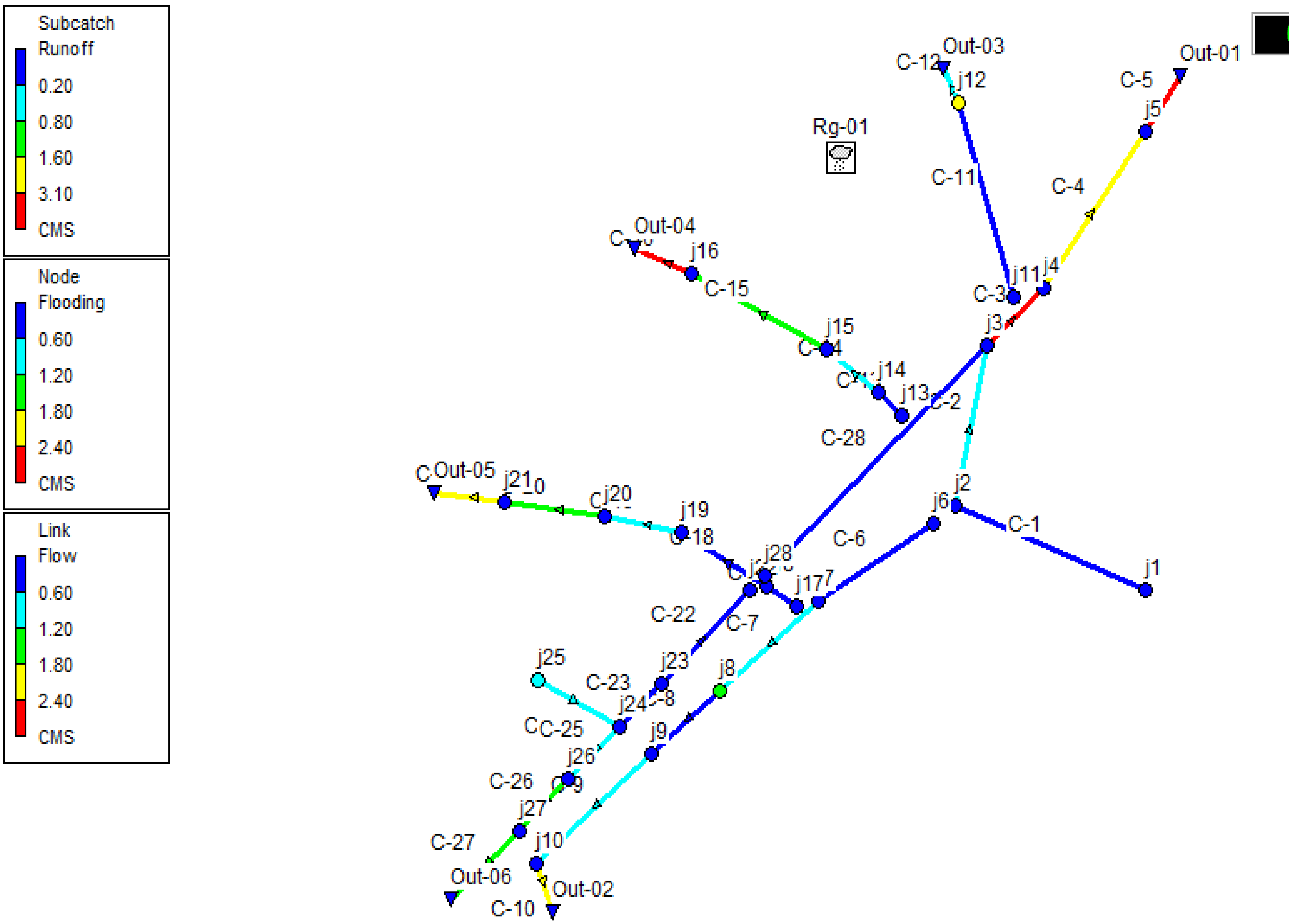

Figure 23: junctions and links of the study area

4.4.2. Comparison of Peak Discharge Simulated with SWWM and Rational Method

The simulated discharges values of all sub-catchment with SWWM compared with peak discharges by rational method for 10 year return period in Table 12 below.

Table 13: Comparison of Peak discharge with Simulated SWWM and rational method

Sub-Catch No and Code	SWWM 5.1	Q(m3/s) 10yr	Q(m3/s) 25yr
S-C-01	1.40	1.33	1.6
S-C-02	1.12	1.1	1.3
S-C-03	3.02	2.9	3.6

S-C-04	1.81	2.1	2.6
S-C-05	2.20	2.0	2.5
S-C-06	1.17	1.1	1.4
S-C-07	1.11	1.1	1.4
S-C-08	1.05	1.0	1.2
S-C-09	0.5	0.5	0.7
S-C-10	0.26	0.3	0.4
S-C-11	0.35	0.3	0.4
S-C-12	0.32	0.4	0.6
S-C-13	1.61	1.7	2.1
S-C-14	1.88	1.9	2.3
S-C-15	0.43	0.4	0.5
S-C-16	0.54	0.6	0.7
S-C-17	1.01	1.1	1.3
S-C-18	0.45	0.5	0.6
S-C-19	0.74	0.6	0.8
S-C-20	1.27	1.3	1.6
S-C-21	0.88	1.0	1.3
S-C-22	0.99	1.0	1.2
S-C-23	0.56	0.7	0.8
S-C-24	0.72	0.9	1.1
S-C-25	0.32	0.4	0.5
S-C-26	0.70	0.8	0.9
S-C-27	0.48	0.5	0.6
S-C-28	0.57	0.5	0.6
S-C-29	0.55	0.6	0.7
S-C-30	1.28	1.4	1.7
S-C-31	1.38	1.3	1.6
S-C-32	0.83	0.8	1.0
Total	31.5	32.13	

The total runoff from whole catchments by SWMM is 31.5m3/sec, whereas by a rational method is 32.13m3/sec Results of simulation show that SWMM was compatable with study area parameter’s for existing land use land cover and 3-hr rainfall intensity, relatively the same runoff values as compared to Rational Method. Therefore, it was well matched and acceptable in this study as shown Figure 24 below.

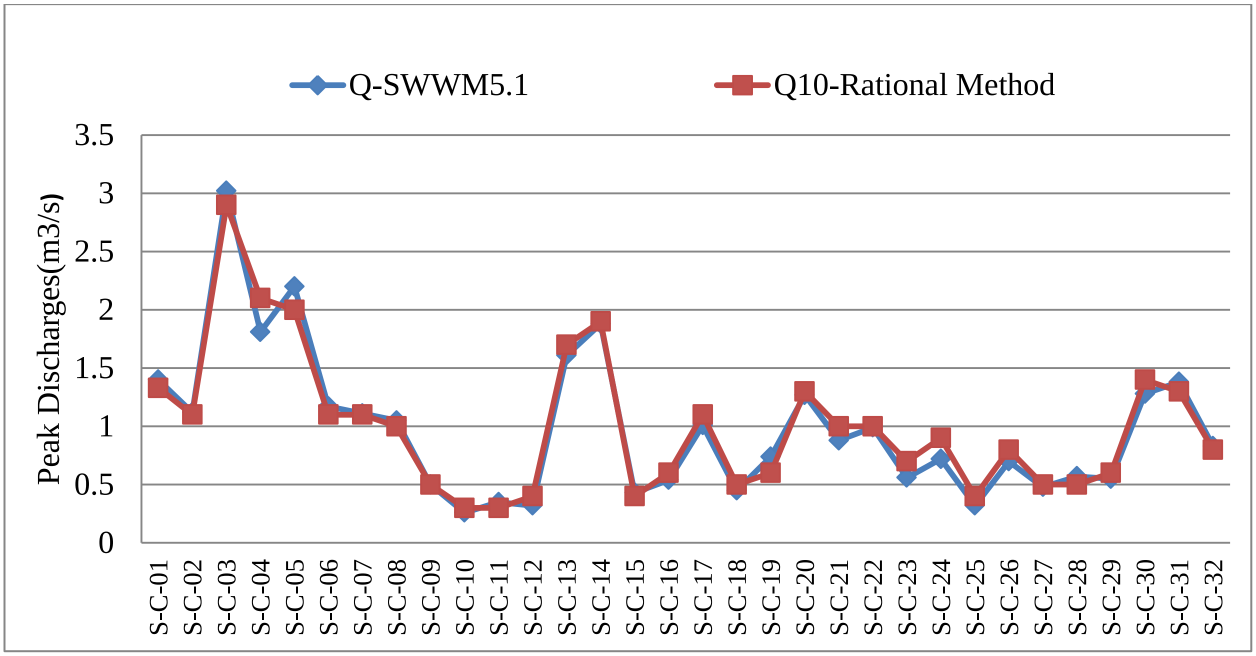

Figure 24: Comparison of peak discharge with SWMM5.1 and Rational method

4.2. Low Impact Development Simulation

In the model, two types of LID (Bio retention & Infiltration trench) measures were applied in different sub-catchments. These measures are selected due to their peculiar characteristic to obstruct a large amount of stormwater runoff and filtering pollutants as well. Parameters of both structures are specified based on the different physical characteristics of the sub-catchment. SWMM manual provides tables, formulas, etc. to compute values for each parameter. Berm height & area of structures were computed based on the expected amount of stormwater to be accumulated and diverted. After the simulation of the model, the impact of LID application induced on the drainage system is examined.

Table 14: Details of LID measures

List of Sub-Catch.	Bio retention				Infiltration trench			
	Pcs	Length (m)	Width (m)	Total Area (m^2)	Pcs	Length (m)	Width (m)	Total Area (m^2)
S-C-01	1	174	10	1740				
S-C-02					1	176	2.67	469.92
S-C-03	1	203	10	2030	1	143	2.67	648.81
S-C-04					1	178	2.67	475.26
S-C-05	1	182	10	1820	1	188	2.67	501.96
S-C-06					1	176	2.67	469.92
S-C-07	1	185	10	1850				
S-C-08	1	160	10	1600				
S-C-13					1	181	2.67	483.27
S-C-14	1	106	10	1060	1	110	2.67	293.7
S-C-20					1	166	2.67	443.22
S-C-30					1	188	2.67	501.96
S-C-31	1	173	10	1730				
Total				13,430				3818.1

4.5.1. Runoff Comparison before and after LID applied at selected sub-catchment

To evaluate the change before and after the application of LID measures, the model simulation value in figure 18 outputs of peak runoff from sub-catchments and peak discharge from conduits along with the peak discharge time was compared in areas where all LID measures are assumed to be applied 10 years return period were taken into consideration. The technique results before applying LID with compared the value after applied infiltration trenches and bio-retention are shown in the table 15 below to model the drainage system network.

Table 15: Comparison of Peak runoff before and after LID applied.

List of Sub Catchment	Before LID applied Peak Runoff value Q (m^3/s)	After LID applied Peak Runoff LID value Q (m^3/s)	Peak Runoff reduction (%)
S-C-01	1.40	1.27	9.28
S-C-02	1.12	0.97	13.4
S-C-03	3.02	2.48	17.88
S-C-04	1.81	1.51	16.57
S-C-05	2.2	1.63	25.91
S-C-06	1.17	0.81	30.76
S-C-07	1.11	0.87	21.62
S-C-08	1.05	0.8	23.80
S-C-13	1.61	1.3	19.25
S-C-14	1.88	1.3	30.85
S-C-20	1.27	1.06	16.53
S-C-30	1.28	1.08	15.65
S-C-31	1.3	1.11	14.61

Based on the above table 14 analysis associated with 10 years return period, LID application helps to decrease the magnitude of peak runoff selected sub-catchment minimum of 9.28% and a maximum of 30.85%. This situation enables significantly improves the drainage system performance of the town if it is done extensively and properly.

5 SUMMARY AND RECOMMENDATIONS

5.1. Summary

This study assessed the performance of the stormwater drainage system of Aleta-Chuko Town especially Kebele-01 and Kebele-02. Currently, the stormwater drainage systems of town Aleta-Chuko was not efficient as a result of drainage systems were not well connected, unavailability of drainage systems in a proper place, and most ditches not having a proper slope. In addition, there was no adequate community awareness to the issue. As a result, solid and liquid wastes were directly disposed into the storm drainage system and inadequate to convey the peak discharge for the required design period, the drainage system was filled with sediment and other plastic materials. From the field survey, As an assessment of the existing drainage system by using GTZ Standards more percentage of drainage structure lines were severely degraded in the study area.

The adequacy of drainage systems was estimated through hydrological analysis, hydraulic analysis, and rational method. Considering the current land use, rainfall intensity, and catchment area, the peak discharge was estimated using the rational formula. The Manning equation was used for fixing the dimensions of the storm drainage canals.

Storm Water Management Model (SWMM 5.1) allows the catchment area to be subdivided into sub-catchments. For this study area is subdivided into 32 sub-catchments with 6 outfalls. The simulated result indicates that in a number of the junctions the flood level was greater than the designed water level. For instance, the drainage conduits and junctions were insufficient in (C2, C7, C8, C47) and (J2, J3, J7, J8) respectively. The total runoff from whole catchments by SWMM is 31.5m3/sec, whereas by a rational method is 32.13m3/sec. Results of simulation show that SWMM was compatable with study area parameter's for existing land use land cover and 3-hr rainfall intensity, relatively the same runoff values as compared to Rational Method.

Among various LID options, bio retention cell and infiltration trench were identified as the best choice for this area because of their effectiveness in reducing peak flow and runoff as well as their low costs. Further, they occupy very small areas and require very little maintenance. infiltration trench and bio retention cell were simulated as LID practices in 13 sub catchments. It was estimated that suggested LID practices, LID application decrease the magnitude of peak runoff selected sub-catchment minimum of 9.28% and a maximum of 30.85% under rainfalls corresponding to return period 25-year storm events can decrease peak flow compared to the existing conditions without LID. These results show that LID is valuable consideration for urban flood control as a storm water management planning tool.

5.2. Recommendation

Based on this study result, appropriate mitigation measures are recommended. In Aleta-Chuko Town inadequacy of storm drainage system and structures have had a serious negative impact on the community at home and road. To avoid these problems, the following appropriate mitigation measures are recommended;

- Urban drainage systems not be designed isolated from the communities that they serve. Communities construct houses on drainage pathways and floodplains, it may be necessary to relocate some of the houses for the construction of drains. So, the drainage system and community service should be isolated.
- Local government bodies should create awareness among the communities, remove solid and waste materials from drainage canals, and repair damage either partially or fully during a non-rainfall season.
- Establish well-connected drainage lines, and waste management techniques to safely release wastewater, to properly manage and use stormwater drainage lines as it were their objective.
- When the drainage system is designed the slope of ditches should be more considered.
- Modify drainage channels dimension around primary hospital, stadium, slaughterhouse, and coffee site. And also new drainage lines in the proper place to reduce peak runoff and flooding hazards happened in the area
- Design and implementation of stormwater drainage system network around areas without a drainage system. for example around the coffee store, Genet church, Bariso coffee site, back of City Administration of Mayor office, and agricultural bureau.

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APPENDICES

Appendix Table 1. Max. daily rainfall computation from 1990-2019

Year	Max Daily Rainfall	Descending Order	Rank	Y=Log X	Y_o-Y_m	$(Y_o-Y_m)^2$	$(Y_o-Y_m)^3$
1990	49.2	121.2	1	1.691965	-0.02270	0.00052	-0.00001
1991	50.6	70	2	1.704151	-0.01051	0.00011	0.00000
1992	69.7	69.7	3	1.843233	0.12857	0.01653	0.00213
1993	69.7	69.7	4	1.843233	0.12857	0.01653	0.00213
1994	48.2	66.3	5	1.683047	-0.03162	0.00100	-0.00003
1995	36.2	65.2	6	1.558709	-0.15596	0.02432	-0.00379
1996	65.2	65	7	1.814248	0.09958	0.00992	0.00099
1997	60.1	63.5	8	1.778874	0.06421	0.00412	0.00026
1998	58.3	60.1	9	1.765669	0.05100	0.00260	0.00013
1999	36.8	59.5	10	1.565848	-0.14882	0.02215	-0.00330
2000	33.2	58.3	11	1.521138	-0.19353	0.03745	-0.00725
2001	66.3	58	12	1.821514	0.10685	0.01142	0.00122
2002	45.6	58	13	1.658965	-0.05570	0.00310	-0.00017
2003	41	56.6	14	1.612784	-0.10188	0.01038	-0.00106
2004	44.5	50.6	15	1.64836	-0.06630	0.00440	-0.00029
2005	45.2	50.2	16	1.655138	-0.05953	0.00354	-0.00021
2006	56.6	50	17	1.752816	0.03815	0.00146	0.00006
2007	58	50	18	1.763428	0.04876	0.00238	0.00012
2008	65	49.2	19	1.812913	0.09825	0.00965	0.00095
2009	70	48.2	20	1.845098	0.13043	0.01701	0.00222
2010	43.2	45.6	21	1.635484	-0.07918	0.00627	-0.00050
2011	102	45.2	22	2.0086	0.29394	0.08640	0.02540
2012	59.5	44.5	23	1.774517	0.05985	0.00358	0.00021
2013	50	43.2	24	1.69897	-0.01569	0.00025	0.00000
2014	63.5	41	25	1.802774	-0.01569	0.00025	0.00000
2015	50	37	26	1.69897	-0.01569	0.00025	0.00000
2016	37	36.8	27	1.568202	-0.14646	0.02145	-0.00314
2017	50.2	36.2	28	1.700704	-0.01396	0.00019	0.00000
2018	58	33.2	29	1.763428	0.04876	0.00238	0.00012
2019	28	25.3	30	1.447158	-0.26751	0.07156	-0.01914

Appendix Table 2. Annual average rainfall of four stations and their cumulative in mm

Year	Aleta-.Wendo	Tefer-Kella	Kebado	Aposto	Average of annual RF of 3 Stations	Cumm. Of average of 3stations	Cumm. Of Aleta-Wendo
2019	1330.5	1174.9	1351.5	1178.2	1234.9	1431.73	1274.2
2018	1354.6	1143.6	1027.7	991.6	1054.3	2858.79	3003.9
2017	1645.6	2045.3	1439.8	1159.7	1548.3	4208.94	5572.7
2016	1722.7	1056.6	1473.1	1109.2	1213.0	5506.24	7084.4
2015	1571.7	1132.4	1622.8	1146.9	1300.7	6473.86	8365.4
2014	1428.9	1537.6	1511.7	1058.9	1369.4	8036.23	10327.3
2013	1649.9	1490.8	1762.6	1471.4	1574.9	9546.49	12079.1
2012	2004.5	1186.0	1685.7	1356.0	1409.2	10968.81	13458.5
2011	1582.6	1240.5	1404.4	923.3	1189.4	12795.18	15450.8
2010	1233.4	1353.3	1331.4	675.0	1119.9	14648.71	17281.8
2009	1418.9	1912.7	1123.2	872.4	1302.8	15981.21	18638.4
2008	1956.0	2060.1	1576.2	928.8	1521.7	17294.68	20099.2
2007	1438.3	1536.0	1261.4	829.1	1208.8	19237.91	22178.3
2006	1496.0	1615.9	1150.7	849.2	1205.3	20966.81	23956.8
2005	1287.0	1062.5	1146.0	929.3	1045.9	22459.38	25712.3
2004	1755.5	2171.3	1275.0	1031.4	1492.6	23505.31	26999.3
2003	1778.5	2076.4	1603.8	1506.5	1728.9	24710.58	28495.3
2002	2079.1	2834.1	1627.2	1368.4	1943.2	25919.41	29933.6
2001	1460.8	1652.0	1299.0	989.4	1313.5	27441.11	31889.5
2000	1356.6	1787.5	1196.2	1013.8	1332.5	28743.88	33308.4
1999	1830.9	2587.5	1654.1	1319.0	1853.5	29863.78	34541.8
1998	1992.3	2726.0	1414.3	1338.8	1826.4	31053.18	36124.4
1997	1379.4	1841.4	1337.5	1088.1	1422.3	32462.41	38128.9
1996	1751.9	2049.9	1240.3	1240.6	1510.3	34037.34	39778.8
1995	1961.9	1966.4	1517.8	1203.0	1562.4	35406.74	41207.7
1994	1280.9	1064.4	893.0	945.5	967.6	36707.44	42779.4
1993	1511.7	2002.4	852.5	1037.0	1297.3	37920.41	44502.1
1992	2568.9	1384.5	1466.0	1199.9	1350.1	39468.66	46147.7
1991	1729.7	1625.7	1396.3	1259.2	1427.1	40522.95	47502.3
1990	1274.2	1258.1	1992.3	1044.8	1431.7	41757.81	48832.8

Appendix Table 3. Outlier test K_N value

Sample Size, n	K_N	Sample Size, n	K_N	Sample Size, n	K_N	Sample Size, n	K_N
10	2.036	24	2.467	38	2.661	60	2.837
11	2.088	25	2.486	39	2.671	65	2.866
12	2.134	26	2.502	40	2.682	70	2.893
13	2.175	27	2.519	41	2.692	75	2.917
14	2.213	28	2.534	42	2.700	80	2.940
15	2.247	29	2.549	43	2.710	85	2.961
16	2.279	30	2.563	44	2.719	90	2.981
17	2.309	31	2.577	45	2.727	95	3.000
18	2.335	32	2.591	46	2.736	100	3.017
19	2.361	33	2.604	47	2.744	110	3.049
20	2.385	34	2.616	48	2.753	120	3.078
21	2.408	35	2.628	49	2.760	130	3.104
22	2.429	36	2.639	50	2.768	140	3.129
23	2.448	37	2.650	55	2.804	141	3.131

Appendix Table 4. Kz=F (Cs, T) for Log-Pearson Type III Distribution

Skew coefficient (g)	*Recurrence interval (T-yr)*						
	2	*5*	*10*	*25*	*50*	*100*	*200*
	Per cent chance (annual probability of occurrence P(%))						
	50	*20*	*10*	*4*	*2*	*1*	*0.5*
3.5	–0.396	0.420	1.180	2.278	3.152	4.051	4.970
2.5	–0.360	0.518	1.250	2.162	3.048	3.845	4.652
2.0	–0.307	0.609	1.302	2.219	2.912	3.605	4.298
1.8	–0.282	0.643	1.318	2.193	2.848	3.499	4.147
1.6	–0.254	0.675	1.329	2.163	2.780	3.388	3.990
1.4	–0.225	0.705	1.337	2.128	2.706	3.271	3.828
1.2	–0.195	0.732	1.340	2.087	2.626	3.149	3.661
1.0	–0.164	0.758	1.340	2.043	2.542	3.022	3.489
0.9	–0.148	0.769	1.339	2.018	2.498	2.957	3.401
0.8	–0.132	0.780	1.336	1.993	2.453	2.891	3.312
0.7	–0.116	0.790	1.333	1.967	2.407	2.824	3.223
0.6	–0.099	0.800	1.328	1.939	2.359	2.755	3.132
0.5	–0.083	0.808	1.323	1.910	2.311	2.686	3.041
0.4	–0.066	0.816	1.317	1.880	2.261	2.615	2.949
0.3	–0.050	0.823	1.309	1.849	2.211	2.544	2.856
0.2	–0.033	0.830	1.301	1.818	2.159	2.472	2.763
0.1	–0.017	0.836	1.292	1.785	2.107	2.400	2.670
0	0.000	0.841	1.282	1.751	2.054	2.326	2.576
–0.1	0.017	0.846	1.270	1.716	2.000	2.252	2.482
–0.2	0.033	0.850	1.258	1.680	1.945	2.178	2.388

Appendix Table 5. Reduced mean, Yn in Gumbel's Extreme Value Distribution

N	0	1	2	3	4	5	6	7	8	9
10	0.4952	0.4996	0.5035	0.5070	0.5100	0.5128	0.51570	0.51810	0.52020	0.55200
20	0.5236	0.5252	0.5268	0.5283	0.5296	0.5309	0.53200	0.53320	0.53430	0.53530
30	0.5362	0.5371	0.538	0.5388	0.5396	0.5402	0.54100	0.54180	0.54240	0.54300
40	0.5436	0.5442	0.5448	0.5453	0.5458	0.5463	0.54680	0.54730	0.54770	0.54810
50	0.5485	0.5489	0.5493	0.5497	0.5501	0.5504	0.55080	0.55110	0.55150	0.55180
60	0.5521	0.5524	0.5527	0.5530	0.5533	0.5535	0.55380	0.55400	0.55430	0.55450
70	0.5548	0.5550	0.5552	0.5555	0.5557	0.5559	0.55610	0.55630	0.55650	0.55670
80	0.5569	0.5570	0.5572	0.5574	0.5576	0.5578	0.55800	0.55810	0.55830	0.55850
90	0.5586	0.5587	0.5589	0.5591	0.5592	0.5593	0.55950	0.55960	0.55980	0.55990
100	0.5600									

Appendix Table 6. Reduced Standard Deviation Snin Gumbel's Extreme Value Distribution

N	0	1	2	3	4	5	6	7	8	9
10	0.9496	0.9676	0.9833	0.9971	1.0095	1.0206	1.0316	1.0411	1.0493	1.0565
20	1.0628	1.0696	1.0754	1.0811	1.0864	1.0915	1.0961	1.1004	1.1047	1.1086
30	1.1124	1.1159	1.1193	1.1226	1.1255	1.1285	1.1313	1.1339	1.1363	1.1388
40	1.1413	1.1436	1.1458	1.1480	1.1499	1.1519	1.1538	1.1557	1.1574	1.1590
50	1.1607	1.1623	1.1638	1.1658	1.1667	1.1681	1.1696	1.1708	1.1721	1.1734
60	1.1747	1.1759	1.1770	1.1782	1.1793	1.1803	1.1814	1.1824	1.1834	1.1844
70	1.1854	1.1863	1.1873	1.1881	1.1890	1.1898	1.1906	1.1915	1.1923	1.1930
80	1.1938	1.1945	1.1953	1.1959	1.1967	1.1973	1.1980	1.1987	1.1994	1.2001
90	1.2007	1.2013	1.2020	1.2026	1.2032	1.2038	1.2044	1.2049	1.2055	1.2060
100	1.2065									

Appendix Table 7. Gumbel method for the study area

		Return Period	ln(T/T-1)	YT=-ln(lnT/T-1)	K=(YT-Yn/Sn)	KS	X_T
		2	0.693	0.367	-0.15	-2.255	51.44
Yn	0.5371	5	0.223	1.500	0.86	12.729	66.42
Sn	1.1159	10	0.105	2.250	1.54	22.650	76.34
Xmean	53.69	25	0.083	2.484	1.74	25.743	79.44
STDV.S	14.75	50	0.020	3.902	3.02	42.215	95.91
		100	0.010	4.600	3.64	50.064	103.76

Table 8: Comparison of log Pearson-III and Gumble distribution method

Return period(T)	log Pearson-III	Gumbel's extreme value
2	52.12	51.44
5	61.70	66.42
10	73.04	76.34
25	82.37	88.88
50	88.92	98.18
100	95.17	103.76

Appendix Table 9. Goodness of fit test by Easyfit software

#	Distribution	Kolmogorov Smirnov		Anderson Darling		Chi-Squared	
		Statistic	Rank	Statistic	Rank	Statistic	Rank
1	Beta	0.09794	15	0.29819	17	0.64058	20
2	Burr	0.09674	8	0.26485	1	0.23174	3
3	Burr (4P)	0.49396	56	14.227	57	N/A	
4	Cauchy	0.12072	33	0.68838	41	3.6757	42
5	Chi-Squared	0.13717	39	1.1606	43	1.3149	31
6	Chi-Squared (2P)	0.09773	14	0.31031	20	0.75008	25
7	Dagum	0.09762	13	0.27223	2	0.22148	2
8	Dagum (4P)	0.88603	58	77.516	58	521.91	55
9	Erlang	0.10458	23	0.36923	28	0.93033	27
10	Erlang (3P)	0.121	34	0.45237	32	0.93961	28
11	Error	0.15697	43	0.53772	37	2.8038	40
12	Error Function	0.97115	59	186.13	59	3581.2	56
13	Exponential	0.42782	54	7.7522	54	29.377	52
14	Exponential (2P)	0.24656	49	3.7428	48	8.8947	48
15	Fatigue Life	0.09683	9	0.30125	18	0.63349	15
16	Fatigue Life (3P)	0.09578	7	0.29053	9	0.64375	24
17	Frechet	0.16306	45	1.0362	42	0.43127	5
18	Frechet (3P)	0.10571	25	0.34498	24	0.58247	8
19	Gamma	0.10245	22	0.29591	15	2.1274	38
20	Gamma (3P)	0.09716	11	0.29481	14	0.64371	23
21	Gen. Extreme Value	0.09954	18	0.29351	11	2.0995	34
22	Gen. Gamma	0.0947	3	0.27822	3	0.30857	4
23	Gen. Gamma (4P)	0.09701	10	0.29443	12	0.64298	22

24	Gen. Pareto	0.12851	37	4.2959	49	N/A	
25	Gumbel Max	0.11323	30	0.37512	30	0.59537	9
26	Gumbel Min	0.18445	46	1.7152	45	1.477	32
27	Hypersecant	0.13633	38	0.37158	29	3.0179	41
28	Inv. Gaussian	0.09977	19	0.30283	19	0.59654	10
29	Inv. Gaussian (3P)	0.12481	35	0.47339	35	2.0973	33
30	Johnson SU	0.10564	24	0.29457	13	0.62907	13
31	Kumaraswamy	0.11004	29	0.36428	26	2.1165	37
32	Laplace	0.16164	44	0.59021	39	4.6675	45
33	Levy	0.56881	57	11.261	56	63.744	54
34	Levy (2P)	0.42543	53	5.519	52	27.264	51
35	Log-Gamma	0.10191	21	0.3273	21	0.60196	11
36	Log-Logistic	0.13779	40	0.45477	33	0.82721	26
37	Log-Logistic (3P)	0.10171	20	0.28255	4	0.19444	1
38	Log-Pearson 3	0.09916	16	0.29132	10	0.63806	16
39	Logistic	0.12728	36	0.33191	22	2.5028	39
40	Lognormal	0.09719	12	0.29763	16	0.63094	14
41	Lognormal (3P)	0.09466	2	0.28811	7	0.64126	21
42	Nakagami	0.11712	32	0.48517	36	0.46531	6
43	Normal	0.11637	31	0.40978	31	3.8935	43
44	Pareto	0.3054	51	6.3089	53	16.388	50
45	Pareto 2	0.43065	55	7.8376	55	29.645	53
46	Pearson 5	0.10995	28	0.36654	27	0.58103	7
47	Pearson 5 (3P)	0.09555	5	0.28594	5	0.64018	18
48	Pearson 6	0.09536	4	0.28847	8	0.63905	17

49	Pearson 6 (4P)	0.09555	6	0.28599	6	0.64032	19
50	Pert	0.14015	41	0.57851	38	0.62614	12
51	Power Function	0.3315	52	5.1521	51	4.9924	46
52	Rayleigh	0.23356	48	3.0562	47	11.548	49
53	Rayleigh (2P)	0.10618	27	0.35254	25	2.1073	35
54	Reciprocal	0.25788	50	2.7473	46	7.6667	47
55	Rice	0.08458	1	0.46753	34	1.3055	30
56	Student's t	0.99936	60	218.87	60	67710.0	57
57	Triangular	0.22098	47	1.4139	44	4.1021	44
58	Uniform	0.14759	42	4.6156	50	N/A	
59	Weibull	0.09945	17	0.61432	40	1.1868	29
60	Weibull (3P)	0.10574	26	0.34386	23	2.1082	36
61	Johnson SB	No fit					

Appendix Table 10. Percentage of imperviousness for particular landuse

Land Use	Percent of Imperviousness
Industrial	76
Commercial	56
Institutional	34
Low density residential	19
Medium density residential	38
Paved surface with gravel joint	70
Open urban land	11
Roofs	90
Concrete and asphalt surface	80
Gravel path with undeveloped part of soil	20
Gravel road sharply slope mountain road	40
High density residential	51

Source: NRSC, 2010

Appendix Table 11. List of main Meteorology Stations in rainfall region

Meteorological Region	Station	Years of Record	Meteorological Region	Station	Years of Record
A1	Axum	17	B	Bedele	39
	Mekele	46		Gore	56
	Maychew	32		Nekempte	40
A2	Gondar	52		Jima	54
	Debre Tabor	15		Arba Minch	23
	Bahir Dar	45		Sodo	49
	Debre Markos	55		Awasa	36
	Fitche	44	C	Kombolcha	57
	Addis Ababa	57		Woldiya	29
	Debre Zeit	55		Sirinka	27
A3	Nazareth	46	D1	Gode	33*
	Kulumsa	43		Kebri Dihar	40
	Robe/Bale	29	D2	Kibre Mengist	33
A4	Metehara	24		Negele	51
	Dire Dawa	58		Moyale	29
	Mieso	42		Yabelo	34

Appendix Table 12. 24hr rainfall depth vs frequency

Return Period Years	24 hr Rainfall Depth (mm) vs Frequency (yr)							
	2	5	10	25	50	100	200	500
RR-A1	50.30	66.02	76.28	89.13	98.63	108.06	117.48	130.00
RR-A2	51.92	65.52	74.45	85.70	94.07	102.45	110.91	122.27
RR-A3	47.54	59.61	67.66	77.92	85.62	93.34	101.13	111.58
RR-A4	50.39	63.83	72.28	82.55	89.97	97.20	104.32	113.63
RR-B1	58.87	71.26	79.29	89.35	96.84	104.37	112.02	122.41
RR-B2	55.26	69.95	79.68	92.03	101.29	110.61	120.07	132.87
RR-C	56.52	71.04	80.54	92.52	101.48	110.50	119.66	132.06
RR-D	56.23	76.84	90.37	107.46	120.23	133.05	146.00	163.44

Source: ERA Manual, 2013

Appendix Table 13: IDF Curve developed for Aleta-Chuko Town

Duration(min)	Return Periods					
	T-2	T-5	T-10	T-25	T-50	T-100
	I(mm/hr)	I(mm/hr)	I(mm/hr)	I(mm/hr)	I(mm/hr)	I(mm/hr)
5	98.8	116.9	138.4	156.1	168.5	180.4
10	82.4	97.6	115.5	130.3	140.6	150.5
15	70.9	83.9	99.3	112.0	120.9	129.4
20	62.2	73.7	87.2	98.4	106.2	113.6
25	55.5	65.8	77.8	87.8	94.8	101.4
30	50.2	59.4	70.4	79.3	85.6	91.7
35	45.8	54.3	64.2	72.4	78.2	83.7
40	42.2	49.9	59.1	66.7	72.0	77.0
45	39.1	46.3	54.8	61.8	66.7	71.4
50	36.4	43.1	51.1	57.6	62.2	66.5
55	34.1	40.4	47.8	53.9	58.2	62.3
60	32.1	38.0	45.0	50.8	54.8	58.6
65	30.3	35.9	42.5	47.9	51.7	55.4
70	28.7	34.0	40.3	45.4	49.0	52.5
75	27.3	32.3	38.3	43.2	46.6	49.9
80	26.0	30.8	36.5	41.1	44.4	47.5
85	24.9	29.4	34.9	39.3	42.4	45.4
90	23.8	28.2	33.4	37.6	40.6	43.5
95	22.8	27.0	32.0	36.1	39.0	41.7
100	21.9	26.0	30.8	34.7	37.4	40.1
105	21.1	25.0	29.6	33.4	36.0	38.6
110	20.4	24.1	28.5	32.2	34.7	37.2
115	19.7	23.3	27.5	31.1	33.5	35.9
120	19.0	22.5	26.6	30.0	32.4	34.7
125	18.4	21.8	25.8	29.1	31.4	33.6
130	17.8	21.1	25.0	28.2	30.4	32.5
135	17.3	20.5	24.2	27.3	29.5	31.6

140	16.8	19.9	23.5	26.5	28.6	30.6
145	16.3	19.3	22.8	25.8	27.8	29.8
150	15.9	18.8	22.2	25.1	27.0	28.9
155	15.4	18.3	21.6	24.4	26.3	28.2
160	15.0	17.8	21.1	23.8	25.6	27.5
165	14.7	17.3	20.5	23.2	25.0	26.8
170	14.3	16.9	20.0	22.6	24.4	26.1
175	14.0	16.5	19.6	22.1	23.8	25.5
180	13.6	16.1	19.1	21.5	23.3	24.9

Appendix Table 14. Design storm frequency(yrs) by Geometric design criteria

Structure type	Geomtric Design Standard			
	DS1/DS2	DS3/DS4	DS5/DS6/DS7	DS8/DS9/DS10
Gutters and inlets	10 or 5	2	2	
Side ditches	10	**10**	5	5
Ford/low water bridge				5
Culvert, pipe(span<2m)	25	10	5	5
Culvert(2m<span<6m)	50	25	10	10
Short span bridges (6m<span<15m)	50	50	25	25
Medium span bridges (15m<span<50m)	100	50	50	50
Long span bridges (span>50m)	100	100	100	100
Check/review flood	200	100	100	100

Source: (ERA DDM, 2011)

Appendix Table 15. Manning’s n for open channels

Channel types	Manning’s n	Channel types	Manning’s n
Lined channels		Excavated or dredged	
Asphalt	0.011-0.017	Earth, Straight & uniform	0.02-0.03
Brick	0.012-0.018	Earth, winding, fairly uniform	0.025-0.04
Concrete	0.011-0.020	Rock	0.03-0.045
Rubble or Riprap	0.020-0.035	Unmaintained	0.05-0.14
Vegetal	0.030-0.40		

Source: ASCE(1982). Gravity Sanitary sewer design and construction ,ASCE Manual of Practice No.60, New York, NY

Appendix Table 16. Runoff Coefficient

Description of Area	Runoff Coefficient (C)
Business:	
Commercial areas	0.70-0.95
Neighborhood areas	0.50-0.70
Residential:	
Single-family areas	0.30-0.50
Multi-units, detached	0.40-0.60
Multi-units, attached	0.60-0.75
Suburban	0.25-0.40
Industrial:	
Light areas	0.50-0.80
Heavy areas	0.60-0.90
Parks, Cemeteries	0.10-0.25
Playgrounds	0.20-0.40
Railroad yard areas	0.20-0.40
Unimproved areas	0.10-0.30
Streets:	
Asphaltic	0.70-0.95
Concrete	0.80-0.95

Brick	0.70-0.85
Drives and Walks	0.75-0.85
Roofs	0.75-0.95

Appendix Table 17. GTZ Standards to assess performance of the drainage canal

Classifiction Indicators	Surface condition
Very good	Shapes of USWD lines as still in original design condition
Good	No siginificant depressions, undulations, and deformation
Light	The shape of the USWD lines deteriorate, but still sheds water
Severe	The total collapse of the USWD lines structure and barely passable

Appendix Table 18. Hydrological Soil Group

Group	Descriptions	Rate of transmission(mm/hr)
A	Sand, loamy sand or sandy loam. Soil has low runoff potential due to high infiltration rates. These soils primarly consist of deep, well-drainned sands and gravels.	>7.62
B	Silt loam, or loam. Soil has a moderately low runoff potential due to moderate infiltration rates, low bulk density, or contain greater than 35 percent rock fragments	3.81-7.62
C	Sandy clay loam. Soils have a moderately high runoff potential due to slow infiltration rates. Soils typically have between 20% and 40% clay and less than 50% sand and have loam, silt loam, sandy clay loam, clay loam, and silt clay loam textures.	1.27-3.81
D	Clay loam, silt clay loam, sandy clay, silt clay or clay. Soils have a high runoff potential due to very slow infiltration rates. These soils have a swelling properties and very low rate of water transmission.	0-1.27

Appendix Table 19. Runoff coefficient, percentage of imperviousness and time of concentration

S-C No	Landuse	Area (Ha)	R. Coff	% imp	Av. Cwe	Av. %imp	H1	H2	L	S	Tc=(t1+t2+t...+)	Tc recommended by ERA
S-C-01	Commercial	2.67	0.75	69	0.64	46.5	1884	1874	475	0.021	4	15
	Gravel road	0.6	0.6	35								
	Mixed	3.73	0.7	47								
	Green area	1.5	0.3	35								
	Total	8.5										
S-C-02	Cobblstone	0.55	0.75	68	0.75	68.5	1874	1870.7	520	0.006	4	15
	Commercial	5.45	0.75	69								
	Total	6										
S-C-03	High School	13.4	0.6	57	0.63	45.50	1873	1870	450	0.007	18	18
	Muncipality	1.2	0.7	40								
	Kale-Hwoyt Church	0.8	0.6	40								
	Mixed Area	5.6	0.7	45								
	Total	21										
S-C-04	Gravel road	1.74	0.55	37	0.66	45.00	1870.7	1862.7	455	0.014	12	15
	Catholic Church	1.93	0.6	40								
	Residential	6.93	0.65	38								
	Commercial	3.9	0.75	65								
	Total	14.5										
S-C-05	Gudumale	1	0.3	30	0.4	42	1884	1871.7	907.5	0.014	8	15
	Gravel road	3	0.4	38								
	Finance burua	0.9	0.4	40								
	Commercial	2.6	0.6	65								
	Residential	11	0.5	37								
	Total	18.5	0.4									
S-C-06	Medihanalem Church	4.2	0.6	40	0.64	47.67	1871.7	1871.1	115	0.01	11	15
	Gravel road	0.5	0.55	38								
	Commercial	1.8	0.75	65								
	Total	6.5										
S-C-07	Hiwt-brhan Church	1.3	0.55	45	0.57	51.00	1871.1	1870.5	215	0.003	12	15
	Hospital	1.7	0.6	50								
	Cobblstone	0.15	0.75	65								
	Residential	3.05	0.6	55								
	Gravel road	1.1	0.5	40								
	Total	7.3		00								

S-C-08	Wereda adiministrati on	0.35	0.6	40	0.50	41.33	1875 .5	1870	600	0.009	6	15
	Cobblstone	0.15	0.7	65								
	Lemela K/hweyt Church	0.4	0.6	40								
	Gravel road	0.55	0.5	38								
	Residential	4	0.6	35								
	Green area	1.7	0.3	30								
	Total	7.15										
S-C-09	Gravel road	0.55	0.5	38	0.64	51.5	1874	1871	393	0.008	6	15
	Commercial	2.55	0.67	65								
	Total	3.1										
S-C-10	Commercial	1.4	0.75	65	0.75	65	1872	1871	220	0.005	22	22
S-C-11	Commercial	2.5	0.75	65	0.75	66.5	1871 .2	1870.5	280	0.003	20	20
	Cobblstone	0.1	0.75	68								
		2.6										
S-C-12	Cobblestone	0.1	0.75	68	0.65	47	1870 .5	1870	367	0.001	29	29
	Gravel road	0.1	0.55	38								
	Residential	2.3	0.65	35								
	Total	2.5										
S-C-13	Mulu wengel church	1.55	0.55	40	0.55	37.67	1869	1867	580	0.003	21	21
	Residential	7.5	0.6	35								
	Gravel road	1.95	0.5	38								
	Total	11										
S-C-14	Residential	9.5	0.6	60	0.58	60.00	1869	1868	438	0.002	30	30
	Gravel road	2.2	0.5	60								
	Total	11.7										
S-C-15	Commercial	2	0.75	65	0.75	65	1868 .4	1868	98	0.004	13	15
S-C-16	Residential	2.95	0.65	36	0.65	36.00	1867 .4	1866.9	163	0.003	19	19
	Gravel road	0.15	0.55	36								
	Total	3.1										
S-C-17	Green area	3.1	0.3	30	0.49	34.33	1866 .9	1861.9	343	0.01	21	21
	Gravel road	1.4	0.55	38								
	Residential	3.3	0.65	35								
	Total	7.8										

S-C-18	Cobblestone	0.15	0.75	65	0.75	65	1868	1867.4	98	0.01	6	15
	Commercial	2.05	0.75	65								
	Total	2.2										
S-C-19	Green area	0.6	0.3	30	0.58	34.67	1867	1866	135	0.01	15	15
	Gravel road	0.6	0.55	38								
	Residential	2.8	0.65	36								
	Total	4										
S-C-20	Green area	2.3	0.3	30	0.53	43.50	1866	1860	384	0.02	24	24
	Gravel road	1.7	0.5	36								
	Commercial	2	0.7	69								
	Residential	4	0.6	39								
	Total	10										
S-C-21	Asphalt	0.6	0.7	80	0.55	51.60	1870.8	1869.8	215	0.005	26	26
	Gravel road	1	0.5	38								
	M/eyesus church	1	0.5	40								
	Residential	2	0.55	35								
	Commercial	3	0.6	65								
	Total	7.6	2.78									
S-C-22	School	0.3	0.5	45	0.61	48.00	1869.8	1867.8	156	0.01	13	15
	Gravel road	0.5	0.5	38								
	Mixed Area	3	0.6	40								
	Commercial	2.7	0.65	69								
	Total	6.5										
S-C-23	Asphalt	0.15	0.8	80								
	Gravel road	0.5	0.55	38	0.72	56.75	1871	1870	226	0.004	17	17
	Commercial	2.35	0.75	69								
	Mixed Area	0.8	0.7	40								
	Total	3.8										
S-C-24	Gravel road	0.85	0.55	38	0.71	52.33	1870	1869	148	0.01	20	20
	Menaharya	0.55	0.7	50								
	Commercial	3.7	0.75	69								
	Total	5.1										
S-C-25	gravel road	0.4	0.55	38	0.64	38.00	1868.8	1866.8	214	0.01	16	16
	residential	2.4	0.65	38								
	Total	2.8										
S-C-26	Asphalt	0.2	0.8	80	0.61	56.50	1871	1870	190	0.01	22	22
	gravel road	0.9	0.5	38								
	Commercial	2	0.65	68								
	Mixed Area	1.5	0.6	40								
	Total	4.6										

S-C-27	Asphalt	0.15	0.8	80	0.62	56.25	1874	1872	190	0.01	9	15
	Gravel road	0.6	0.5	38								
	Commercial	2	0.65	69								
	Residential	0.7	0.6	38								
		3.45										
S-C-28	Gravel road	0.5	0.5	38	0.5	38	1874	1873	200	0.01	14	15
	Residential	3.5	0.6	38								
		4										
S-C-29	Genet church	1.2	0.6	40	0.69	48.67	1873	1872	150	0.002	7	15
	Gravel road	0.3	0.55	38								
	Commercial	2.2	0.75	68								
		3.7										
S-C-30	Gravel road	2.2	0.55	38	0.65	38.67	1878	1873	615	0.01	16	16
	Residential	5.3	0.65	38								
	Mixed Area	4	0.7	40								
		11.5										
S-C-31	Residential	5.7	0.65	38	0.62	38.00	1876	1873	605	0.01	16	16
	Gravel road	2.8	0.55	38								
		8.5										
S-C-32	Commercial	3.1	0.75	68	0.72	62.00	1873	1872	180	0.01	11	15
	Asphalt	0.3	0.9	80								
	Gravel road	0.9	0.55	38								
		4.3										

Appendix Table 20. Existing Drainage lines capacity of study area by using Manning’s equation

Sub-Catch No and Code	Drainage Type	slope	N	d(m)	w(m)	A(m2)	P(m)	R	V(m/s)	Q(m3/s)	
S-C-01- DL 01	Masonory	0.021	0.013	0.70	0.5	0.35	1.90	0.184	3.61	1.26	
S-C-02- DL 02	Masonory	0.01	0.013	0.60	0.5	0.3	1.70	0.176	1.91	0.57	
S-C-03- DL 03	Masonory	0.01	0.013	0.80	0.6	0.6	2.20	0.273	2.60	1.56	
S-C-04- DL 04	Concrete	0.01	0.011	1.50	1	1.5	4.00	0.375	4.93	3.71	
S-C-05- DL 05	Masonory	0.02	0.013	0.60	0.5	0.3	1.70	0.176	3.45	1.0	
S-C-05- DL 06	Masonory	0.005	0.012	0.60	0.5	0.3	1.70	0.176	1.78	0.5	1.5
S-C-06- DL 07	Masonory	0.01	0.013	0.60	0.6	0.36	1.80	0.2	2.45	0.88	
S-C-07- DL 08	Masonory	0.005	0.013	0.60	0.7	0.42	1.90	0.221	1.92	0.81	
S-C-08- DL 09	Masonory	0.002	0.013	1.00	0.6	0.6	2.60	0.231	1.36	0.8	
S-C-09- DL 10	Masonory	0.01	0.013	0.60	0.6	0.36	1.80	0.2	2.22	0.8	
S-C-10- DL 11	Masonory	0.01	0.013	0.60	0.6	0.36	1.80	0.2	2.31	0.83	
S-C-11- DL 12	Masonory	0.003	0.013	0.50	0.5	0.25	1.50	0.167	1.16	0.36	
S-C-12- DL 13	Masonory	0.001	0.013	0.80	0.6	0.48	2.20	0.218	1.03	0.49	
S-C-13- DL 14	Masonory	0.007	0.013	0.70	0.6	0.42	2.00	0.21	2.22	0.93	
S-C-14- DL 15	Masonory	0.003	0.013	0.70	0.9	0.63	2.30	0.274	1.87	1.18	
S-C-15- DL 16	Masonory	0.010	0.013	0.6	0.5	0.3	1.7	0.176	2.44	0.73	
S-C-16- DL 17	Masonory	0.006	0.013	0.90	0.6	0.54	2.40	0.225	2.23	1.20	
S-C-17- DL 18	Masonory	0.01	0.013	0.90	0.6	0.54	2.40	0.225	3.07	1.66	
S-C-18- DL 19	Masonory	0.01	0.013	0.60	0.5	0.3	1.70	0.176	2.44	0.73	
S-C-19- DL 20	Masonory	0.01	0.013	0.90	0.6	0.54	2.40	0.225	2.45	1.32	
S-C-20- DL 21	Masonory	0.02	0.013	0.90	0.6	0.54	2.40	0.225	3.56	1.92	
S-C-21- DL 22	Masonory	0.005	0.013	0.90	0.6	0.54	2.40	0.225	1.94	1.05	
S-C-22- DL 23	Masonory	0.01	0.013	0.90	0.6	0.54	2.40	0.225	3.22	1.74	
S-C-23- DL 24	Masonory	0.004	0.013	0.90	0.6	0.54	2.40	0.225	1.89	1.02	
S-C-24- DL 25	Masonory	0.01	0.013	0.90	0.6	0.54	2.40	0.225	2.34	1.26	
S-C-25- DL 26	Masonory	0.00	0.013	0.90	0.6	0.54	2.40	0.225	1.95	1.05	
S-C-26- DL 27	Concrete	0.01	0.011	1.30	1	1.17	3.50	0.334	2.69	3.14	
S-C-27- DL 28	Masonory	0.02	0.013	0.70	0.6	0.42	2.00	0.21	3.41	1.43	
S-C-29- DL 29	Concrete	0.02	0.011	2.0	1.0	0.3	1.70	0.176	3.77	1.13	
S-C-32- DL 30	Concrete	0.01	0.011	2.0	1.0	0.3	1.70	0.176	1.80	0.54	

NB: DL = drainage line

Appendix Table 21. Inputs of Junction's

Junctions	Elevation	Maximum Depth(m)	Inverted Depth
J1	1885	0.7	1884.3
J2	1875	0.8	1874.2
J3	1872	1.2	1870.8
J4	1872	1.5	1870.5
J5	1867	1.5	1865.5
J6	1875	0.6	1874.4
J7	1873	0.6	1872.4
J8	1872	0.6	1871.4
J9	1872	1.0	1871.0
J10	1871	1.0	1870.0
J11	1870	0.7	1869.3
J12	1868	0.7	1867.3
J13	1869	0.6	1868.4
J14	1868	0.6	1867.4
J15	1867	0.9	1866.1
J16	1863	0.9	1862.1
J17	1872	0.6	1871.4
J18	1872	0.9	1871.1
J19	1871	0.9	1870.1
J20	1870	0.9	1869.1
J21	1869	0.9	1868.1
J22	1873	1.3	1871.7
J23	1872	1.3	1870.7
J24	1872	1.5	1870.5
J25	1875	0.7	1874.3
J26	1871	2.0	1869.0
J27	1870	2.0	1868.0
J28	1873	1.0	1872.0

Appendix Table 22. Inputs and characters of Outfalls

Outlets	Elevation	Maximum Depth(m)	Inverted Depth	X	Y	
Out-1	1851	1.5	1849.5	728942.90	427315.95	
Out-2	1872	1.5	1870.5	727809.35	425890.09	
Out-3	1866	0.6	1865.4	727828.40	425868.24	
Out-4	1857	0.9	1856.1	727978.70	427015.49	
Out-5	1860	0.9	1859.1	727596.29	426589.33	
Out-6	1870	2	1868	727850.32	425901.02	

Appendix Table 23. Inputs of Conduits

Conduit's	Conduits type and shape	Depth(m)	Width(m)	Length(m)	Roughness Coefficient
C-1	Masonory/Open/Rectangular	0.7	0.5	475	0.013
C-2	Masonory/Open/Rectangular	0.8	0.6	450	0.013
C-3	Concrete/Clossed/Rectangular	1.2	1.0	200	0.011
C-4	Concrete/Clossed/Rectangular	1.5	1.0	455	0.011
C-5	Concrete/Clossed/Rectangular	1.5	1.0	500	0.011
C-6	Masonory/Open/Rectangular	0.5	0.6	434	0.013
C-7	Masonory/Open/Rectangular	0.6	0.6	115	0.013
C-8	Masonory/Open/Rectangular	0.6	0.6	215	0.013
C-9	Masonory/Open/Rectangular	1.0	0.6	315	0.013
C-10	Masonory/Open/Rectangular	1.3	0.6	300	0.013
C-11	Masonory/Open/Rectangular	0.7	0.6	367	0.013
C-12	Masonory/Open/Rectangular	0.6	0.6	500	0.013
C-13	Masonory/Open/Rectangular	0.6	0.5	110	0.013
C-14	Masonory/Open/Rectangular	0.9	0.8	160	0.013
C-15	Masonory/Open/Rectangular	0.9	0.8	340	0.013
C-16	Masonory/Open/Rectangular	0.9	0.8	500	0.013
C-17	Masonory/Open/Rectangular	0.6	0.8	103	0.013
C-18	Masonory/Open/Rectangular	0.9	0.8	240	0.013
C-19	Masonory/Open/Rectangular	0.9	0.8	140	0.013
C-20	Masonory/Open/Rectangular	0.9	0.8	214	0.013
C-21	Masonory/Open/Rectangular	0.9	0.8	500	0.013
C-22	Concrete/Clossed/Rectangular	1.3	1.0	350	0.011
C-23	Concrete/Clossed/Rectangular	1.3	1.0	150	0.011
C-24	Masonory/Open/Rectangular	0.7	0.6	190	0.013
C-25	Concrete/Clossed/Rectangular	1.5	1	160	0.011
C-26	Concrete/Clossed/Rectangular	2	1	400	0.011
C-27	Concrete/Clossed/Rectangular	2	1	500	0.011
C-28	Concrete/Clossed/Rectangular	1	1	400	0.011

Appendix Table 24. Flooded Conduits

Conduit	Hours Both Ends Full	Hours Upstream Full	Hours Dnstream Full	Hours Above Normal Flow	Hours Capacity Limited
C-2	0.67	0.67	0.67	0.01	0.01
C-3	0.01	0.01	0.01	1.85	0.01
C-7	2.99	2.99	2.99	0.01	0.01
C-8	4.34	4.34	4.35	0.02	0.01
C-9	0.01	0.01	0.01	0.01	0.01
C-16	0.01	0.01	0.01	0.02	0.01
C-23	1.89	1.89	1.89	0.01	0.01
C-24	1.89	1.89	1.89	0.02	0.01
C-25	1.89	1.89	1.89	0.01	0.01

Appendix Table 25a: sub-catchment Runoff values after LID applied

Subcatchment Runoff   Click a column header to sort the column.

Subcatchment	Total Precip mm	Total Runon mm	Total Evap mm	Total Infil mm	Total Runoff mm	Total Runoff 10^6 ltr	Peak Runoff CMS	Runoff Coeff
S-C-01	131.41	0.00	0.00	0.93	124.56	9.40	1.27	0.948
S-C-02	131.41	0.00	0.00	2.76	125.55	6.28	0.97	0.955
S-C-03	131.41	0.00	0.00	3.60	119.89	19.18	2.48	0.912
S-C-04	131.41	0.00	0.00	5.68	115.15	13.24	1.51	0.876
S-C-05	131.41	0.00	0.00	5.47	112.97	14.12	1.63	0.860
S-C-06	131.41	0.00	0.00	6.13	107.12	4.82	0.81	0.815
S-C-07	131.41	0.00	0.00	3.52	122.18	6.48	0.87	0.930
S-C-08	131.41	0.00	0.00	4.70	121.01	6.23	0.80	0.921
S-C-09	131.41	0.00	0.00	3.28	121.23	3.76	0.50	0.923
S-C-10	131.41	0.00	0.00	3.21	124.26	1.74	0.26	0.946
S-C-11	131.41	0.00	0.00	3.69	117.43	2.94	0.35	0.894
S-C-12	131.41	0.00	0.00	5.70	112.72	2.82	0.32	0.858
S-C-13	131.41	0.00	0.00	4.12	119.18	10.73	1.38	0.907
S-C-14	131.41	0.00	0.00	3.69	123.31	9.25	1.32	0.938
S-C-15	131.41	0.00	0.00	2.58	128.69	2.57	0.43	0.979
S-C-16	131.41	0.00	0.00	3.72	123.30	3.82	0.54	0.938

Appendix Table 25b: sub-catchment Runoff values after LID applied

S-C-17	131.41	0.00	0.00	4.57	112.34	8.76	1.01	0.855
S-C-18	131.41	0.00	0.00	2.91	126.30	2.78	0.45	0.961
S-C-19	131.41	0.00	0.00	2.94	124.72	4.99	0.74	0.949
S-C-20	131.41	0.00	0.00	3.61	116.16	9.06	1.06	0.884
S-C-21	131.41	0.00	0.00	3.40	118.03	8.97	0.88	0.898
S-C-22	131.41	0.00	0.00	4.10	119.63	7.78	0.99	0.910
S-C-23	131.41	0.00	0.00	4.50	117.72	4.47	0.56	0.896
S-C-24	131.41	0.00	0.00	4.98	116.33	5.93	0.72	0.885
S-C-25	131.41	0.00	0.00	6.51	111.87	3.13	0.32	0.851
S-C-26	131.41	0.00	0.00	4.44	118.71	5.46	0.70	0.903
S-C-27	131.41	0.00	0.00	4.26	121.14	4.18	0.48	0.922
S-C-28	131.41	0.00	0.00	4.76	116.30	4.65	0.57	0.885
S-C-29	131.41	0.00	0.00	4.95	119.69	4.43	0.55	0.911
S-C-30	131.41	0.00	0.00	5.28	107.71	9.69	1.08	0.820
S-C-31	131.41	0.00	0.00	4.09	122.92	7.99	1.11	0.935
S-C-32	131.41	0.00	0.00	3.01	127.52	5.48	0.83	0.970