HAWASSA UNIVERSITY
INSTITUTE OF TECHNOLOGY
FACULTY OF BIOSYSTEM AND WATER RESOURCE ENGINEERING
DEPARTIMENT OF WATER RESOURSE AND IRRIGATION ENGINEERING

PERFORMANCE ASSESSMENT OF URBAN DRAINAGE SYSTEM USING STORM WATER MANAGEMENT MODEL (SWMM): THE CASE OF ALETA WONDO TOWN, SIDAMA REGIONAL STATE

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The thesis entitled as'' Performance Assessment of Urban Drainage System Using Storm Water Management Model (SWMM): The Case of Aleta Wondo Town, Sidama Regional State" has been approved by the school of Bio system and Environmental Engineering for partial fulfillment of the Degree of science in Water Resource Engineering and Management, and has carried out by Samuel Yonkora Id.No.WREMR/0013/12 under our supervision. Therefore, we recommend that the student has fulfilled the requirement and hence hereby can submit the thesis to the department

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EXAMINERS' APPROVAL SHEET

As members of the Examining board of the final open defence ,we certify that we have read and evaluate the thesis prepared by Samuel Yonkora entitled as "Performance Assessment of Urban Drainage System Using Storm Water Management Model (SWMM): The Case of Aleta Wondo Town, Sidama Regional State" and we recommend that it can be accepted as fulfilling the thesis requirement for the degree of science in water resources and Irrigation Engineering(specialization: Water Resource Engineering and Management)

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ABBREVIATION AND ACRONYMS

CN	Curve Number
DEM	Digital Elevation Model
ERA	Ethiopia Road Authority
EPA	Environmental Protection Agency
GOF	Goodness Of Fit
GPS	Geographical Position System
IDF	Intensity Duration Frequency
SCS	Soil Conservation Service
SWMM	Storm Water Management Model
CSA	Central Statistical Agency
CBE	Commercial Bank of Ethiopia
ULG	Urban Local Government
BMPS	Best Management Practice
CAD	Computer Aided Design
CW	Weighted Runoff Coefficient
DDM	Drainage Design Manual
GIS	Geographical Information System
HA	Hectares
HEC-RAS	Hydrologic Engineering Center-River Analysis system
KM^2	Square Kilometer
SC	Sub Catchment
SCS	Soil conservation service
SUDS	Sustainable urban drainage
Tc	Time of concentration

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

ABSTRACT

The drainage system were one of the most important factors to be considered in the urban drainage infrastructure plan. Now in adequate urban storm water drainage problems represent one of the most common source of compliant numerous town of Ethiopia. Drainage problem in Aleta wondo Town is the worest issue of over flooding on the surface runoff at highly rainfall season. Unfortunately, street flooding and overtopping drainage system problem are occurring at the rainy season in a Town.The main objective of this study was to assess the hydraulic performance of drainage system Aleta Wondo Town. The study employed both primary and secondary data collection from the field surveying work was conducted to inspect the existing drainage condition and to measure channel geometric conditions. The meteorological approaches employed in this study include, the identification of the existing drainage network flow direction and delineation of contributing area. The Storm Water Management Model(SWMM5.1) was applied to simulate the water level in the links and junctions. The simulated result indicate that 1.162m^3/s average flow rate,1.632m^3/s maximum flow rate and 12.457*10^3 m^3 total annual volume runoff. The hydrological analysis was computed by the the rational method. The result shows the existing drainage system of Aleta Wondo Town were not well connected drainage lines and silted by different waste material because of the capacity of drainage system can't handle the current runoff that flows over the area. Therefore, some adjustment or improvement to give the best service and needs a series of regular maintenance and drainage should be well designed and constructed to the standard and managed properly.

KeyWords: Aleta Wondo Town, Drainage system ,SWMM5.1, ,performance

1. INTRODUCTION

1.1. General Background

The urban storm water influences the service life of urban infrastructures and the rainfall intensity and characteristics of the catchment area are the major factors for designing urban storm water drainage facilities. These facilities have a paramount advantage to safely dispose of the generated floods to ultimate receiving systems. Storm water management is concerned with the collection, conveyance, storage, treatment, and disposal of storm runoff in a way that minimizes accelerated channel erosion increased flood damage. Proper drainage is an important factor that should be given due consideration in the design of a highway since inadequate drainage facilities can lead to premature deterioration of the highway and the development of adverse safety conditions (Mukherjee, 2014).

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, paved roads, etc.), drainage pattern changes, the overland flow gets faster flooding and environmental problems like land degradation increases. It is a crucial problem facing the existing and future road infrastructure (Belete, 2011).

Urban Drainage system is one of the most significant factors to be considered in the urban road design and other infrastructure. Drainage overflow is affected due to the expansion of urban infrastructure ,without concern of drainage characteristics and an increase in paved surfaces.such happenings effects can be minimized through modeling of flood events and preparing action plans.(Rangari V,2018)

Drainage difficulties in urban areas introduce flooding, decline of roads ,land degradation, sedimentation ,waterlogging and etc. with growth an urban increase in impervious surfaces the drainage pattern changes , the overflow gets faster , flooding and it becomes a critical problem facing the existing and future road infrastructure.(Drainage manual,2002)

Insufficient urban stormwater drainage complications represent one of the greatest common sources of compliant from the citizens in many towns of Ethiopia and this problem is getting worse and worse with the current high rate of urbanization in different parts of the country.

Storm water discharges are produced when the capacity of the land to retain precipitation is exceeded and run-off occurs. A run-off will be influenced by rainfall and intensity (millimeter of rainfall per hour)) and duration, antecedent storms and a number of watersheds, and land use characteristics such as slope, soil type, and impervious surfaces.(Biniyam , 2016)

Drainage design for highway facilities must strive to maintain compatibility and minimize interference with existing drainage patterns, and control flooding of the roadway surface for design flood events. Due to inadequate integration between the road and urban stormwater drainage infrastructure facilities, many areas are exposed to flooding problems and road damages in urban roads. (Belete., 2011).

The primary aim of an urban storm water management system is to ensure storm water generated from developed catchments causes minimal nuisance, danger and damage to people, property and the environment. This requires the adoption of a multiple objective approach, considering issues such as Ecosystem health, both aquatic and terrestrial; Flooding and drainage control; Public health and safety; Economic considerations; Recreational opportunities; Social considerations and Aesthetic values (Queensland Urban drainage manual, 2007).

Service life of infrastructure can be highly reduced by improper drainage system.it can be seen from rainy season which lasts from july to September where the highways are covered by the surface water .this water accelerates the deterioration rate of the raods and results in economic loss .the flooding of highways is the result of improper drainage system of the roads and poor integration of road and storm water drainage network (Ewnetu,2013)

In general road construction without adequate provision of drainage is a major cause of flooding. Lack of urban storm water drainage (USWD) management represents one of the most common sources of complaint from the residents in many urban centers of Ethiopia, and this problem gets worse and worse with the rate of urbanization. Currently Aleta Wondo is the one facing 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, a change in the run-off characteristics within the town, increased runoff and greater susceptibility to flooding.

1.2. Statement of the Problem

Road construction without adequate provision of drainage is a major cause of gully erosion. The road that has proper drainage systems which results in gully erosion may occur anywhere in the world. The problem is particularly severe in developing countries due to neglect in maintenance and lack of provision of safe outlets to the excess runoff. In Sidama region, Aleta Wondo town, road drainage structures are not properly functioning due to insufficient capacity of road ditches, unavailability of organized (networked) drainage structures at proper place, and illegal dumping of household solid and liquid wastes. Currently Aleta Wondo Town is facing the drainage problem, around Zera Bruk, Bekure Tsion, cheffe ,Mesalemiya Round, and Melawo River Outfall Around these area the existing drainage is busy and resulting over flow. A change in the run-off characteristics within the town, increased runoff and greater susceptibility to flooding.

The drainage systems through the Town are inadequate provision of conveying the runoff to the outlet points properly. Additionally the absence of appropriate maintenance of the existing drainage facilities (either partially or fully filled with silts or corrupted and causing the storm water to flow over the road surface) and insufficient capacity of road ditches. This inadequacy results in the blockage of the existing drainage channels and hence impairs their ability to convey the runoff properly. At some stations, the roadside ditch is loaded or silted by debris, silt, waste materials, and disposal of solid and liquid waste in the storm water 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 of the Study

1.3.1. General Objective

The general objective of the study was to evaluate the performance of Urban Drainage Systems of Aleta Wondo Town using the Storm water Management Model (SWMM).

1.3.2. Specific Objectives

The following are the specific objectives of the study:

1. To assess the current situation of the existing drainage system.

2. To evaluate the hydraulic capacity of the existing drainage structures of Aleta Wondo Town.

3. To estimate the water level at junctions and links in the drainage networks resulting from SWMM simulation

1.4. Research Questions

1. What is the current situation of the existing drainage system?

2. What is the hydraulic capacity of the existing drainage structures of the Aleta Wondo Town?

3. What is the water level at junctions and links in the drainage networks resulting from SWMM simulation ?

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 drainage schemes that contribute to better service coverage. To understand problems of damage and preserve the structures by avoiding further deterioration for 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 Storm water drainage system and proper functioning of drainage schemes in the town. In general Aleta Wondo town is the part of which facing the drainage problems, so further investigation is contributed the solution for storm water drainage problems and sustainable drainage systems for future use in the area. Also, the study contributes the following significances to Ethiopia Road Authority (ERA), Aleta Wondo town Administration, academicians, researchers, and other stakeholders who will conduct similar researches on other road drainage structures. preserving the road network and save the asset value, identifying the existing situation in order to know the real problems of the drainage Systems, finding possible solutions based on the recommendations to avoid the problems.

1.6. Scope of the Study

This study specifically focused on the Analysis of urban drainage systems and also for sustainability in drainage schemes that contribute to better service. Based on the available data, the Analysis of the existing drainage systems in Aleta Wondo town shall be done and possible mitigation measures are recommended

2. LITERATURE REVIEW

2.1. Historical Perspectives of Urban Storm Drainage System

Historically, urban drainage systems have been viewed with various perspectives. During different time periods and in different locations, it has been considered a vital natural resource, a convenient cleansing mechanism, an efficient waste transport medium, 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).

2.2. Current Urban Storm Drainage Systems

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. Engineers to improve urban drainage systems as a convenient waste disposal system, an important flood control system, the cause or transmitter of disease, a vital system for the protection of public health, an underground refuge for criminals and undesirables, and a source of civic pride. The purposes of urban drainage systems, including flood control, waste transport, water collection and recycling, often evolved through trial-and-error modifications after the systems were initially constructed. Changes in perspective of urban drainage in one city were most often caused by disease outbreaks, scientific discoveries, or technical advances in planning, design, and construction. Currently, well-planned urban drainage is viewed as a vital component of a sustainable urban system. (Burain and Edwards, 2002)

2.3. Urban Storm Drainage System Problems

The problem of urban drainage systems is that as long as road structures and subgrade soil do not have excess water the road will work well. But increase water content reduces the bearing capacity of the soil, which will increase the rate of deterioration and shorten the lifetime of the road. In such cases, the road will need rehabilitation more often than a well-drained road structure. Mainly the problem was observed on poorly working structures, such as culverts and ditches (Saara and Saarenketo, 2006).

The problems created by inadequate drainage channels have attracted a varying dimension of environmental problem. However, the most prominent among such problems being that the absence of drainage channels easily translate into environmental deterioration (Jimoh, 2003). Generally, drainage problems in urban areas include flooding, deterioration of roads, land degradation, sedimentation, blockage of drainage facilities, waterlogging, etc.

2.4. Urban Storm Drainage Systems Practice in World

According to United Kingdom's Department for Environment Flood and Rural Affairs (DEFRA) research framework for implementing integrated urban drainage, it is defined as management of the risk arising from drainage and flooding in urban areas through a portfolio of approaches. The Worldwide number of different countries is 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 integration of drainage system with the environment ultimately lead to sustainability (Balmforth, et al., 2009).

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).

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: culverts and bridges convey flows under road cross-section; energy dissipators used to control the velocities of flows, especially at culvert outlets (Kalantari, 2011).

In addition noted that the main challenge in developing countries has to do with the lack of the adequate number of skilled personnel who are able to plan and implement urban drainage in a timely manner and the lack of funding needed to pay for the work (Armitage, 2010).

2.6. Urban Storm Water Drainage Practice in Ethiopia

In Ethiopia 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 is very essential (MoWUD,2008).

Belete (2011) depicts the major causes of flooding as the blockage of urban storm water drainage lines along with inadequate/poor integration between the road and urban storm water drainage infrastructure. In addition, 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 flooding and environmental problems such as land degradation increases. It is a crucial problem facing the existing and future road infrastructure.

As FUPCoB (2008), the drainage problems in urban areas include flooding, deterioration of roads, land degradation, sedimentation, blockage of drainage facilities, water logging, 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, flooding and environmental problems such as land degradation increases. It is a crucial problem facing the existing and future road infrastructure. In spite of these problems, drainage facilities in most urban centers of the country are nearly absent or at lower coverage. Planning and design rarely guide construction or provision of such facilities and management.

Urban storm water influences the service life of urban infrastructures and the rainfall intensity and characteristics of the catchment area are the major factors for designing urban stormwater drainage facilities. These facilities have a paramount advantage to safely dispose of the generated floods to ultimate receiving systems. According to Urban storm water drainage design Manual, storm water management is concerned with the collection, conveyance, storage, treatment and disposal of storm runoff in a way that minimize accelerated channel erosion, increased flood damage (Meraf, 2015).

Adequate drainage facility is an important factor that should be given paramount importance in the design of roads since it greatly determines the roads 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. Intensity Duration Frequency (IDF) Curve

Intensity duration frequency (IDF) curve is necessary to calculate the intensity of rainfall used for the design. Depending on the location of the study area with respect to the station where the IDF is produced.

The frequency of rainfall of various intensities and duration is used in the hydrologic design of structures that control storm runoff and floods, such as storm sewers, high way culverts. Rainfall frequency analysis typically provides rainfall accumulation values at a point for a specified exceedance probability and various durations. Rainfall frequency analysis is used extensively for the design of engineering works that control storm runoff. These include municipal storm sewer systems, high way and railway culverts, and agricultural drainage systems. Precipitation frequency analysis also plays an important role in a diverse range of nonstructural problems involving natural hazards associated with extreme rainfall events (Maidment, 1993).

2.8. Review of Methods and Computer Software

Several urban drainage computer models are used in different parts of the world like Australia Victoria and United State of America (USA) are ILSAX (O'Loughlin, 1993), RAFTS (WP Software, 1991), RatHGL (WP Software, 1992), CIVILCAD (Surveying and Engineering Software, 1997), RORB (Laurenson and Mein, 1990), SWMM (U.S. Environmental Protection Agency, 1992), WBNM (Boyd et al., 2000) and HEC-RAS (Hydrologic Engineering Centre, 2000), Rational Formula method (Aitken,1975) were used as urban drainage computer models for design and analysis.

The other 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), MOUSE (Danish Hydraulic Institute, 1988), DR3M (Alley and Smith, 1990) and HSPF (Johnson et al.,1984). The models AUSQUAL, STORM, DR3M and HSPF were specially developed for water quality simulation in urban drainage systems while MOUSE was developed for both hydraulic and water quality simulation. Even though there are dozens of software packages available for modeling of sustainable drainage techniques and devices used for both water quantity and quality simulations (Elliott *et al*., 2007 and Zoppou, 2001), some are commercial and some are open-source. Elliott and Trowsdale examined 10 modeling methods for SUDS with respect to water quantity and quality simulations, but most of the reviewed models contain functions on hydrological simulation in terms of rainfall generation and runoff routing and only a few are capable of simulating the drainage network hydraulics, such as SWMM and MOUSE (the old version of Mike Urban). Also, with the diverse models for SUDS, many of them are still claimed to be non-user friendly because of their technical complexity (Viavattene, 2008.92). Open-source models require a nominal cost; however, they provide very little technical support for users. In contrast, commercial models support the beginners well, but their costs are often too high for widespread use (Zoppou, 2001).

Additionally design of storm drainage system evaluated in different studies conducted in different areas in the case of performance assessment of road drainage systems for existing situation by using: HEC-RAS (Hydrologic Engineering Center-River Analysis System) model and Soil Conservation Service (SCS) method (Meraf, 2015), manning's, statistical analysis,

GIS, DEM and aerial photograph (Mulualem, 2016), storm CAD, Terra Model, GIS and Flow Master (Kirubel, 2014) and statistical analysis, geographic information system (GIS) (Belete, 2011) and ,Geographic information system (GIS), DEM and aerial photograph, Method ,HEC-RAS (Hydrologic Engineering Center-River Analysis System) model and Soil Conservation Service (SCS) method .

Based on the result of the study the drainage system is insufficient at a different area and most of the stations of the catchments were investigated that required construction of additional storm drainage system, lack of management/poor management and the existed drainage inadequate.

Generally Statistical rational method and storm water management model (SWMM 5.1) was selected due to its simplicity and availability.

Table 2. 1Comparison some features of the modeling tools

Some features of the modeling tools		SWMM5.0	PCSWMM	Storm CAD	Civil storm	XPSWMM
Developer/publisher		EPA	CHI	Bentley	Bentley	XPsolutions
Water system	Storm water	yes	yes	Yes	yes	Yes
	Waste water	yes	yes	No	No	Yes
	River system	No	yes	No	No	Yes
Area of use	Water quantity	yes	yes	Yes	Yes	Yes
	Water quality	yes	yes	No	Yes	Yes
	Sever system	yes	yes	No	No	No
	LID/SUDS/LOD/WSUD	yes	yes	No	Yes	Yes
	Long term predictions	Both	Both	No	No	Both
	Simulation of LI pipe flow	yes	yes	Yes	Yes	Yes
	DD overland flow	No	yes	No	No	Yes
Import/export/connections	GIS	Interchange	Integration	Some conversionutilities from GIS data base	Some conversionutilities from GIS data base	Import/Export
	CAD	No	Yes(varios format)	Can be used with in autocad	Can be used with in autocad	

3. MATERIAL AND METHODS

3.1. Description of the Study Area

3.1.1. Location

Aleta Wondo is a town in Sidama Regional State of Ethiopia. It is located at a distance of 333 kilometers south of the Federal capital town, Addis Ababa and 64 kilometers North East of Sidama Capital, Hawassa. It is classified under the settlement category of Grade 2 ULG. Aleta Wondo town comprises 3 Keble administrations. Aleta Wondo Town has a Latitude and Longitude of $6^0$36 N ' and 38^0 25' E respectively. The town is situated along Negele Borena, Kibre Mengist and Shakiso main road which passes separating the town in to two parts west and east.Aleta Wondo town is one of the Grade-2 reform Cities in the region and it has a town administration status consisting of three Kebele. The three kebele are named as Mesalemia, Chaffee and Della.

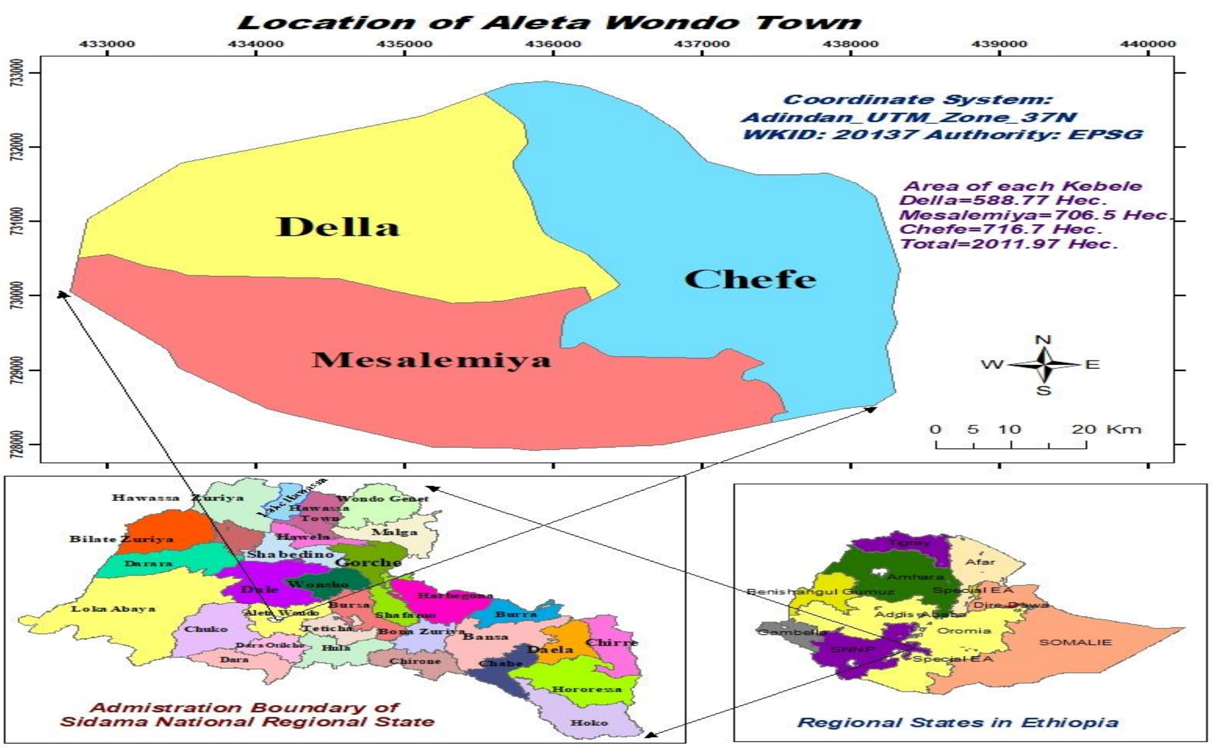


Figure 3. 1 Location map of Aleta Wondo Town from shape file.

3.1.2. Topography of the Town

The geographical area of the town is naturally sloppy and its altitude is 1950m above sea level. The topography of Aleta Wondo subjected to erosion.

3.1.3. Climate

The climate of the Aleta Wondo town is classified as a mid-land climate. It has a mean Annual Temperature of 18ºC; Mean Annual Rainfall of 1201-1670mm. January and February are the hottest months while July and August are the coldest months. The principal natural constraints for the physical expansion of the town is hilly & steep slope in the northern part, due to high rain fall in the area there is major problem of mud during summary and dust during winter in the town .There are also two rivers, called Melewo river and Tercha river flow in the town. These river are washing away the soil although the natural vegetation coverage protects as much as possible.

3.1.4. Population

Total population of the Aleta town in accordance with CSA estimation of July 2017 is 49,157 with male population of 24,890 and that of female population of 24,267. Annual average growth rate of population of the town is estimated at 4.8%.

3.1.5. Land Use Type

Land use of the town is grouped into ten such as residential, commercial, administrations, service, recreation, manufacturing and storage, Transport and roads, Forest and informal greenery, Urban agriculture, Special Function and non built up area, as it was shown on the table ,figure and using land use map of Aleta Wondo Town .

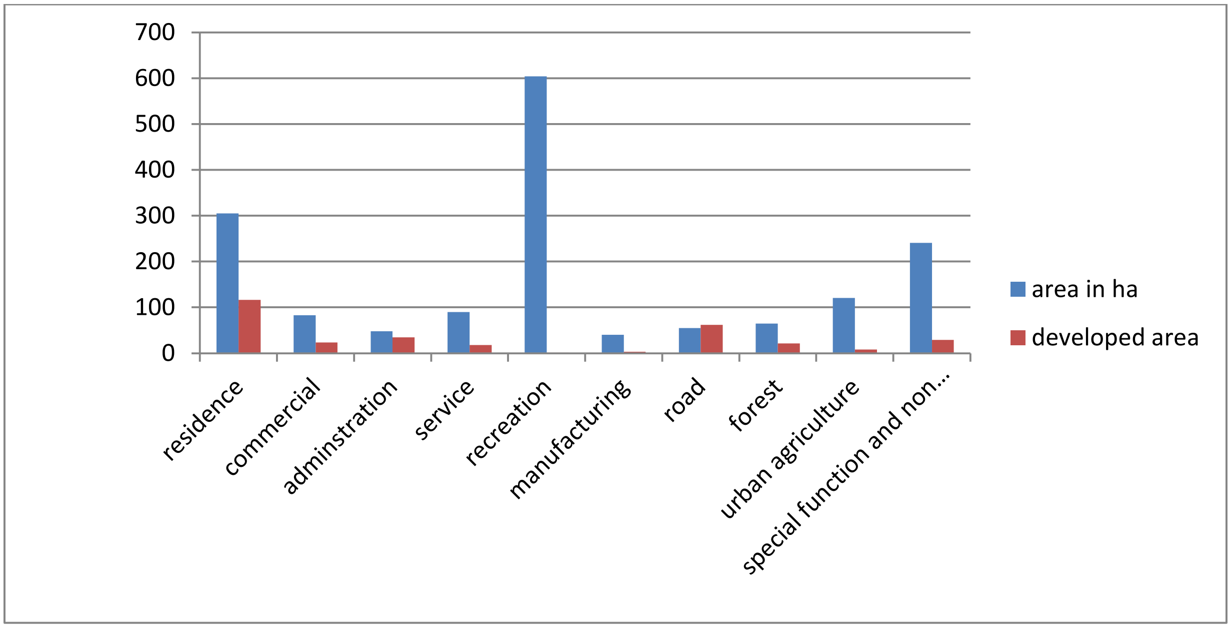


Figure 3. 2 Developed and undeveloped land use

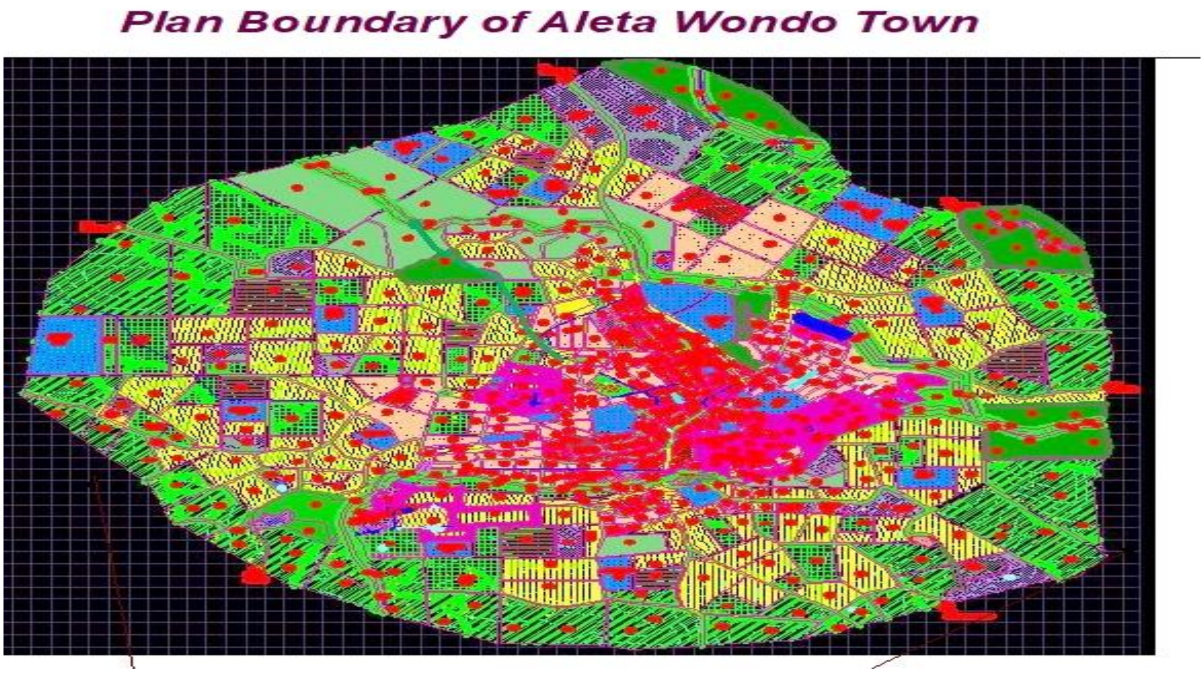


Figure 3. 3 Land use land cover

3.1.6. Soil Data

The soil in Aleta Wondo town is clay sandy soil. It covered by different type of vegetation. The topography of Aleta Wondo is subjected to erosion. There are also flood susceptible areas in the town along the river mentioned above. The discussion with municipal staff indicates that there were severe flood incidences along Melewo River few years back. The flood extent was serious that damaged several properties including the human life.

3.1.7. Drainage and Streetlight (Road Usability Enhancers)

The total length of drainage lines along the road sides is only 36.142km and out of which almost all km is open masonry drain type. However, majority of masonry drains in the sub-arterial roads have been abused as waste dumping sites especially in plane areas of the city. These activities are highly revealed in Cheffe kebele where the daily market exists and the market leftover are excessive and where waste disposal system is not well managed. All of the drainage systems in the city are deteriorated due to progressive erosion, absence of buffer zone and lack of preventive maintenance. The drainage density (drainage network to total planned area of the city is 2 km/ km^2. Street light coverage (Mean spacing of street light on municipal road) the city stood at 33%.

- The Backdrop Image

Aleta Wondo Town administration has Ground Control Points (GCPs) which are used for geo-referencing the aerial photo using either the digital data of the Structure Plan or the coordinates of Ground Control Points to carry out a process of geo-referencing. It is important to recognize that the images provided by the Federal Mapping Agency which was taken October 2009E.C. have what is termed a Rational Polynomial Coefficient embedded within the image text file, which is not the case with directly downloaded Google Earth images. Full Ortho-rectification takes account of perspective tilts and relief displacement in the landscape, which improves the accuracy of linear measurement in a two-dimensional mapping environment. Therefore, in these study used the aerial photo & Google Earth Images as a backdrop image and links to GIS directly without passing through geo-referencing.

Figure 3. 4 Aleta Wondo Town Drainage System map

3.2. Data Collection and analysis

3.2.1. Assessment of Current Situation Existing Drainage System

For this study, Before using and processing of any research, the primary task will be collecting relevant information and data of the study area. This section identifies and discusses the types of data and sources will be used for the study.

i. Primary Data

Field survey (visit) on the study area was carried out to evaluate the existing performance condition of drainage structures. Observing flood marks, measuring the size of the existing drainage structures, measuring the elevation difference between river/stream bed and flood mark as well as gathering information about the overall performance of drainage structures during the rainy season.The materials that will be used for the study of the research are digital camera, GPS device and measuring tape. All these materials was used during field visit of the study area for getting the primary data.

ii. Secondary Data

From the national Meteorological Service collected weathered data of rail fall in daily from 1991 to 2019 (29year) different stations of Aleta Wondo,Yirgalem,Aposto,Kabado and Teferkella observation gauged station

- ✓ Soil map
  - From ministry of agriculture soil classification of Ethiopia
  - From Aleta Wondo master plan Socio economic report
- ✓ Land use and land caver

From Sidama region urban planning management office use the land cover with delineated by Arc GIS and the Google earth maps views used .

- ✓ Material used for study

All rainfall data recorded daily intervals were, collected from National meteorological Agency. Generally the data collection and material used for each results of study used as follows:

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

3.2.2. Rainfall data

At present,there are several meteorological stations, which were installed by the National Meteorolical Agency (NMA) of Ethiopia.For the study area, rain falldata were obtained from NMSA and from Aleta Wondo-station which has a record of 24 hours daily rainfall for different durations of record (1991-2019). The rainfall data is the basic input to many hydrological models as it affects the resulting runoff peak and volume as well as sets in one rain gauge stations but due to the incompleteness of rainfall data additional four rain guage stations were used for this thesis to fill unrecorded rainfall data.

3.2.3. Estimating missing rainfall data

Measured precipitation data were important to many problems in hydrologic analysis and design. For gages that require periodic observation, the failure of the observer to make the necessary visit to the gage may result in missing data. Vandalism of recording gages was another problem that results in incomplete data records, and instrument failure because of mechanical or electrical malfunctioning can result in missing data. Any such causes of instrument failure reduce the length and information content of the precipitation record. A number of methods have for estimating missing rainfall data were station average method, normal ratio method, and inverse-distance weighting method, and regression methods. From the methods, the normal ratio is used for this study. There were 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 (Dingman,2002).Normal ratio method which is recommended to estimate missing data in region where annual rainfall among stations differ by more than 10%(Dingman,2002).This approach enables estimation of missing rainfall data by weighting the observation at Nx gauges by their respective with mean annual rain fall values of N1,N2,N3,and N4 then missing data Px can be estimated(Yilma,2011).as expressed by the equation below.

$$px = 1/4(p1\frac{Nx}{N1} + p2\frac{Nx}{N2} + P3\frac{Nx}{N3} + p4\frac{Nx}{N4}) \text{---} 3.1$$

Where

Px- missing rainfall data (daily, monthly or yearly)

P1,p2,p3 and p4 –rainfall data at nearest different stations (daily, monthly or yearly)

Nx-mean annual rainfall at the missed station.

N1,N2,N3 and N4 –mean annual rainfall at the different nearest station.

Table 3. 1 the percentage of missed data at each stations

Stations	Observed(year)	Available data(day)	Observed data(day)	Missing data(day)	% of missing data
Aleta wondo	29	10490	9950	540	5.14
Yirgalem	29	10490	10130	360	3.43
Aposto	29	10490	9920	570	5.43
Kabado	29	10490	10280	210	2.00
Tefer kella	29	10490	300	300	2.85

3.2.4. Data processing and checking the quality

i. Data quality checking for consistency

The consistencies of the data set of the given stations were checked by the double mass-curve method in-reference to their neighborhood stations. The double mass curve was plotted by using the annual cumulative total rainfall of the base station as ordinate and the average annual cumulative total of neighboring stations as abscissa.A consistent record was one where the characteristics of the record have not changed with time. Adjusting for gage consistency involves the estimation of an effect rather than a missing value. An inconsistent record may result from any one of a number of events; specifically, an adjustment may be necessary due to changes in observation procedures, changes in exposure of the gage, change in land use that make it unreasonable to maintain the gage at the old location, and where vandalism frequently occurs. Double mass curve analysis were used to check for inconsistency in a gage record. The method for checking the consistency of hydrological or meteorological records was considered an essential tool for taking it for analysis purposes. It was determined by plotting the cumulative values of the observed time series of the station for which consistency needs to be checked on the y-axis versus the cumulative values of observed time series of a group of the station on the x-axis . A break in slope of the curve would indicate that conditions have changed that location that was inconsistency and adjusted to consistent values by the rainbow. For this thesis Double- mass curve analysis were used to adjust inconsistent gage data.

ii. **Data quality checking for higher and lower outlier**

OUTLIER TEST

an outlier test is an observation hat deviates significantly from the bulk of the data helps to avoid those data lie out of the range in between the lowest datum and the highest datum,which may due to errors in data collection, or recording, or due to natural causes. The presence of outliers in the data causes difficulties when fitting distribution of the data .low and high outliers are both possible and have different effects on the analysis. The lowest and the Highest datum are calculated as follows:-

LOWEST DATUM

Y=LOGX---3.2

RL=10YL---3.3

Where YL= Yaveg-kn€n-1---3.4

Highest datum

RH=10YH---3.5

Where YH= Yaveg+ kn€n-1---3.6

Where Yavg = mean of data

€n-1=standard deviation of the data

kn = factor from (appendix.2) corresponding to the number of year data.

kn =2.549 for N= 29

Yavg=1.715414 n-1=standard deviation of the data=0.12269

YL = yavg-kn* εn-1

YL=1.715414-2.549*0.12269 = 1.402677

RL = 10YL = 25.27mm

YH=1.715414+2.549*0.12269=2.0516

RH =10YH =106.7mm

Therefore, the smallest and highest of the given datum were 25.27 and 106.7 mm respectively. Since there is no any data which is lower than 25.27 mm and higher than 106.7 mm. That means all the available data were satisfy the given condition.

3.2.5. Identification of the goodness fit tests of the probability distribution

The term frequency analysis refers to the techniques whose objectives are to analyze the occurrence of the hydrologic variable within the Statically framework, by using measured data and on statistical laws. The objective of frequency analysis of hydrologic data is to relate the magnitude of extreme events to their frequency of occurrence through the use of probability distributions. The historical rainfall data available is a 24hr duration rainfall hence appropriate IDF reduction methods need to be used to obtain rainfall intensities of shorter duration.

- The Goodness of Fit Test frequency

The goodness of fit (GOF) tests measure the compatibility of a random sample with a theoretical probability distribution function. These tests show how well the selected distribution fits data. The IDF reduction formula suggested in the Ethiopian Road Authority drainage Design Manual 2013 was used in this study. That is any probability distribution can be used as the model but the reliability of the distribution was checked by the goodness of fit tests. These tests are checked by (Easy Fit 5.6 software).

- **EasyFit5.6professionalsoftware**

The other method used in order to identify which distribution fits the theoretical probability distribution, GOF test conducted using Easy Fit 5.6 professional software. Kolmogorov Smirnov test, Chi-Squared tests, and Anderson-Darling were used in this. In all three tests, a parameter or statistic unique to each method was calculated for the required distribution types and these distributions are ranked based on their parameter values .

Kolmogorov-SmirnovTest

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). When comparing different di

Anderson-DarlingTest

The Anderson-Darling procedure is a general test to compare the fit of an observed cumulative distribution function to an expected cumulative distribution function. When comparing different distribution, lower statistics mean a better fit.stribution, lower statistics mean a better fit.

Table 3. 2 Goodness of fittest

No	Distribution	kolmogorovsmirnov		Anderson-Darling		Chi-squared	
		statistic	Rank	statistic	Rank	statistic	Rank
1	General extreme value	0.12814	1	0.41009	1	0.12638	1
2	Log-person 3	0.13024	4	0.43258	4	0.82992	4
4	Gumbel Max	0.13278	2	0.45785	2	0.84527	2
3	Gumbel min	0.17356	3	2.8606	3	1.2831	3

Therefore, the result obtained from Easy Fit 5.6 professional software the Gumbel distribution method was selected for the analysis of design rainfall for the required return period or 10 years for design and 25 years for check in this study. Because of the flood hazards that happened per 10 years in once we need to check the max hazard flood for the longest return period which means that for 25 years as suggested by Ethiopian Drainage Design Manual (ERA, 2013).

- **Gumbel Distribution Method**

It is one of the widely used probability distribution function for estimation of peak or maximum rain falls and expressed by this equation.

$X_T = \bar{X} + K_T * \delta n\text{-}1$ ---3.7

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

$K_T = (Y_T - \bar{y}n)/Sn$ --3.8

Y_T be a reduced variety, a function of T and is given by,

$Y_T = -\ln(\ln(T/(T-1)))$ --3.9

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

3.3. Development Of Intensity Duration frequency(IDF) Curve

The urban drainage design requires the Intensity-Duration-Frequency curve in the case of the specific study area. So that in this thesis for Aleta Wondo town the IDF Curve is developed specifically. The development of this curve is important for the study area in table.

Table 3. 3 Intensity Duration frequency (IDF) Curve

Duration(min)	T=2	T=5	T=10	T=25	T=50	T=100
5	49.4	77.1	95.4	118.6	135.7	152.8
10	78.3	122.1	151.1	187.8	215.0	242.0
20	59.0	92.0	113.9	141.6	162.1	182.4
30	47.6	74.3	92.0	114.3	130.8	147.3
40	40.0	62.4	77.3	96.0	109.9	123.7
50	34.6	53.9	66.7	82.9	95.0	106.9
60	30.4	47.5	58.8	73.0	83.6	94.1
70	27.2	42.5	52.6	65.3	74.8	84.2
80	24.7	38.5	47.7	59.2	67.8	76.3
90	22.6	35.2	43.6	54.1	62.0	69.8
100	20.8	32.5	40.2	49.9	57.2	64.3
110	19.3	30.1	37.3	46.3	53.0	59.7
120	18.0	28.1	34.8	43.2	49.5	55.7

130	16.9	26.4	32.6	40.5	46.4	52.2
140	15.9	24.8	30.7	38.2	43.7	49.2
150	15.0	23.5	29.0	36.1	41.3	46.5
160	14.3	22.2	27.5	34.2	39.1	44.1
170	13.6	21.1	26.2	32.5	37.2	41.9
180	12.9	20.2	25.0	31.0	35.5	40.0

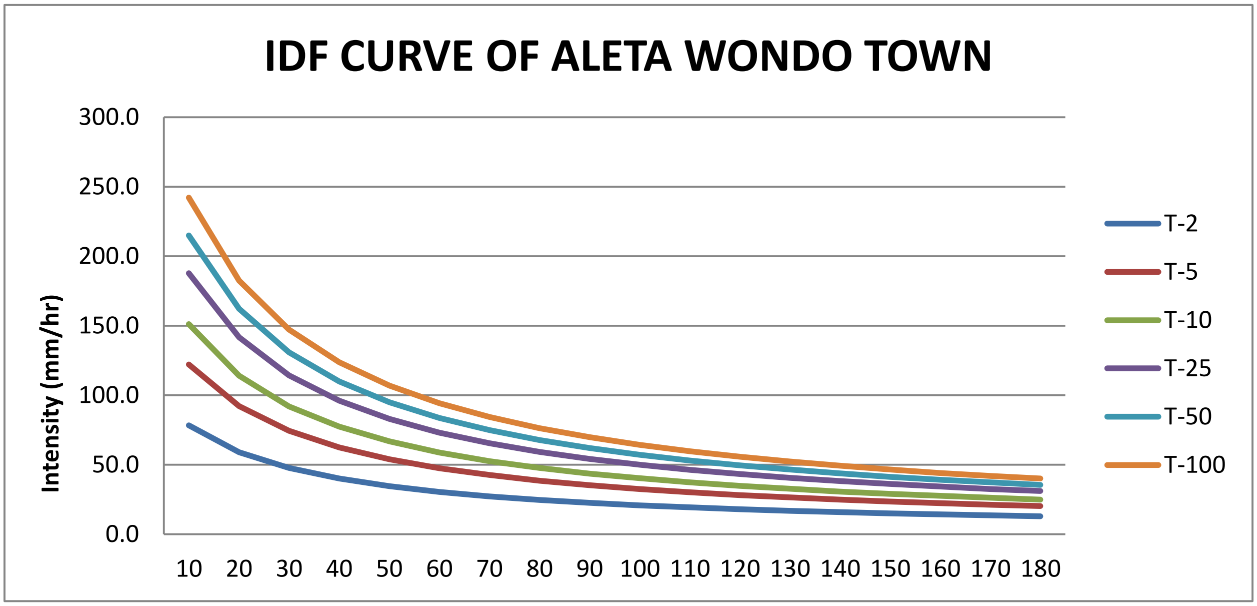


Figure 3. 5 Design rainfall of shorter duration

3.4. 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 reduction formula. Design and analysis of drainage structures require rainfall intensity-duration relationship of shorter duration. Because rainfall data of shorter duration is unavailable, appropriate IDF derivation for the shorter duration is required. Ethiopian Road Authority (ERA) Drainage Design Manual of 2013 suggests the following equation for calculation of shorter duration rainfall from 24hour duration rainfall were used.

$RRt = t(b + 24)n/24(b + t)n$---3.10

Where:

RRt = Rainfall depth ratio Rt: R24

Rt= Rainfall depth in a given duration t

R24 = 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, Gumbel distribution method were selected and R24 was calculated for 2, 5, 10, 25,50 and 100 year return period.

Rearranging the above equation:

$RRt = t(b+24)^n/24(b+t)^n$

Substituting Intensity (mm/hr) in the above equation

It = Rt/t --3.11

$It = R24(b+24)n/24(b+t)n$---3.12

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

3.5. Existing drainage system

From field survey, site visit and the cities CAD file document, storm water drainage system consists mainly of open channel drainage/ rod side (ditches and culverts). The storm water drainage system collects the runoff.

3.6. Hydraulic capacity and design of the Existing Drainage systems

Hydraulic properties and design of the existing drainage systems of Aleta wondo town were collected from field surveying as shown (Appendix.3, table 3)

- **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, and shape, roughness, also geometry and canal type.. The hydraulic design of the existing drainage system would be checked using the Manning formula.

$Q=AR^{2/3}S^{1/2}/n$

Where, Q = The volumetric flow rate passing through the channel reach in m3/s

A= the cross-sectional area of channel in m2

S= the bottom slope of the channel in m/m

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

n=manning roughness coefficient.

R=the hydraulic radius, which is the cross-sectional area of flow normal to the floe direction in m2 divided by the wetted perimeter of the cross-sectional area of flow in m.

Tape meter: to measure the existing storm water drainage lines depth, width, and diameter which helps to evaluate the capacity of the drainage system.

3.7. Design Flood Estimation

3.7.1. Rational Method

According to ERA Drainage Design Manual (EDDM) 2013, this method estimates peak runoff rate for small urban and rural watersheds of less than 50ha (0.5 m2) .The rational formula estimates the peak rate of runoff at any location in the catchment area as a function of the catchment area, runoff coefficient, and rainfall intensity for duration equal to the time of concentration (the time required for water to flow from the remote point of the basin/watershed to the outlet (interest) point. This method will used to calculate the peak runoff of the existing carrying capacity of the main drainage canal system for a certain return period of rainfall intensity as equation 3.13.

Qp = 0.00278CiA --3.13

Where: QP - Peak run-off the catchment (m3/sec), C=Runoff coefficient of the catchment, i = rainfall intensity in (mm/hr.), A = catchment area in (ha). (Source: VDOT, 1995).

The following procedure outlines the rational method for estimating peak discharge:

Determine the watershed area, the time of concentration, assure consistency with the assumptions and limitations for application of the rational method, Select the appropriate runoff coefficient; and Compute the peak discharges for the watershed for the desired frequency. And the main input variable of the rational method: Rainfall intensity, Time of concentration, Rainfall duration, Rainfall frequency, Catchment area.

a. Runoff Coefficient (C)

The runoff coefficient C is the function of the land use land cover of the study area and 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 = (A1C1 + A2C2 +---+ AnCn) / (A1 + A2 +---+ An) --------------------------------------3.14

Where: Cw= Weighted Runoff Coefficient, C1, C2, ---- Cn = coefficient of runoff for parts of the drainage area. A1, A2, An = parts of drainage areas with different runoff coefficients.

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.

Tc = Tc inlet + Tt--3.15

Where: Tc = time of the concentration (minute), Tci = time of concentration inlet (hrs.) and

Tt = the travel time (hr)

- Time of concentration inlet (Tci)

The time of concentration of drainage area is the time required for runoff from the farthest part of the drainage area to reach the outlet used as the duration for the design storm. The inlet time for the time of concentration is estimated using Airport or Federal Aviation Administration (1970) methods. This method could desirably be used for inner areas especial for developed areas of urban centers as equation 3.16.

$$Tci = \frac{3.64(1.1-c)(\frac{L}{1000})^{\wedge}0.83}{H^{0.33}} \text{---} 3.16$$

Where: Tci = time of concentration inlet (hrs.), C = runoff coefficient

L = flow length from the remotest point to the point t of interest (km).

H = elevation difference between the upstream and downstream of the canal (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 is the ratio of flow length to flow velocity as equation 3.17.

T_t = L/ (3600V) ---3.17

Where: T_t = 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.

- Flow velocity (V)

The flow velocity is computed using the Manning formula as shown in equation 3.18.

$$V = \frac{1}{n} * R^{\frac{2}{3}} * S^{1/2} \text{---} 3.18$$

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)

Is the average rainfall rate in (mm/hr) for a particular drainage basin or sub-basin the design duration is equal to the time of concentration for the drainage area under consideration. And selected on the basis of the design rainfall duration and return period.

d.Catchment Area (A)

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

Generally, In the design of storm water drainage system, the main purpose of hydrologic analysis is to determine the maximum amount of run-off (peak discharge) that can be accumulated at certain storm drainage outlet (usually a ditch) along a highway/access road alignment section. The Rational method, one of the most commonly used simplified models for road storm drainage, is primarily based on the concept that the peak discharge from a watershed will always occur when the rain lasts long enough at its maximum intensity to enable all portions of the basin to contribute to the flow. Rational Method is appropriate of the area catchment is less than 50 hectare (0.5sqkm). The peak runoff is given by the following expression by

Q = 0.00278 * C * I * A --3.19

Where: Q = Discharge at outlet (m^3/s)

C = Rainfall-Runoff Coefficient

I =Maximum probable rainfall Intensity (mm/hr)

A=Catchment Area (hectares)

The procedure outlines the rational method for estimating peak discharge:

- ✓ Determine the watershed area
- ✓ Determine the time of concentration
- ✓ Assure consistency with the assumptions and limitations for application of the rational method
- ✓ Select the Select the appropriate runoff coefficient
- ✓ Compute the peak discharges for the watershed for the desired frequency

3.7.2. Soil Conservation Service (SCS) Method

The method is developed by the U.S. Soil Conservation Service for calculating rates of runoff requires the same basic data as the Rational Method: Catchment area, a runoff coefficient, time of concentration, and rainfall. However, the SCS method, is more sophisticated in that it considers also the time distribution of the rainfall, the initial rainfall losses to interception and depression storage, and an infiltration rate that decreases during the course of the storm, either real or fabricated, by subtracting infiltration and other losses from the rainfall to obtain effective precipitation. It is therefore, potentially more accurate than the rational method and is applicable when the catchment area is larger than 50 hectares (ERA, 2011).

The main input variables for the SCS methods:

- ✓ Drainage area,
- ✓ weighted runoff curve number (CN),
- ✓ Time of concentration (Tc) and
- ✓ Design point rainfall (P)

$$\boldsymbol{qp = \frac{0.208Ard}{0.5d+0.6tc}} \quad \text{--3.20}$$

Where: qp = peak discharge (m^3/s)

rd = the excess rainfall depth (mm)

A = watershed area (km^2)

d= duration of rain fall event (min)

tc = time of concentration(min)

The depth of runoff resulting from a required return period rainfall depth of duration corresponding to the time of concentration tc is estimated by

$\boldsymbol{rd} = \frac{(\boldsymbol{P-0.2S})^{\mathbf{2}}}{\boldsymbol{P+0.8S}}$for P > 0.2S ---3.21

Where: Ia = Initial Abstraction (Ia = 0.2S)

rd = Excess Rainfall

P = Total Rainfall

S = Potential Maximum Storage

The potential maximum soil water retention, S, is related to hydrologic soil properties, land cover and management hydrologic soil properties, land cover and management conditions as well as the soil moisture status of the catchment prior to rainfall event and there relation with curve number(CN) is

$S = \frac{25400}{CN} - 254$---3.22

3.8. EPA's Storm water Management Model (SWMM)

The storm water management model was first developed in 1971, at Washington DC and has undergone several major upgrades. Since then, it continues to be widely used throughout the world for planning, analysis and design related to storm water runoff, combined sewers, sanitary sewers, and other drainage systems in urban areas. SWMM is a physically based, discrete-time simulation model. It 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 and pollutant loads. The routing portion of SWMM transports this runoff through a system of pipes, channels storage/treatment devices, pumps, and regulators (Lewis and Huber, 2016).

3.8.1. Modeling Capabilities of the SWMM

SWMM accounts for various hydrologic processes that produce runoff from urban areas. including: Time varying rainfall ,Evaporation of standing up surface water ,Snow accumulation and melting ,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 ,Nonlinear reservoir routing of overland flow .

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).

3.8.2. Application of SWMM

SWMM applications have been used in sewer and storm water studies throughout the world. Its typical applications include:- designing and sizing of 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).

3.8.3. 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 storm water runoff quantity and quality through the following physical processes.

- ✓ Surface Runoff

Inflow comes from precipitation and any designated upstream sub catchments. Each subcatchments 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 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 over the course of a long rainfall event. The maximum and minimum infiltration rates, a decay coefficient that describes how fast the rate decreases over time and a 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 in excess of 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 the flow area (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 really 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 .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

Is 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 a thirty seconds or less (Rossman, 2004).

In this study, the rational method and Stormwater management model (SWMM) was used for the design flood computation and its analysis. The rational method would be compared with the existing carrying capacity of the main drainage canal system for a certain return period of rainfall intensity. Whereas the stormwater management model is one-dimensional models which allow the flow properties to vary along or within the channel only rather than to account the changes across the channel (Rossman, 2004).

Model Set up procedures are:

- ✓ Set the coordinates of area map/image
- ✓ Draw network representative and describe sub catchments
- ✓ Edit the properties of the object that make up the system
- ✓ Descibe how the system is operated

Select a set of analysis options Run Simulation for Rainfall/Runoff and Flow routing

3.8.4. Model Preparation Area

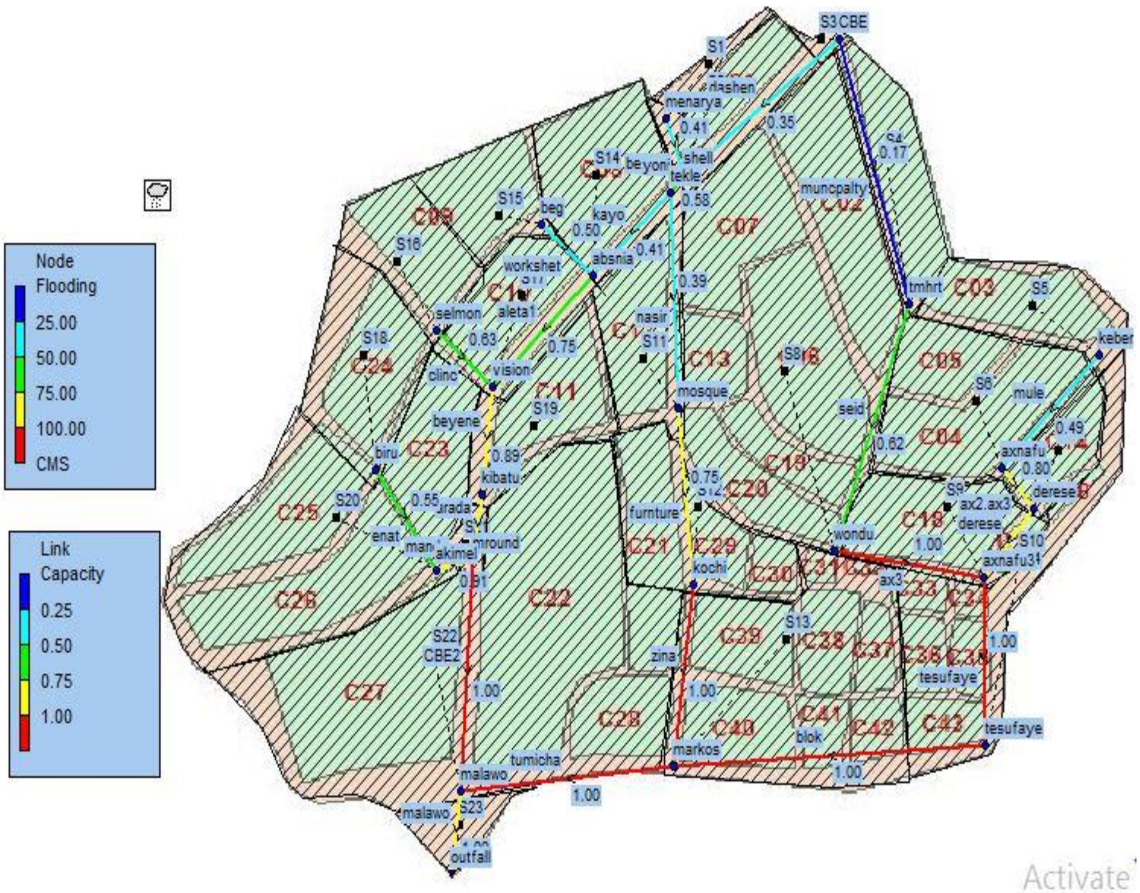


Figure 3. 6 Model Preparation Area

The above figure shows modeled area of outlet one starts from CBE to Shell. That simulated model of each 23 sub catchments connected into 22nodes and 13 channels and flow routing outfalls into Malawo that drainage line contribution to Malawo River.

The manholes/Junctions were modeled as rectangular channel with different depth. It had assumed that there are no energy losses in the manholes. The precipitations were introduced into the model by associating each sub-catchment to the rainfall time-series.

3.8.5. Calibration and validation of model

The SWMM Model was able to predict the observed output with the reasonable accuracy; the sensitivity analysis was performed by changing each parameter while keeping all others constants and observing the changing in model output using the recent swmm5.1 obtained from the hydraulic model calibration and validation process were done for Aleta Wondo town. The most parameters used for sensitivity analysis and allowable range of change proposed by(Weaver and Nachabe, 2018) The Observed data were used in the model calibration, a process made by a manual trial and error method and the simulated and the observed runoff in the outlets were compared in table.4

Table3.4 Name Of sensitivity parameters of model and values

<table>
<tr><th>Name of parameter</th><th colspan="2">Meaning</th><th>Value Range</th><th>Initial Values</th><th>Used Values/Sensitivity parameters</th></tr>
<tr><td>N-Imperv</td><td colspan="2">Manning’s roughness Coeff. For Impervious Area</td><td>0.011-0.015</td><td>0.011</td><td>0.014</td></tr>
<tr><td>N-perv</td><td colspan="2">Manning’s roughness Coeff. For pervious Area</td><td>0.05-0.8</td><td>0.1</td><td>0.7</td></tr>
<tr><td>Destore-Imperv</td><td colspan="2">Depth of depression storage on Impervious Area</td><td>0-3</td><td>1</td><td>2.5</td></tr>
<tr><td>Destore- perv</td><td colspan="2">Depth of depression storage on pervious Area</td><td>3-10</td><td>3</td><td>8</td></tr>
<tr><td>Smooth massonery Roughness</td><td colspan="2">Manning’s roughness Coeff. For open channel with smooth massonery</td><td>0.033</td><td>0.033</td><td>0.033</td></tr>
<tr><td rowspan="3">Infiltration method</td><td rowspan="3">Green Amp</td><td>Suction</td><td>3.5</td><td>3.5</td><td>3.5</td></tr>
<tr><td>Conductivity</td><td>0.5</td><td>0.5</td><td>0.5</td></tr>
<tr><td>Initial deficit</td><td>0.25 – 0.26</td><td>0.25</td><td>0.26</td></tr>
</table>

Calibration and validation data

SWMM model calibration and validation checked rainfall through basin and recorded stream flow at drainage system was needed. Because of no flow gauges installed in Aleta Wondo town drainage system and unavailability of recorded data is difficult to obtained data. In order to model accuracy recorded flow depth data in open rectangular channel from Aleta Wondo Town rainfall station selected around outlet .this flow depth was recorded at different interval for 12 days from June to quarter of September to calibration of sensitivity parameter and validate swmm5.1 model for the area.

Table 3.5 Recorded flow depth for calibration

Guaged Area	Type channel	Date of Data measured	Recorded flow depth(m)	Flow calculation $Q=1/nAR^{2/3}S^{1/2}$
For Calibration	Rectangular	June 15,2021	0.6	Each values will be filled in result and descution part
	Rectangular	July 8,2021	0.51	
	Rectangular	July14,2021	0.5	
	Rectangular	Augest5,2021	0.53	
	Rectangular	Augest 22,2021	0.6	

RECORDED FLOW DEPTH FOR VALIDATION MODEL

Guaged Area	Type channel	Date of Data measured	Recorded flow depth(m)	Flow calculation $Q=1/nAR^{2/3}S^{1/2}$
For Validation	Rectangular	July2,2021	0.6	Each values will be filled in result and descution part
	Rectangular	July 22,2021	0.51	
	Rectangular	July 30,2021	0.5	
	Rectangular	Augest 15,2021	0.53	
	Rectangular	september 5,2021	0.6	

From the above table 3.5 six days rainfall data observer used for calibration the model with flow calculated using maximum flow depth (m) recorded and the other left six days used for validation of the model.

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

$Q=1/nAR^{2/3}S^{1/2}$---3.23

Whereas Q=flow discharge in m3/s

n=manning roughness coefficient

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

R (A/P) =hydraulic radius in m

P=wetted parameter in m

S=channel slope in m/m fraction

4. RESULTS AND DISCUSSIONS

4.1. Data Availability

4.1.1. Data Quality Check Using Double Mass Curve

The consistencies of the data set of the given stations were checked by the double mass-curve method in-reference to their neighborhood stations. The double mass curve was plotted by using the annual cumulative total rainfall of the base station as ordinate and the average annual cumulative total of neighboring stations as abscissa.

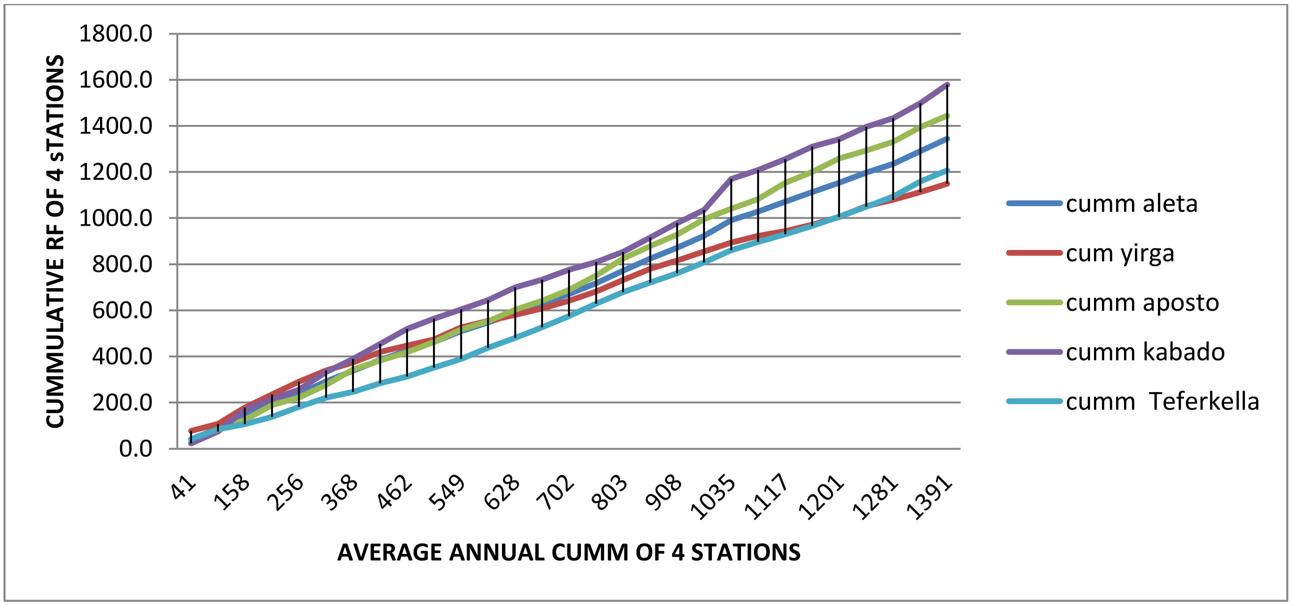


Figure 4. 1 Double mass curve before consistency of rainfall data was done.

A break in slope of the curve would indicate that conditions have changed that location was inconsistency and adjusted to inconsistent values by the rainbow. For this thesis Double mass curve analysis and rainbow soft ware used to adjust inconsistent gage data.

Inconsistency of record is corrected by using double mass curve technique:-

$Pcorrected = Px\ Mx/Ma$---4.1

Where Mc = corrected slope

Ma = original slope

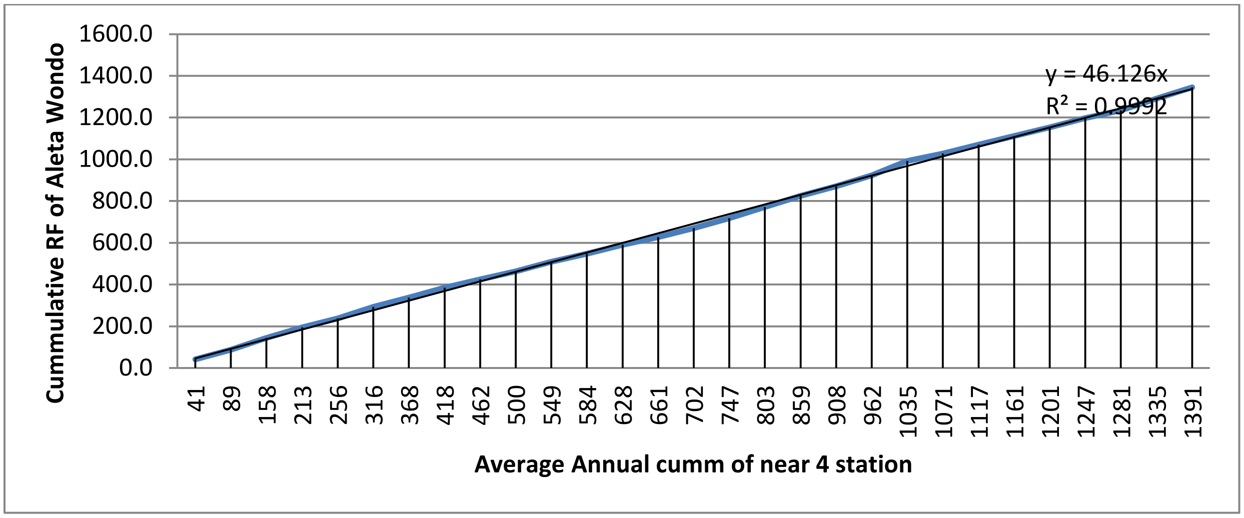


Figure 4. 2 Double mass curve after Consistency of rainfall check

4.1.2. Data quality checking for higher and lower outlier

Outlier test

an outlier test is an observation hat deviates significantly from the bulk of the data helps to avoid those data lie out of the range in between the lowest datum and the highest datum,which may due to errors in data collection, or recording,or due to natural causes.The presense of outliers in the data causes difficulties when fitting distribution of the data .low and high outliers are both possible and have different effects on the analysis.The lowest and the Highest datum are calculated as follows:-

- ✓ Lowest datum

Y=LOGX

RL=10YL

Where YL= Yaveg-kn€n-1

- ✓ Highest datum

$RH=10^{YH}$

Where YH= Yaveg+ kn€n-1

Where Yavg = mean of data

€n-1=standard deviation of the data

kn = factor from (appendix.1) corresponding to the number of year data.

kn =2.549 for N= 29

Yavg=1.715414 n-1=standard deviation of the data=0.12269

YL = yavg-kn* εn-1

YL=1.715414-2.549*0.12269 = 1.402677

RL = 10YL = 25.27mm

YH=1.715414+2.549*0.12269=2.0516

RH =10YH =106.7mm

Therefore, the smallest and highest of the given datum were 25.27 and 106.7 mm respectively. Since there is no any data which is lower than 25.27 mm and higher than 106.7 mm. That means all the available data were satisfy the given condition.

4.2. Current Situation of Existing Drainage System of the Aleta Wondo Town

From field survey, site visit and the Aleta Wondo Town drainage system consists mainly of open channel drainage / rod side (ditches and culverts). Currently, condition of existing drainage services within the study area based on performances and strength of drains. It was found that most of the drains condition were worst type (i.e., 21.49m), followed by 14.7m worse, 4.8m bad, and 0.722m good drainage condition respectively. Therefore, both engineered infrastructures and non-structural approaches are needed to improve existing urban drainage system and to reduce related problems considering geographical location Aleta Wondo Town. Similarly, the quality of the present drainage facilities is not good. Therefore, drainage facilities related data was also collected from the study area to as certain the quality of existing urban drainage system . The collected data are presented in (Table 3). Here, 77.5 % of the urban drainage system was found not well designed, highly fractured, very narrow gutters, and not well-plastered gutters in Aleta Wondo Municipal.In general, the quality of drainage facilities in Aleta Wondo Town was found to be poor in quality. Hence, there is need to improve the existing Urban drainage system with-a-view-to provide a better environment for city dwellers as-well-as ensure sustainable development.

Table 4.1 Drainage categories condition assessment

Drainage conditionassessments						
		Condition				
Types of drainage	Total length(M)	Very good (M)	Good (M)	Moderate (M)	Poor (M)	Very poor (M3)
Masonry drainage	21.49		4.888	0.722	1.151	14.726
			22.%	3.5%	5.5%	68.5%

4.2.1. Performance of the Drainage System in the Study Area

Drainage begins by examining the performance of current drainage systems, coverage of the drainage system and evaluating the current conditions of the drainage system**.**

Table 4.2 Drainage Categories Condition Assessment

Drainage	Drainage pavement	Road start	Road End	Length(m)	width	Depth(m)	Existing condition
Open drainage	Masonery	Zerabruk hotel	Mesalemiya round	850	0.7	0.5	Good
Open drainage	Masonery	Zerabruk hotel	Mesalemiya round	850	0.7	0.5	Moderate
Open drainage	Masonery	CBE	Telle-communication	500	0.7	0.5	Moderate
Open drainage	Masonery	Tekle Hotel	CheffeLocal market	500	0.7	0.5	Moderate
Open drainage	Masonery	Tekle Hotel	CheffeLocal market	250	0.7	0.5	Moderate

Open drainage	Masonery	Local market	malawo	495	0.7	0.5	Poor
Open drainage	Masonery	Local market	malawo	495	0.7	0.5	Poor
Open drainage	Masonery	Melestiya Elementary school	Mesalemiya round	240	0.7	0.5	Poor
Open drainage	Masonery	Bus station	Temket Baher	340	0.7	0.5	Poor
Open drainage	Masonery	Melestiya elementary school	Oromia bank	220	0.7	0.5	Poor

Table 4. 3 Condition indicators of drainage channels and pipes

Level	Rating g scale	Condition indicator
1	Very Good	Shape of drain still in original design condition
2	Good	Drainage functions easily fulfilled
3	Moderate	Drainage effective but slightly impaired
4	Poor	Design function impeded due to siltation, vegetation or scour
5	Very Poor	Drainage non-functional

Drainage systems are not well connected; do not have the capacity to carry large amounts of water, hence resulting in overflowing. In some areas, drainage systems were not provided, and some of the existing drainages have been silted by sand, other rubbish, and waste materials. Especially, the great problem in the study area was lack of waste management techniques (like manholes and trash bin). In the case of Aleta Wondo Town, the existed manholes were out of service and have been clogged with waste and blocked due to lack of clearance. Additionally, at different places, they were not constructed. as a result, the runoff that is generated in that sub-basin overflows with a higher velocity which erodes the ditches as well as the road and walkways.

The spatial data management system employed for this study area in GIS. The system automatically generates a unique feature . However, the issue that requires attention is that the placement of the start and end points of linear features such as roads and water mains. Features are represented on the map based on their physical nature and descriptive indicators. To this end, roads are represented with their centerlines, having start and end nodes. They are described according to their attributes . The same applied for drainage and water main pipes. Valves, streetlights, reservoirs, public standpipes, and other point features are represented using points on the map and described according to their characteristic attribute information collected during in this study stage.

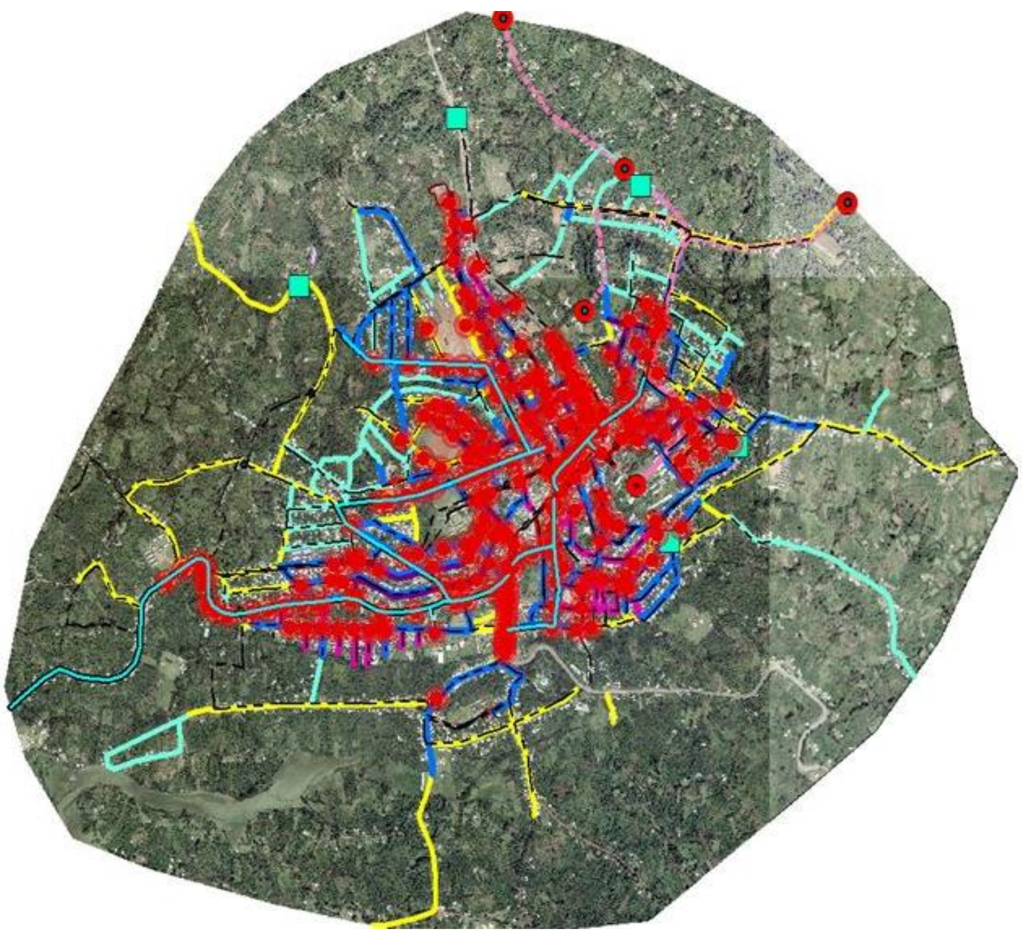


Figure 4. 3 Aleta Wondo City Partial Road and Drainage centerline nodes map

4.2.2. Drainage Network

Table 4. 4 Urban Drainage System of AletaWondo Town

S/N	Asset Category	Features	Unit	Total
	Drainage Network	Masonry open drainage	KM	21.49

Figure 4.4 Aleta WondoTown Drainage System map

4.2.3. Residential increment and impervious surface

Land use of the town is grouped in residential, commercial, administrations, service, recreation, manufacturing and storage, Transport and roads, Forest and informal greenery, Urban agriculture, Special Function and non built up area, as it was indicated on the land use map of Aleta Wondo Town.

Table 4.5 Land use Land cover

S/N	LAND USE TYPE	Planned LU. in Ha	Share from total land (%)	Developed LU		Difference	
				in Ha	In %	in Ha	In %
1	Residence	305	15.14	83.38	27.33	221.62	(72.66
2	Commerce	82.6	4.10	35.85	43.4	46.74	56.6
3	administration	48	2.38	2.26	0.47	45.74	95.3
4	Social services	90	4.47	61.5	68.33	28.5	31.66
5	Greenery and recreation	604.2	29.9	62.84	0.14	603.6	99.86
6	industry	40	1.99	23.289	58.22	16.71	41.78
7	road	604.2	29.99	61.6	7.8	542.61	89.9
8	agriculture	65	3.23	7.94	12.22	57.06	87.78
9	Open areas and forest	55	2.73	0.6	0.17	54.4	98.9
10	Special fnction	120.6	5.99	11.1	15.86	109.5	90.79
TOTAL		2014.16		549.25		1464.91	

Due to residential increased through the town without drainage system addition, the existing drainage systems become busy and create high runoff through the town. Basically, the residential increment of the Aleta Wondo Town creates a very high run-off, and a good adequate drainage system would be a proactive method of combating the effect of excess flood in town. Additionally, due to the highly impervious surface as a result of built-up area, the rainfall doesn't infiltrate into the ground. This causes in undation over the entire area, reduce groundwater recharges, a higher velocity which creates scouring of the drainage structures and increases the surface runoff by which it affects those channel that conveys this runoff into the existing drainage systems.

4.2.4. Dumping of solid wastes into storm drainage Systems

Collecting and managing solid and human waste is an important challenge of the Town. This problem is often magnified in Town where a dense concentration of people leads to a substantial amount of waste generation. Waste management is a major problem in Town. Trash, debris, junk, floatables, gross pollutants, rubbish or solid waste) has become a major problem through the town. It typically consists of manufactured materials such as bottles, cans, plastic and paper wrappings, newspapers, shopping bags, cigarette packets and remains of chat. As a result of dumping such solid wastes into drains the drainage systems have been clogged and cause flooding over streets and drainage failure ,as shown fig .

Figure 4.5 Failed drainage system by the dumping of solid waste

4.3. Hydraulic capacity and design of the Existing Drainage systems

Hydraulic properties and design of the existing drainage systems of Aleta wondo town were collected from field surveying .The existing storm drainage facilities were generally classified into closed and open drainage lines and constructed by masonry and rectangular geometry. By applying the field survey, the dimensions of drainage systems were recorded. So that the amount of discharge conveyed in the existing drainage system could be determined by the Manning equation.

Table 4.6 Hydraulic capacity of existing drainage system

Channel type	Record depth (m)	Manning roughness used(n)	channel slope(S)	Width (m)	Area (m2)	Parameter (m)	Hydrulic radius (m)	Velocity (m/s)	Discharge Q=A*V(m^3/s)
Rectangular	0..5	0.013	0.012	0.7	0.35	1.7	0.206	2.93	1.02
Rectangula	0.5	0.013	0.023	0.7	0.35	1.7	0.206	4.06	1.42
Rectangula	0.5	0.013	0.027	0.7	0.35	1.7	0.206	4.4	1.54
Rectangula	0.5	0.013	0.024	0.7	0.35	1.7	0.206	4.16	1.45
Rectangula	0.5	0.013	0.027	0.7	0.35	1.7	0.206	4.06	1.54
Rectangula	0.5	0.013	0.018	0.7	0.35	1.7	0.206	3.6	1.26
Rectangula	0.5	0.013	0.023	0.7	0.35	1.7	0.206	3.18	1.42
Rectangula	0.5	0.013	0.011	0.7	0.35	1.7	0.206	2.81	0.98
Rectangula	0.5	0.013	0.025	0.7	0.35	1.7	0.206	4.2	1.47
Rectangula	0.5	0.013	0.018	0.7	0.35	1.7	0.206	3.9	1.26
Rectangula	0.5	0.013	0.025	0.7	0.35	1.7	0.206	3.9	1.46
Rectangula	0.5	0.013	0.015	0.7	0.35	1.7	0.206	3.2	1.15

4.4. Peak discharge of study area

Proper selection of the runoff coefficient calculated depending on the different land cover that contributes to the sub –catchment of the drainage area. The portion of the total rainfall that would reach the storm drains depends on the percent imperviousness, slope and ponding character of the surface. The frequency analysis of the extreme rainfall events analyzed using the Gamble method and then peak flood is calculated using rational method as presented in table.

Table 4.7 Estimated drainage capacity and Existing drainage capacity

SC	A(m2)	L(m)	h1-h2(m)	C	S	v(m/s)	TC(m in)	intesity10(mm/hr)	intensty 25	Q(m3/s)10	Q25(m3 /s)
S1	0.32 11	200	7	0.8	0.035	2.82	10	150	167.3	0.11	0.12
S2	1.87 4	250	13	0.7	0.051	13.5	12	120	130	0.44	0.47
S3	0.32 11	50	1	0.8	0.024	4.3	5	170.2	183.7	0.12	0.13
S4	0.86 6	250	8	0.5	0.030	3.73	22	97	106	0.12	0.13
S5	0.86 6	200	8	0.6	0.038	0.74	19	103	125.1	0.15	0.18
S6	1.82 3	250	12	0.7	0.048	2.89	14	119	137	0.42	0.49
S7	0.52 5	50	18	0.5	0.354	2.88	5	170.2	183.7	0.12	0.13
S8	5.74 8	400	20	0.7	0.050	2.88	18	104	126	1.16	1.41
S9	1.32	100	4	0.5	0.043	3.73	12	120	130	0.22	0.24
S10	0.03 5	250	4	0.8 5	0.016	3.58	12	120	130	0.01	0.01
S11	0.89 6	200	2	0.7	0.008	1.17	23	96	105	0.17	0.18
S12	1.65 6	250	16	0.7	0.062	3	13	119.6	137.6	0.39	0.44
S13	3.76	200	8	0.5	0.038	2.88	19	103	125	0.54	0.65
S14	1.36 6	210	1	0.7	0.004	1.21	28	97.5	127	0.26	0.34
S15	0.76 4	250	6	0.7	0.023	3.58	17	105.1	126.1	0.16	0.19
S16	0.76 4	250	7	0.7	0.028	3.58	16	106.7	126.7	0.16	0.19
S17	0.83 2	180	7	0.7	0.039	1.17	14	119	137	0.19	0.22
S18	0.88	200	2	0.7	0.011	3.58	19	103	125	0.18	0.21
S19	0.88 5	300	10	0.7	0.033	3.33	17	105.1	126.1	0.18	0.22
S20	2.44	400	9	0.7	0.021	3.22	22	97	106	0.46	0.50
S21	1.09 8	50	4	0.7	0.079	2.89	5	170.2	183.7	0.36	0.39
S22	6.11 8	350	6	0.7	0.017	1.05	26	96.3	129.4	1.15	1.54

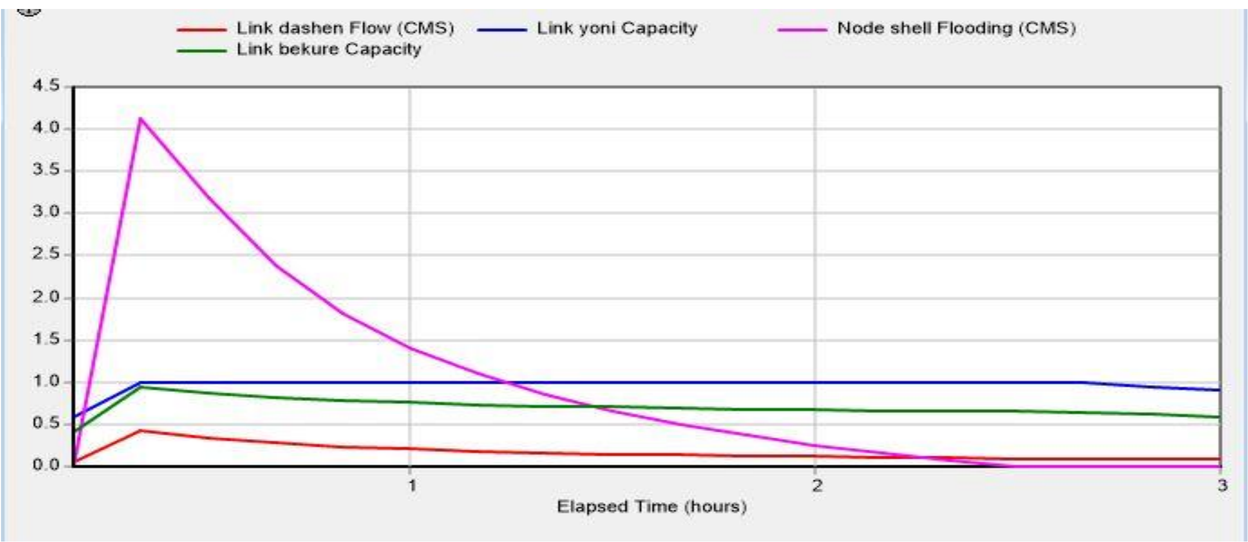
4.5. The Storm water Management Model (SWMM) simulation

4.5.1. Water Level and Flood Level in the Junctions

After the input parameters of the Model were concluded, the Storm water management model run was successful. The simulation results were obtained based on the duration of 3hrs rainfall intensity. So that the simulation results at junctions and Links were presented below.

A. Drainage from CBE to Shell

The maximum designed water level of drainage canal of juntions Dashen , Yoni and Bekure were 0.4 m,1m and 0.97 m respectively. The flooding level attained with 3hrs rainfall intensity injunction in these three junctions are 4.1m. Based on the simulation result of the model for the current land use condition, the designed canal depth was insufficient. Therefore there is the overflow of the water at junctions CBE and Shell due to inefficient canal capacity .



Link Flow — Click a column header to sort the column.

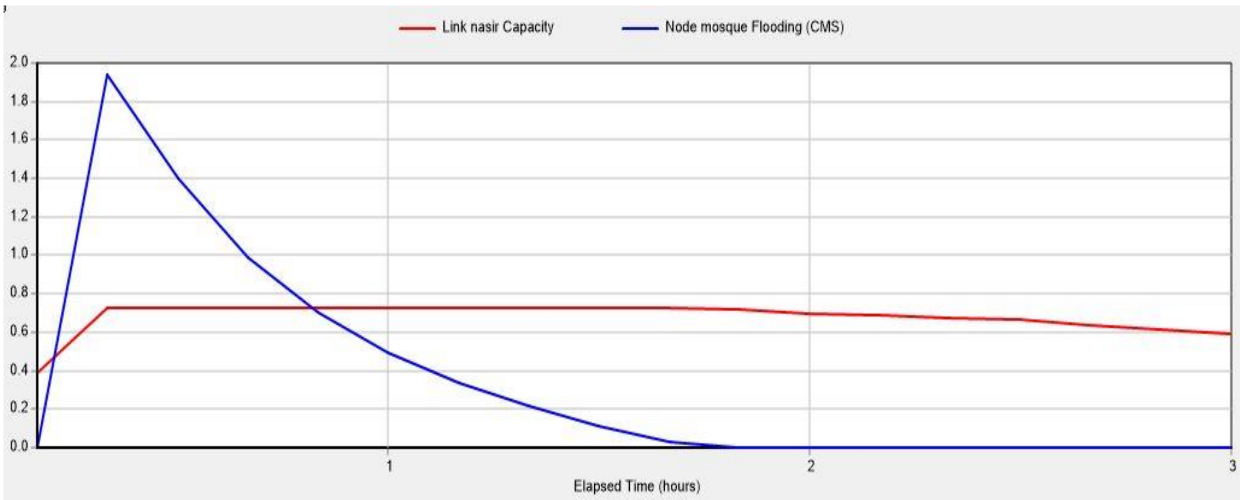
Link	Type	Maximum |Flow| CMS	Day of Maximum Flow	Hour of Maximum Flow	Maximum |Velocity| m/sec	Max / Full Flow	Max / Full Depth
dashen	CONDUIT	0.435	0	00:20	1.80	0.27	0.69
bekure	CONDUIT	0.747	0	00:20	2.25	0.87	0.95
yoni	CONDUIT	1.227	0	00:13	3.50	1.05	1.00

Figure 4.6 Drainage from CBE To Shell

From the abave figure4.6 result,it was studied that the drainage system overflow of the water at junctions, Dashen,Yoni and Bekure .Which are limited in capacity and that not to convey flow inside the channel at different high rain season time.

B. Drainage around Areb sefer

The maximum designed water level of drainage canal of juntions mosque was 0.73m. The flooding level attained with 3hrs rainfall intensity injunctions is 1.94m3/s.Based on these thesimulation result of the model for the current land use condition,the designed canal depth was insufficient .Therefore there is the overflow of the water at junctions to inefficient canal capacity.



Link Flow — Click a column header to sort the column.

Link	Type	Maximum |Flow| CMS	Day of Maximum Flow	Hour of Maximum Flow	Maximum |Velocity| m/sec	Max / Full Flow	Max / Full Depth
nasir	CONDUIT	0.660	0	01:43	2.83	0.35	0.73

Node Flooding — Click a column header to sort the column.

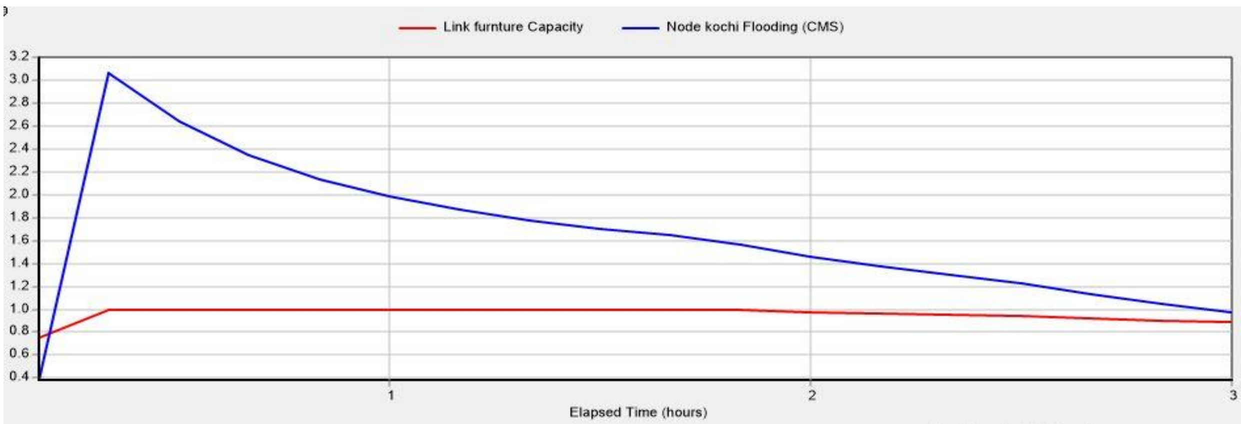
Node	Hours Flooded	Maximum Rate CMS	Day of Maximum Flooding	Hour of Maximum Flooding	Total Flood Volume 10^6 ltr	Maximum Ponded Depth Meters
mosque	1.51	1.943	0	00:20	3.551	0.000

Figure 4.7 Drainage around Areb sefer

From the above figure result, it was studied that the drainage system overflow of the water at junctions, Mosque,.Which are limited in capacity and that not to convey flow inside the channel at different high rain season time.

C. Drainage around Local Market

The maximum designed water level of drainage canal of junctions furniture was 1.0m. The flooding level attained with 3hrs rainfall intensity injunctions is 3.0m.Based on these the simulation result of the model for the current land use condition, the designed canal depth was insufficient .Therefore there is the overflow of the water at junctions to inefficient canal capacity.



Link Flow Click a column header to sort the column.

Link	Type	Maximum |Flow| CMS	Day of Maximum Flow	Hour of Maximum Flow	Maximum |Velocity| m/sec	Max / Full Flow	Max / Full Depth
furnture	CONDUIT	1.510	0	00:13	4.31	0.99	1.00

Node Flooding Click a column header to sort the column.

Node	Hours Flooded	Maximum Rate CMS	Day of Maximum Flooding	Hour of Maximum Flooding	Total Flood Volume 10^6 ltr	Maximum Ponded Depth Meters
kochi	2.89	3.059	0	00:20	17.524	0.000

Figure 4.8 Drainage around Local Market

D. Drainage from Local Market to markos furniture

The maximum designed water level of drainage canal of junctions furniture were 0.7m. The flooding level attained with 3hrs rainfall intensity injunctions is 1.0m.Based on these thesimulation result of the model for the current land use condition,the designed canal depth was insufficient .Therefore there is the overflow of the water at junctions to inefficient canal capacity.

Link Flow — Click a column header to sort the column.

Link	Type	Maximum |Flow| CMS	Day of Maximum Flow	Hour of Maximum Flow	Maximum |Velocity| m/sec	Max / Full Flow	Max / Full Depth
zina	CONDUIT	0.506	0	00:10	1.45	1.00	1.00

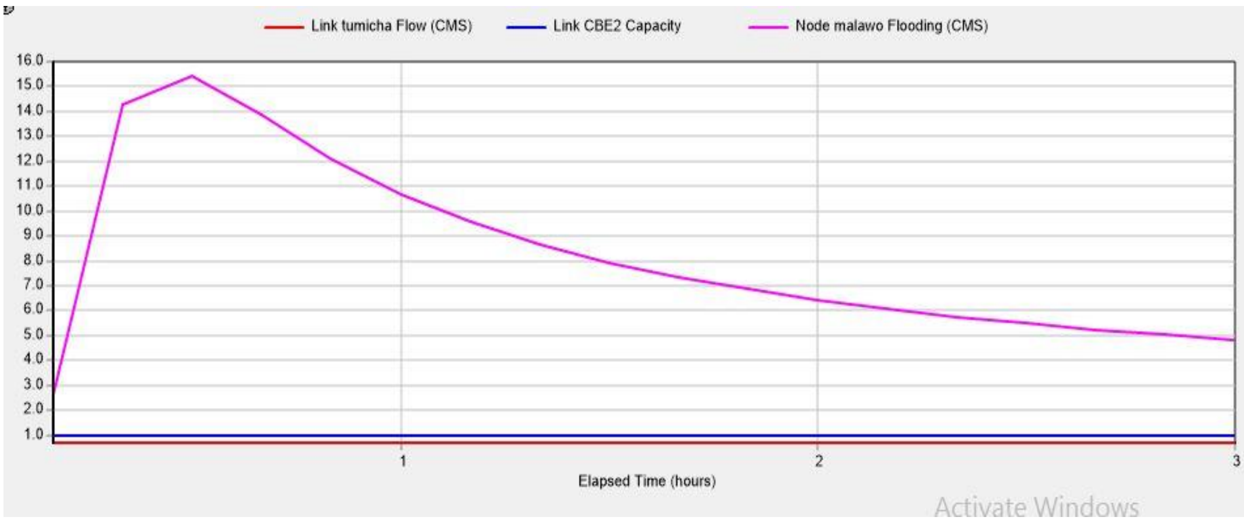
Node Depth — Click a column header to sort the column.

Node	Type	Average Depth Meters	Maximum Depth Meters	Maximum HGL Meters	Day of Maximum Depth	Hour of Maximum Depth
mosque	JUNCTION	0.61	0.70	6275.70	0	00:12

Figure 4.9 Drainage from Local Market to markos furniture

E. Drainage around Malawo

The maximum designed water level of drainage canal of juntions were CBE and Tumcha 1.0m3/s and 1.0m3/s. The flooding level attained with 3hrs rainfall intensity injunctions is 15.4m3/s. Based on these the simulation result of the model for the current land use condition,the designed canal depth was insufficient .Therefore there is the overflow of the water at junctions to inefficient canal capacity.



Link Flow | Click a column header to sort the column.

Link	Type	Maximum |Flow| CMS	Day of Maximum Flow	Hour of Maximum Flow	Maximum |Velocity| m/sec	Max / Full Flow	Max / Full Depth
CBE2	CONDUIT	1.316	0	00:06	3.13	1.00	1.00
tumicha	CONDUIT	0.695	0	00:08	1.98	0.98	1.00

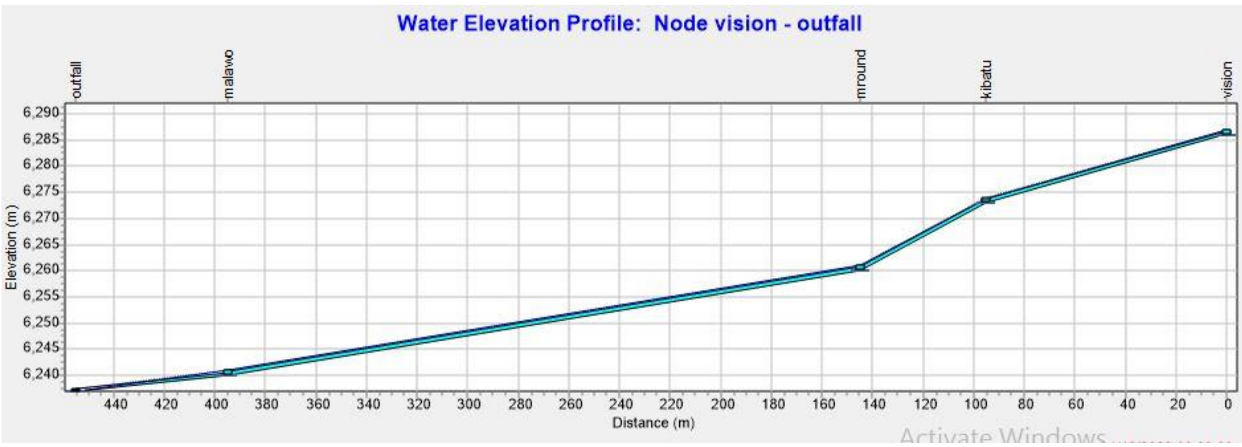
Node Flooding | Click a column header to sort the column.

Node	Hours Flooded	Maximum Rate CMS	Day of Maximum Flooding	Hour of Maximum Flooding	Total Flood Volume 10^6 ltr	Maximum Ponded Depth Meters
malawo	2.94	15.453	0	00:30	87.165	0.000

Figure 4.10 Drainage around Malawo

4.5.2. Water depth and flow in the links

Figure shows thatbthe flow with in the link and nodes around vision ,kibatu,mround and malowo.The simulation result indicates that junctions are flooded due to insufficient design water depth .The link is busy at this time and the over flow of runoff has happened. But outfall link(shell-tekle,markos-malawo)as the simulation result indicates the junctions are sufficient to carry the generated runoff and there is no flooded as shown fig .



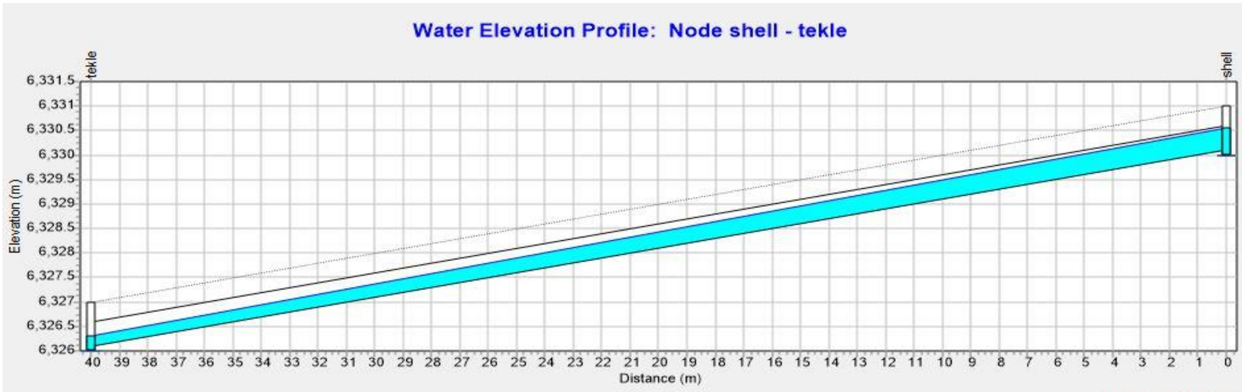
Summary Results

Outfall Loading — Click a column header to sort the column.

Outfall Node	Flow Freq. Pcnt.	Avg. Flow CMS	Max. Flow CMS	Total Volume 10^6 ltr
outfall	100.00	1.162	1.632	12.457

Figure 4.11 Water Elevation profile: node vision-outfall

From the figure 3.16 shows the water profile plot is obtained from node Vision-Outfall the maximum flow rate at ends is 1.632m3/s and total volume 12.457*10^3 . The water elevation profile is overflowed at Vision, Kibatu, Mround and malawo junctions. Therefore, water elevation profile is not sufficient hydraulic performance.



Summary Results

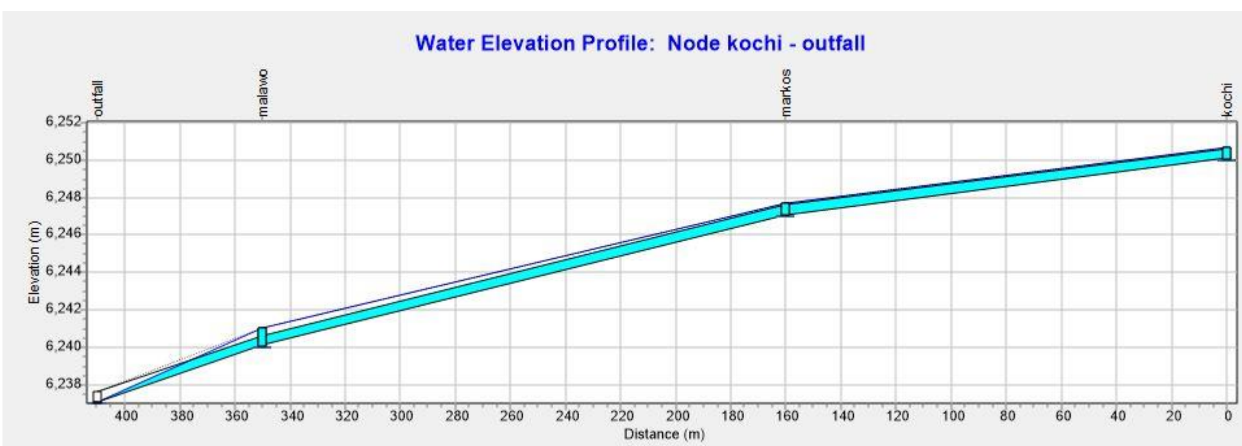
Outfall Loading

Click a column header to sort the column.

Outfall Node	Flow Freq. Pcnt.	Avg. Flow CMS	Max. Flow CMS	Total Volume 10^6 ltr
outfall	100.00	1.162	1.632	12.457

Figure 4. 12 Water Elevation profile:node Shell-tekle

From the figure 3.16 shows the water profile plot is obtained from node Shell- Tekle the maximum flow rate at ends is 1.632m3/s and total volume 12.457*10^3 . The water elevation profile is overflowed at Shell –Tekle node. Therefore, water elevation profile is sufficient hydraulic performance.



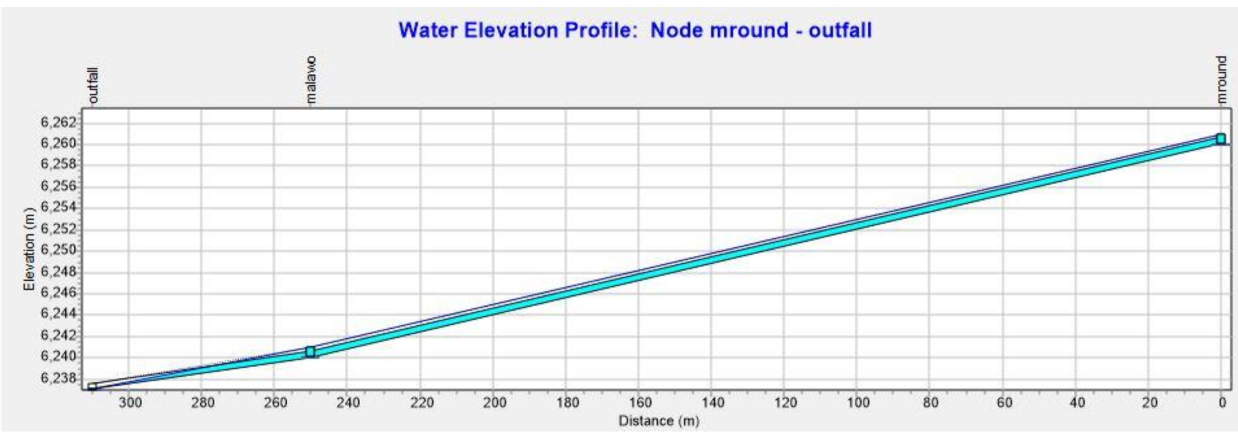
Summary Results

Outfall Loading — Click a column header to sort the column.

Outfall Node	Flow Freq. Pcnt.	Avg. Flow CMS	Max. Flow CMS	Total Volume 10^6 ltr
outfall	100.00	1.162	1.632	12.457

Figure 4.13 Water Elevation profile:node kochi-outfall

From the figure 3.16 shows the water profile plot is obtained from node Kochi-outfall the maximum flow rate at ends is 1.632m3/s and total volume 12.457*10^3 . The water elevation profile is overflowed at node Kochi-outfall junction. Therefore, water elevation profile is sufficient up to Markos node and not sufficient from Markos –Koch hydraulic performance



Summary Results

Outfall Loading — Click a column header to sort the column.

Outfall Node	Flow Freq. Pcnt.	Avg. Flow CMS	Max. Flow CMS	Total Volume 10^6 ltr
outfall	100.00	1.162	1.632	12.457

Figure 4.14 Water Elevation profile:node mround-outfall

From the figure 3.16 shows the water profile plot is obtained from node Mround –outfall the maximum flow rate at ends is 1.632m3/s and total volume 12.457*10^3 . The water elevation profile is sufficient hydraulic performance.

Table 4.8 Out flow discharge and volume of runoff outfall

Summary Results

Outfall Loading — Click a column header to sort the column.

Outfall Node	Flow Freq. Pcnt.	Avg. Flow CMS	Max. Flow CMS	Total Volume 10^6 ltr
outfall	100.00	1.162	1.632	12.457

Generally study simulated result intable are one outlet the total simulation area of subcatchment is 44ha .the drainage system are 22 nodes and 24 subcatchment outlet result in table showed that 1.162m^3/s average flow rate,1.632 m^3/s maximum flow rate and 12.457*10^3 m^3.

The same study using swmm 5.1 model located in Athens, Greece, and covers on area of 89ha most of the drainage area is closely developed . The portion of drainage network to the catchment consisted of 79 pipes and 112 junctions.(Eyosias B,2018)

4.5.3. Sub Catchments runoff

The figure 4. Shows total simulation area of sub catchment is 44ha .the drainage system are 22 nodes and 22 subcatchment.outlet result in table showed that 1.162m^3/s average flow rate,1.632 m^3/s maximum flow rate and 12.457*10^3 m^3 total volume.

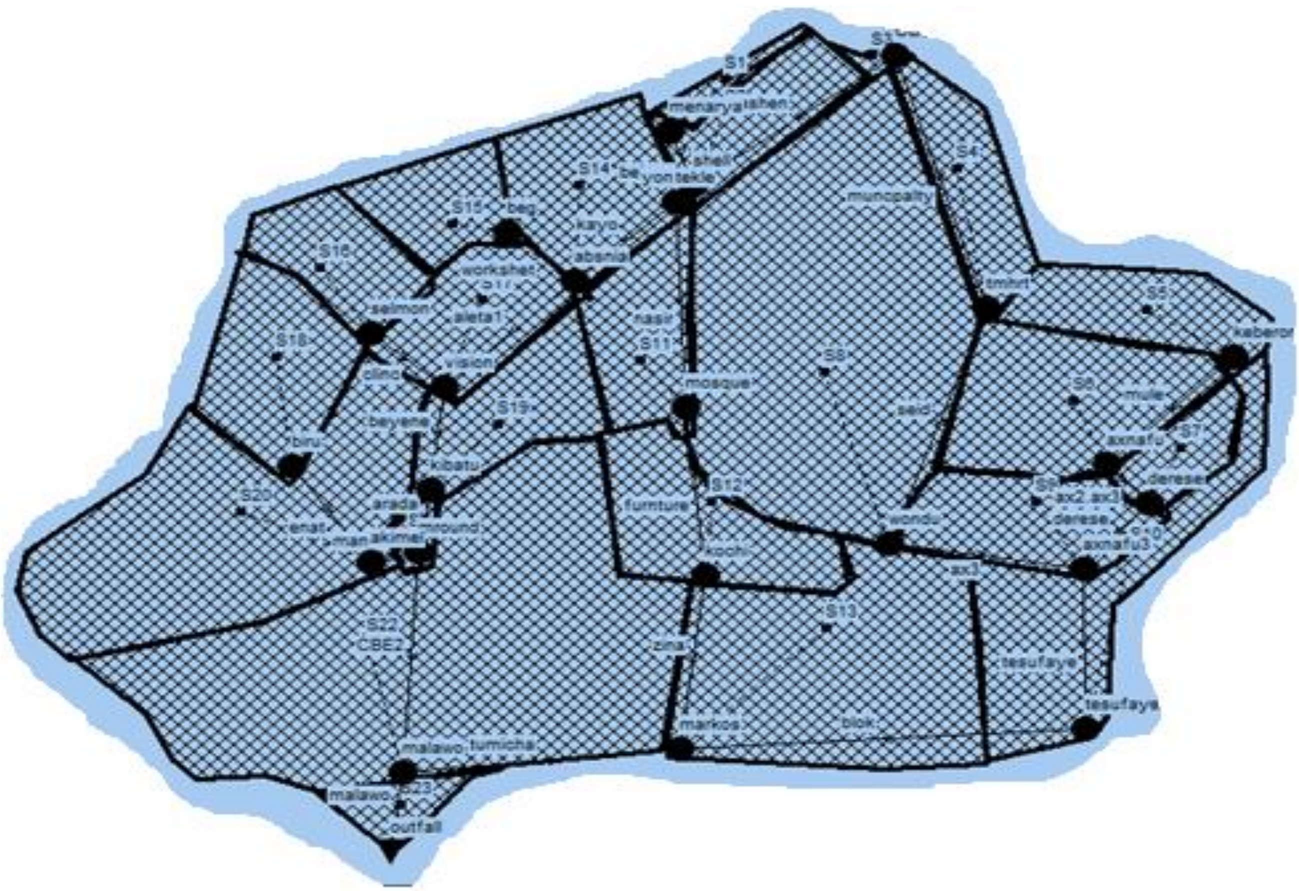


Figure 4.15 outfall run off simulated maps

4.5.4. Model calibration and validation

For validation

Table 4.9 Hydraulic capacity for validation

Channel type	Record depth (m)	Manning roughness used(n)	channel slope (S)	Width (m)	Area (m^2)	Parameter (m)	Hydraulic radius (m)	Velocity (m/s)	Discharge Q=A*V (m^3/s)
Rectangular	0.6	0.013	0.012	0.5	0.3	1.7	0.176	2.64	0.79
Rectangula	0.51	0.013	0.023	0.7	0.357	1.7	0.206	4.06	1.421
Rectangula	0.5	0.013	0.027	0.7	0.35	1.7	0.236	4.01	1.403
Rectangula	0.53	0.013	0.024	0.7	0.371	1.7	0.206	4.50	1.57
Rectangula	0.5	0.013	0.027	0.7	0.35	1.7	0.206	4.06	1.42

In order to evaluate the capacities of the drainage system the current runoff has to be estimated by using SWMM Model. Therefore, this part was prepared to briefly the hydrologic investigation and evaluation of the existing drainage structures drainages of the selected roots which includes road starts from Zerabruk Hotel to Malewo, Road to elementary school from malewo and road starts from Keberon church to Malewo and other minor drainage lines for minor roads. Most of minor roads were cobblestone surfaced and the major ones were Bitumen Asphalt road. obtained sample result were presented in table 14. The rest of the result is given in the APPENDIX .

Table 4. 10 Model result for Channel

No	Flow Direction		Channel Type	Flow(m^3/s)
	From	to		
1	CBE	Shell	Rectangular	0.435
2	Shell	Tekle Hotel	Rectangula	1.227
3	Tekle Hotel	mosque	Rectangula	0.660
4	Mosque	kochi	Rectangula	1.510
5	kochi	markos	Rectangula	0.506
6	markos	Malawo	Rectangular	0.695

4.5.5. Comparison of hydraulic capacity and model runoff

The hydraulic capacities of the open channels in the study area were determined using the flow master. Accordingly, the peak rate of runoff and hydraulic capacities of the channel constructed were computed and the obtained sample result were presented in table 15.

Table 4.11 Comparison of hydraulic capacity and model runoff

NO (1)	FLOW DIRECTION		CHANNEL TYPE (4)	Model flow $Q(M^3/s)$ (5)	Hydraulic Capacities $Q(M^3/s)$ (6)
	FROM (2)	TO (3)			
1	CBE	Shell	Rectangular	0.75	1.02
2	Shell	Tekle Hotel	Rectangula	3.8	1.42
3	Tekle Hotel	mosque	Rectangula	2.8	1.54
4	Mosque	Local Market	Rectangula	2.06	1.57
5	Local Market	Markos furniture	Rectangula	5.8	1.42
6	Markos furniture	Tumicha Hotel	Rectangula	4.18	1.558
7	Tumicha Hotel	Malawo	Rectangular	0.75	1.558

As some of the existing drainage systems are overtopped during heavy rainfalls. The road side drains, walkways and the asphalt are flooded sometimes even during average storm. As it can be seen from table 7, all the channels) are not sufficient to carry the runoff water contributed to them with regard to their hydraulic property. This show that the hydraulic capacity of the channels is insufficient to carry runoff generated based on 10 years of return period design discharge. The main problem of this area is in sufficient capacity of the channel and inlet sizes of the culverts, which did not design based on the contributing catchment area. The contributing catchment area is 44ha. To overcome this problem resizing the drainage system for both inlets and channels are mandatory. Also changing the channels section is an alternative way to reduce the flooding problem.

4.5.6. Runoff comparison SWMM 5.1 and Rational Method

The evaluation of hydrological model behavior and performance is commonly made and reported through comparison of simulated and observed variables. Frequently comparison are made between simulated and measured stream flow at the catchment outlet .in this study the model

SWMM was compared with calculated flow using rational method . This shows that model simulated by model is considered real time computation.

Table 4. 12 Comparison SWMM 5.1 and Rational Method

Sub-catchment	Area(ha)	Runoff of coefficient	Time of concentration	Model Discharge(m3/s)	Discharge(m3/s) By rational method
S1	0.554117	0.954	10	0.75	0.11
S2	1.55322	0.946	12	3.8	0.44
S3	1.733458	0.954	5	2.8	0.12
S4	1.227953	0.950	22	2.06	0.12
S5	0.596762	0.950	19	5.8	0.15
S6	1.406045	0.944	14	4.18	0.42

4.6. Basic Problem of the Aleta Wondo Town drainage system

Apart from significant flood regime change, field visits and survey reveals that there are different Problems which makes the process of disposing runoff in to water ways made difficulties in this area.

- Dumping of solid and liquid wastes in to storm water drainages

Dumping of garbage, particularly plastics, causes serious reduction in waterways of main drainage channels. In Aleta wondo space for construction of roadside drains is a major problem. Even 60% to 70% of the houses which do not have a sewer connection or a septic tank are discharging their toilet waste into the existing drains, causing serious environmental problems. Dumping solid waste materials in to drainages and streams is the other challenge of storm water drainage system. Urban litter (alternatively called trash, debris, junk, floatables, gross pollutants, rubbish or solid waste) has become a major problem in these areas. Typically, it consists of manufactured materials such as bottles, cans, plastic and paper wrappings, newspapers, shopping bags, cigarette packets and remains of chat. As a result of damping these solid wastes in to drains the drainage system has been clogged and causes flooding over streets and walk ways.

Figure 4. 16 Dumping of solid and liquid wastes in to storm water drainages

- Lack of frequent clearance of drainage system.

The construction debris in the new drains is not cleared before commissioning and garbage is dumped on the road sides which easily find their way into the adjacent drains. Sediment load, solid wastes blocked most of the drainage system. So without scheduled clearance the service life of those ditches could be out of their life span Due to lack of frequent clearance of drainage lines they have become out of services.

Figure 4. 17 shows blocked ditches in the study area

- Demographic Conditions of Town

Due to the faster growth of population and rapid increase in the land prices habitation has extended to the low lying areas which do not have proper drainage outlets.

- The Topographic Conditions of Town

Due to hilly terrain with sufficient gradients to drain off water easily in to the two main natural drainage channel i.e. river Malawo and Tercha, The Town has a steep slope. Aleta Wondo topography is a significant factor for the drainage designer.

- Drainage inlet design

The hydraulic capacity of a storm drain inlet depends upon its geometry as well as the characteristics of the gutter flow. Inlet capacity governs both the rate of water removal from the gutter and the amount of water that can enter the storm drainage system. Inadequate inlet capacity or poor inlet location may cause flooding on the road resulting in a hazard to the travelling public.

4.7. Proposed best management practice (BMP)

This section provides proposed best management practices to solve the problem of urban storm water drainage that has been hindering the drainage systems. Typically, there is not a single answer to the question of which BMP (or BMPs) should be selected for a site. There are usually multiple solutions ranging from standalone BMPs to treatment trains that combine multiple

BMPs to achieve the water quality objectives. In order to select specific BMP for the area which has a problem, there are factors that were considered based on the Urban Storm Drainage Criteria Manual Volume 3, Best Management Practices, 2011. In this study one sample place from the study area is selected to propose these BMP, the selection was based on the fact that this place has been damaged by runoff during the rainy season and contribute most of its flow to malawo.

- Solid & Liquid waste Management practice

The generations of solid wastes in Aleta Wondo town are mostly generated from residential houses and commercial and other institution. Households in the city administration store the garbage in plastic bags known as 'Madaberia' and put it in front of their houses. In Aleta Wondo town there are no collection points or transfer stations organized by the city administration, but only dust bins placed along the roads and residential areas. Neither Micro & small enterprises nor any other privet businesses are made to participate in the collection process of solid wastes. Solid wastes in the city of Aleta Wondo are transported to disposal site by use of donkey carts and 3 wheel Bajaj Motors daily without being stored at the source.The solid wastes in Aleta Wondo town are dumped very near to the town, which is only 0.8 kilo meter away from the centre of the town. The disposal site is located with a river add many people use river water. Legal house are also built near the damping site, and the people living around this area said that they are suffering from bad smell all the time. Solid waste disposal mechanism of Aleta Wondo city is mainly fenced dumping site. There is no enough dumpster in the city and waste is mainly transported using 2 three wheel Bajaj truck. On the average the cities collected and disposed close to 50% of the estimated waste generated. The city has inadequate capacity in terms of staffing as the total number of staff per 1000 population is just 0.5.

- Selecting BMP for study area

In order to select the best management practice that could fit to this area Urban Storm Drainage Criteria Manual Volume 3 was used (UD and FC, 2011). The following factors have been considered to decide the best suited practice.

✓ Physical site characteristics

A/ Topography of this area is moderately steep

B/ contributing drainage area

C / This place is surrounded by pavement urban area

- Types of best management practice fit for

Since this area has not been polluted with sewages from the surrounding environment, those best management practices with chemical and biological treatment were screened and the only source of pollution in this case is sediment. Based on the manual, for this specific site the following best management practices could fit.

a) Grass Swale, or i

b) Grass Buffer Channel

- Criteria For the best management practice

a.Aesthetic

aesthetically both of them could be qualified, however the area of this place quite small. So in order to implement grass buffer enough space is necessary. Since the area for the selected site isvery small, this leads us to use Grass Swale channel.

b. Safety

By safety means, if the BMP implemented, what would be the worst case scenario caused by this management practice regarding to the safety of the environment. So both practice could have worked out better than other BMPs regarding to safety of the environment.

c. Maintenance requirement

Grass buffer frequently damaged by vehicles when they are adjacent to roadways and unprotected which requires frequent maintenance. However, in Grass swales removal of sediment and associated constituents through filtering (straining) is simpler than in the other BMPs. So based on this factor Grass swale is best suited BMP.

5. CONCLUSIONS AND RECOMMENDATIONS

5.1. CONCLUSIONS

In this study the existing drainage of Aleta Wondo Town has been evaluated with the help of Orthophoto ,Structural plan ,Tape Meter,Google Earth including GIS and SWMM Model to analyze the data. The results from this study shows that the drainage system.From the results and discution we can conclude the following points

- The main drainage problems in the existing system is the capacity of drainage system can't handle the current runoff that flows over the area, this is due to urbanization and lack of assessment of the hydrology of the area.
- Solid and liquid waste disposal are the main contribution for the clogging of the drainage canals and community awareness was quite poor, even those who have the awareness over helmed to dump wastes over the drainage system because there were lack of mechanisms to dispose and take care of wastes.
- The maximum flood occur at location node vision to outfall, node mround to out fall, are the most flooded exposed junctions and links.
- Generally, the performance of these urban drainage were not satisfactory. Therefore, it is recognized that its capacity has shown lower which needs some adjustment or improvement to give the best service and needs a series of regular maintenance and also provide drainage networking for the areas without drainage systems for its complete service.

5.2. RECOMMENDATIONS

Based on the result and discussion drainage structure of Aleta Wondo Town were inadequacy and have had serious negative impact on the community and road structures. In order to alleviate the problems that has been hindering the drainage systems in this study area, the following recommendations are made for better and sustainable urban drainage system.

- Adaptation of different Best Management Practices (BMP) which will not only reduce the peak runoff but also have aesthetic values and improve the environment. Even though urbanization cannot be avoided, the runoff that can be generated due to impervious areas can be infiltrated in to the ground through different best management practices that is supported by strong institutional setup, policy framework, and the public at large.

- Create awereness with in the community to use the drainage system in away that the drainage system could be able to serve as the life span and the community should also know how to manage solid and liquid wastes.
- The current drainage network line totally over flows so we recommend that the removing the silt from the channel,reconstructed the failed infrastructures and the existing drainage system must be properly controlled management.

REFERENCES

Armitage, N. (2010) The Challenges of Sustainable Urban Planning in Developing Countries Urban Water Management Group, University of Cape Town, South Africa.

Balmforth, D., Digman, C., Butler, D. Ziervogel & Shaffer, P., 2009. Research Framework -The implementation of Integrated Urban Drainage, Bristol: Environmental Agency.

Belete, D.A. (2011) Road and Urban Storm Water Drainage Network Integration in Addis David Butler and John W.Davies, 2000. Urban drainage London second edition.1pp.

Burian and Edwards, (2002). Technical challenge to drain the urban area expeditiously to include. the consideration of social, economic, political, environmental, and regulatory factors.

Elliott, A.H.; Trowsdale, S.A. A review of models for low impact urban stormwater drainage.

Environ. Model. Softw. 2007, 22, 394–405.

Ethiopian Roads Authority. (2011), Drainage Design Manual. Addis Ababa.

Ethiopian Roads Authority. (2013). "Drainage Design Manual", Federal Democratic Republic of Ethiopia.

Federal Highway Administration, U.S.Government printing office,(1995).Hydraulic Design series No.4.Design of Roadside Drainage Channels.

Federal Urban Planning Coordinating Bureau (FUPCOB).2008. Urban Storm Water Drainage Design Manual. Addis Ababa v-1pp.

Jimoh, (2003). Inadequate drainage channels easily translate into environmental deterioration.

Johnson, R.C., Imhoff, J.C., Kittle, J.L. Jr., and Donigian, A.S. Jr., 1984. “Hydrological Simulation Program- FORTRAN (HSPF): User’s Manual for Release 8.0”.

Kalantari, Z (2011) “Adaptation of Road Drainage Structures to Climate Change” TRITA LWR LIC 2061.

Kirubel W., 2014. Investigation on Storm Drainage System Problem of Hawassa City.

Lewis A. Rossman and Wayne C. Huber. 2016. Storm Water Management Model Reference Manual Volume. III Water Quality. US 3pp.

Lewis A. Rossman, 2004. Stormwater management Model User's Manual (version 5.1).

Maidment, 1993. Hand Book of Hydrology, McGraw-Hill. Inc. USA Rao, K.H, & Hamed .Flood frequency analysis.

Meraf Elias Beshah .2015. Highway drainage facilities Problems. Adama.

MoWUD, August 2008 Addis Ababa, Urban Storm Water Drainage Design Manual, page-1.

Mukherjee, D. (2014) Highway Surface Drainage System & Problems of Water Logging In Road Section, the International Journal of Engineering and Science, Volume 3, Issue 11.

Mulualem Bekele Nora and K.Naga Sahadeva Reddy. 2016. Performance Assessment of Road Drainage Systems Burayu Vol. 2 No. 2. Pp. 40-55.

O'Loughlin, G., 1993. "The ILSAX Program for Urban Storm Water Drainage and Analysis", User's Manual for Microcomputer, Version 2.13.

Saara and Saarenketo, (2006).Design and repair of roads suffering spring thaw weakening, European.

SUDS: Background, 2005. Adapting Sustainable Urban Drainage Systems to Storm water Management in an Informal Setting.

Surveying and Engineering Software, 1997. CIVILCAD, User Manual, Volume 2, Chapter 8.

Systems", Australian Water Resources Council, Technical Paper No.10.

White, M.J. and Cattell, F.C.R., 1992, "AUS-QUAL, an Australian Water Quality Management

WP Software, 1991. "RAFTS-XP (Runoff Analysis and Flow Training Simulation)", Version 2.7, User Manual, Australia.

WP Software, 1992. "RatHGL-XP", User Manual, Australia.

Yilma, S., (2005, et al). Engineering Hydrology. Addis Ababa.

Zoppou, C. Review of urban stormwater models. Environ. Model. Softw. 2001, 16, 195–231

APPENDICESS

Table 1.:max rainfall data from (1991-2019) and its statistical calculation

			Gumbel EV-1	CORRECTED					
station	year		annual daily maxim	RANK(m)	P=1/T or Rank/	Return Period,(T)=1/p	YT=-ln(ln(T/(T-1)))		XT=u+a*YT
aleta wondo	1991		26	29	0.97	1.03	-1.224127541		16.07412
aleta wondo	1992		58	12	0.40	2.50	0.671726992		55.92347
aleta wondo	1993		50.2	16	0.53	1.88	0.271624945		47.51364
aleta wondo	1994		37	25	0.83	1.20	-0.583198081		29.54594
aleta wondo	1995		50	17	0.57	1.76	0.17883003		45.56317
aleta wondo	1996		60	10	0.33	3.00	0.902720456		60.77877
aleta wondo	1997		50	18	0.60	1.67	0.087421572		43.64183
aleta wondo	1998		59.5	11	0.37	2.73	0.783600688		58.27496
aleta wondo	1999		102	1	0.03	30.00	3.384294493		112.9395
aleta wondo	2000		43.2	23	0.77	1.30	-0.375203292		33.91783
aleta wondo	2001		70	2	0.07	15.00	2.673752092		98.00444
aleta wondo	2002		65	8	0.27	3.75	1.170683338		66.41113
aleta wondo	2003		58	13	0.43	2.31	0.565661963		53.69406
aleta wondo	2004		56.6	14	0.47	2.14	0.464246379		51.56239
aleta wondo	2005		45.2	21	0.70	1.43	-0.185626759		37.90257
aleta wondo	2006		44.5	22	0.73	1.36	-0.278961034		35.94076
aleta wondo	2007		41	24	0.80	1.25	-0.475884995		31.80158
aleta wondo	2008		45.6	20	0.67	1.50	-0.094047828		39.82749
aleta wondo	2009		66.3	5	0.17	6.00	1.701983355		77.57863
aleta wondo	2010		33.2	28	0.93	1.07	-0.996228893		20.86436
aleta wondo	2011		36.8	26	0.87	1.15	-0.700571065		27.07886
aleta wondo	2012		66.3	6	0.20	5.00	1.499939987		73.33184
aleta wondo	2013		60.1	9	0.30	3.33	1.030930433		63.47364
aleta wondo	2014		65.2	7	0.23	4.29	1.325375512		69.66264
aleta wondo	2015		36.2	27	0.90	1.11	-0.834032445		24.2736
aleta wondo	2016		48.2	19	0.63	1.58	-0.003296669		41.73501
aleta wondo	2017		69.7	4	0.13	7.50	1.944205697		82.66996
aleta wondo	2018		69.7	3	0.10	10.00	2.250367327		89.10523
aleta wondo	2019		50.6	15	0.50	2.00	0.366512921		49.50811
		mean	53.93						
		n	29			T	YT=-ln(ln(T/(T-1)))		XT=u+a*YT
		standerd de	15.21			2	0.366512921		49.5
		a	21.0192031			5	1.499939987		73.3
		u	41.80430065			10	2.250367327		89.1
						25	3.198534261		109.0
						50	3.901938658		123.8
						100	4.600149227		138.5
						200	5.295812143		153.1
						1000	6.907255071		187.0

Table 2: kn value for different sample size (used for outlier)

S/ size (n)	kn	S/ size (n)	kn	S/ size (n)	kn	S/ size (n)	kn
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	2.7	80	2.94
15	2.247	29	2.549	43	2.71	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
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	2.76	130	3.104
22	2.429	36	2.639	50	2.768	140	3.129 ,
23	2.448	37	2.65	55	2.804		

Source: (Chow, 1988)

Table 3.Computation of the capacity of the existing drainage system

Sub catchment code	Geometer	Material	b(m)	Y(m)	A(m^2)	n	Slope (%)	P(m)	R(m)	V(m)	Manning Q(m^3/s)
S1	Rectangular	masonery	0.45	0.5	0.225	0.013	1.5	1.45	0.155	2.82	0.612
S2	Rectangular	masonery	0.5	0.7	0.35	0.013	2.5	1.9	1.18	13.5	4.75
S3	Rectangular	masonery	0.6	0.7	0.42	0.013	2.5	2	0.21	4.3	1.8
S4	Rectangular	masonery	0.5	0.6	0.3	0.013	2.5	1.7	0.17	3.73	1.12
S5	Rectangular	masonery	0.5	0.6	0.3	0.013	1	1.7	0.17	0.74	0.22
S6	Rectangular	masonery	0.5	0.6	0.3	0.013	1.5	1.7	0.17	2.89	0.86
S7	Rectangular	masonery	0.5	0.5	0.25	0.013	1.5	1.5	0.16	2.88	0.72
S8	Rectangular	masonery	0.5	0.5	0.25	0.013	1.5	1.5	0.16	2.88	0.72
S9	Rectangular	masonery	0.5	0.6	0.3	0.013	1.5	1.7	0.17	3.73	1.12
S10	Rectangular	masonery	0.5	0.5	0.25	0.013	2.5	1.5	0.16	3.58	0.89
S11	Rectangular	masonery	0.5	0.7	0.35	0.013	2	1.7	0.20	1.17	0.41
S12	Rectangular	masonery	0.5	0.7	0.35	0.013	1.5	1.9	0.18	3	1.05
S13	Rectangular	masonery	0.5	0.5	0.25	0.013	1.5	1.5	0.16	2.88	0.72
S14	Rectangular	masonery	0.6	0.7	0.42	0.013	2	2	0.21	1.21	0.51
S15	Rectangular	masonery	0.5	0.5	0.25	0.013	2.5	1.5	0.16	3.58	0.89
S16	Rectangular	masonery	0.5	0.5	0.25	0.013	2.5	1.5	0.16	3.58	0.89
S17	Rectangular	masonery	0.5	0.7	0.35	0.013	1	1.7	0.20	1.17	0.41
S18	Rectangular	masonery	0.5	0.5	0.25	0.013	2.5	1.5	0.16	3.58	0.89
S19	Rectangular	masonery	0.6	0.7	0.42	0.013	1.5	2	0.21	3.33	1.39
S20	Rectangular	masonery	0.6	0.6	0.36	0.013	1.5	1.8	0.2	3.22	1.16
S21	Rectangular	masonery	0.55	0.5	0.275	0.013	1.5	1.55	0.17	2.89	0.79
S22	Rectangular	masonery	0.53	0.55	0.291	0.013	2	1.63	0.17	1.05	0.30

APPENDIX4 ; Model result for channels

No	Flow direction		Section	Flow (m^3/s)
	From	To		
1	CBE	Shell	Rectangular	0.435
2	Shell	Tekle	Rectangular	1.227
3	Tekle	Absnia	Rectangular	0.567
4	Menaria	Shell	Rectangular	0.747
5	begi	absnia	Rectangular	1.793
6	absnia	Vision	Rectangular	1.779
7	vision	kibatu	Rectangular	1.747
8	kibatu	mround	Rectangular	2.373
9	selemon	vision	Rectangular	1.916
10	biru	akimel	Rectangular	1.787
11	mande	mround	Rectangular	1.140
12	mround	malawo	Rectangular	1.316
13	Tekle	mosque	Rectangular	0.660
14	mosque	kochi	Rectangular	1.510
15	kochi	markos	Rectangular	0.506
16	markos	Malawo	Rectangular	0.695
17	CBE	tmhrt	Rectangular	0.312
18	tmhrt	wondu	Rectangular	2.554
19	keberon	axnafu	Rectangular	1.919
20	axnafu	derese	Rectangular	1.281
21	derese	axnafu	Rectangular	1.103
22	wondu	axnafu	Rectangular	0.740
23	axnafu	Tesufaye	Rectangular	1.055
24	Tesufaye	markos	Rectangular	0.974

Appendix5 ; Simulated catchment characterstics result

Outflow element	Area	Runoff coefficient	Flow(m^3/s)
S1	0.554117	0.954	0.75
S2	1.55322	0.946	4.18
S3	1.733458	0.954	0.75
S4	1.227953	0.950	2.73
S5	0.596762	0.950	2.73
S6	1.406045	0.944	5.06
S7	1.174604	0.954	0.97
S8	1.366258	0.923	13.77
S9	1.528665	0.950	2.53
S10	0.832977	0.947	3.87
S11	0.885565	0.950	2.8
S12	0.896924	0.951	2.06
S13	0.492805	0.942	5.80
S14	0.525443	0.947	3.80
S15	0.09441	0.950	2.50
S16	0.127712	0.950	2.50
S17	1.098351	0.950	2.66
S18	0.428378	0.950	2.76
S19	0.695377	0.950	2.77
S20	1.091549	0.942	5.86
S21	3.255199	0.954	0.75
S22	1.098745	0.922	14.33
S23	0.880407	0.954	0.75

Appendix:Estimated drainage capacity and Existing drainage capacity

area (m2)	C	H1	H2	L(m)	H1(m)	H2(m)	S	v(m/s)	h1-h2(m)	Tt(hr)	Tci(hr)	TC(min)	I10	I25	Q10	Q25
0.3211	0.8	6359	6336	200	1938	1931	0.035	2.82	7	0.020	0.151	10	150	167.3	0.11	0.12
1.874	0.7	6366	6324	250	1940	1928	0.051	13.5	13	0.005	0.199	12	120	130	0.44	0.47
0.3211	0.8	6366	6362	50	1940	1939	0.024	4.3	1	0.003	0.085	5	170.2	183.7	0.12	0.13
0.866	0.5	6366	6341	250	1940	1933	0.030	3.73	8	0.019	0.354	22	97	106	0.12	0.13
0.866	0.6	6341	6316	200	1933	1925	0.038	0.74	8	0.075	0.245	19	103	125.1	0.15	0.18
1.82	0.	63	62	25	192	191	0.0	2.89	12	0.0	0.20	14	119	137	0.	0.

3	7	16	77	0	5	3	48			24	4				42	49
0.525	0.5	6316	6258	50	1925	1907	0.354	2.88	18	0.005	0.070	5	170.2	183.7	0.12	0.13
5.748	0.7	6341	6276	400	1933	1913	0.050	2.88	20	0.039	0.254	18	104	126	1.16	1.41
1.32	0.5	6292	6278	100	1918	1914	0.043	3.73	4	0.007	0.200	12	120	130	0.22	0.24
0.035	0.85	6278	6265	250	1914	1910	0.016	3.58	4	0.019	0.183	12	120	130	0.01	0.01
0.896	0.7	6280	6275	200	1914	1913	0.008	1.17	2	0.047	0.333	23	96	105	0.17	0.18
1.656	0.7	6326	6275	250	1928	1913	0.062	3	16	0.023	0.186	13	119.6	137.6	0.39	0.44
3.76	0.5	6275	6250	200	1913	1905	0.038	2.88	8	0.019	0.294	19	103	125	0.54	0.65
1.366	0.7	6250	6247	210	1905	1904	0.004	1.21	1	0.048	0.411	28	97.5	127	0.26	0.34
0.764	0.7	6324	6305	250	1928	1922	0.023	3.58	6	0.019	0.258	17	105.1	126.1	0.16	0.19
0.764	0.7	6309	6286	250	1923	1916	0.028	3.58	7	0.019	0.242	16	106.7	126.7	0.16	0.19
0.832	0.7	6309	6286	180	1923	1916	0.039	1.17	7	0.043	0.184	14	119	137	0.19	0.22
0.88	0.7	6316	6309	200	1925	1923	0.011	3.58	2	0.016	0.298	19	103	125	0.18	0.21
0.885	0.7	6305	6273	300	1922	1912	0.033	3.33	10	0.025	0.253	17	105.1	126.1	0.18	0.22
2.44	0.7	6292	6264	400	1918	1909	0.021	3.22	9	0.035	0.335	22	97	106	0.46	0.50
1.098	0.7	6273	6260	50	1912	1908	0.079	2.89	4	0.005	0.077	5	170.2	183.7	0.36	0.39
6.118	0.7	6260	6240	350	1908	1902	0.017	1.05	6	0.093	0.335	26	96.3	129.4	1.15	1.54

Table : 24hr Rain fall depth Vs frequency

Return period years	2	5	10	25	50	100
RR-A1	50.30	66.02	76.28	89.13	98.63	108.06
RR-A2	51.92	65.52	74.45	85.70	94.07	102.48
RR-A3	47.54	59.61	67.66	77.92	85.62	93.34
RR-A4	50.39	63.83	72.28	82.55	89.97	97.20
RR-B1	58.87	71.26	79.29	89.35	96.84	104.37
RR-B2	55.26	69.95	79.68	92.03	101.29	110.61
RR-C	56.52	71.04	80.54	92.52	101.48	110.50
RR-D	56.23	76.84	90.37	107.46	120.23	133.05

Table: Annual average cumulative of different stations

cumm aleta wondo	cum yirgalem	cumm aposto	cumm kabado	cumm Teferkella	cumm aleta	avg 4 near stations
41.1	77.5	23.1	22.5	41.3	41.1	41
87.5	108.1	84.3	74	83.6	87.5	89
144.9	179	125	169.2	106.2	144.9	158
194.2	235.4	189.4	214	137.9	194.2	213
237.3	291	221	256.4	180.6	237.3	256
292.2	338.7	276.8	332.4	220.7	292.2	316
338.0	373.5	343	388.6	246.8	338.0	368
384.5	419	381.7	454.3	283.1	384.5	418
424.7	447	418.7	519.7	313.2	424.7	462
463.3	474.3	463.3	563.2	352.4	463.3	500
509.1	526.4	516.7	604.5	388.8	509.1	549
547.4	553.5	553.3	644.7	438.2	547.4	584
590.9	580	602.6	700.3	480.5	590.9	628
627.4	608	641.6	733.6	526.2	627.4	661
670.4	642.1	688.3	776.6	574.5	670.4	702
717.7	682.4	750.4	809	629	717.7	747
771.9	731.2	823.9	853.6	679	771.9	803
824.2	780	879.2	916.6	721	824.2	859
871.4	816.8	928.6	979	761.1	871.4	908
923.5	855.4	995	1035	808.6	923.5	962
991.3	894.2	1040.6	1170.4	860.1	991.3	1035
1027.8	923	1082.5	1208.4	897.1	1027.8	1071
1070.8	943.6	1153.6	1255.2	930.6	1070.8	1117
1112.4	972.2	1200.4	1310.2	966.6	1112.4	1161
1152.7	1004.8	1258	1341.2	1006.8	1152.7	1201
1197.7	1051.4	1293.9	1396.3	1049	1197.7	1247
1234.4	1079.8	1330.4	1433.8	1093.6	1234.4	1281
1290.8	1112.4	1393.9	1498.8	1158.1	1290.8	1335
1345.0	1149.2	1445	1578.3	1207.4	1345.0	1391