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ADAMA FLOOD PROBLEM –FLOOD RISK MITIGATION OPTION FOR BOKU SHENEN AREA

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Adama Flood Problem

Flood Risk Mitigation Option for Boku Shenen Area

By

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Abstract

Flooding is becoming a usual event that occurs in urban and rural areas. It is an interaction between mankind and nature. A city like Adama is nowadays becoming a flood prone area. A flood that comes from the adjacent watershed area seriously affects the low lying areas as seen at Adama – Boku shenen Kebele.

Boku Shenen kebele is one of a flood prone area of Adama city every year. Four different watershed catchments that produce flood exist around the kebele. Because of lack of integrated flood risk management system; loss of lives, displacement of people, and destruction of social infrastructures are becoming increasing from time to time.

Adama has been vulnerable to flash flooding from rainfall, in particular of the ungagged Kersa and Golba Tegene catchments which pass through South East of the city.

It has not been tried to estimate the amount of flood and not recommended mitigation options based on the natural situation of the area previously.

Hence, this thesis is to identify peak flood and recommend mitigation options based on the environmental and geological situation of the area that can be affected by extraordinary floods.

This thesis tries to consider more options and fills the gaps not covered by others adopting more than four application soft wares like Easy fit, ArcGIS, Global Mapper and AutoCAD. Specially, peak flood discharge estimation and recommend mitigation measures selection method believed to be the gap not properly covered in the previous flood risk mitigation works. The data usage for this thesis tried to make very intensive by considering different data options like gridded (DEM and contour), digitized (soil type, land use and 1:50,000 scale map) and rainfall data.

For precipitation modelling, ERA Intensity-frequency-duration curve was used for frequency storm and for the gage weights annual maximum daily rainfall for 24 hours and 6 hours duration storm are used since flood estimation requires a properly recorded data more than 30years from metrological stations. For the study area, I just used a rain fall data of 62years of record from Adama meteorological station.

30m x 30m resolution DEM for catchment was used to delineate the watershed area using GIS.

In this thesis, the flood magnitude estimated for a 100years return period was computed by using SCS excel model method for Kersa and Golba tegene catchments and flood mitigation option selected to be detention pond incorporating the designing criteria.

Detention pond/dam was designed for both catchments. For Golba Tegene catchment, after analyzing the area of the reservoir/pond, having 8m of dam height 130962 m^3 volume of flood can be absorbed/held which a 25 years design flood is. The spillway was designed for 100years return flood and can spill maximum of 5.586m^3/s routed discharge. For Kersa catchment, a controlled local pond type of 3m height at 8584m^2 and hence 26421m^3 volume flood could be held which is also a 25years design flood. It also has spillway and controlled outlet gate that can discharge a maximum of 0.78m^3/s. After the junction point of the two gullies a maximum of 6.36m^3/s discharge could pass through the existing gully. However the existing gully was modified so as to accommodate safely the maximum 6.36m^3/s discharge.

In doing so, this thesis will help and become an input in flood risk mitigation process. It will benefit the study area community directly where as any others can be benefited indirectly.

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ABBREVIATIONS

AAD	Average Annual Damage
ACA	Adama City Administrative
ACAEPO	Adama City Administrative Environmental Protection Office
AMC	Antecedent Moisture content
CA	City Administration
CBA	Cost Benefit Analysis
CSA	Central Statistics Authority
DEM	Digital Elevation Model
E	East
Ele.Diff	Elevation Difference
ERA	Ethiopian Roads Authority
FB	Free board
GIS	Geographical Information Systems
GPS	Global positioning system
GT	Golba Tegene
HEC	Hydrologic Engineering Centre
HMS	Hydrologic Modeling System
IDF	Intensity-Duration-Frequency
Km	Kilometer
M	Million
masl	meter above sea level
MoWR	Ministry of Water Resources
P	Precipitation
PLC	Private Limited Company
PMF	Probable Maximum Flood
RAS	River Analysis System

RF	Rainfall
SCS	Soil Conservation Service
UH	Unit Hydrograph
WMO	World Meteorological Organization
Yr	year

1 Introduction

1.1 Background

Urban flooding is a serious and growing development challenge. Against the backdrop of demographic growth, urbanization trends and climate changes, the causes of floods are shifting and their impacts are accelerating. This large and evolving challenge means that far more needs to be done by policy makers to better understand and more effectively manage existing and future risks.

Adama city is one of the cities which is highly affected by flood resulted from unnatural and natural resources situations and events. The runoff from rural areas is the leading source of impairments which affects the city by flooding problems. To ensure sustainable economic development and bring stable society it is important to implement community based participatory flood risk mitigation and management systems. The system may minimize flood and erosion hazard and improve the livelihood of residence of the city. Through comprehensive and integrated natural resources development and productivity enhancement measures also optimize the use of the existing natural resources and untapped potential in both degraded areas and the remaining potential areas of the city.

Interaction man with nature is not certainly defined but probably forecasted with continuous efforts. According to Adama City Administration Environmental Protection Office(ACAEPO); some of the environmental and livelihood problems identified are; Low agricultural production and productivity, Absence of extension services and agricultural input supply, Degradation of natural resource (Soil erosion, Deforestation, Gully formation), Lack of potable water for both human and animals, Lack of road which connect rural and urban Kebeles to Adama City, Lack of school and health centers, Problem of projects constructed without considering the environmental situation especially flood problem, particularly, in Awash macro-catchment where this has adverse effects on the city by runoff as flood sources become seasonal whereas in Migra macro-catchment identified problems are problems of flood from upper catchment, absence of proper drainage system for water disposal, lack of proper water way treatments, absence of cutoff drain, deforestation and unmanaged use of the adjacent boundary area, improper (misuse of compensation) fee for the displaced farmers and improper mining of the minerals that create sever gorges. Deforestation in search of income for food, construction and expansion of agriculture cause soil erosion leading to low soil fertility, occasioned and aggravated by inadequate and inappropriate soil conservation structures on sloppy lands are among the hotspots identified. Traditional cultivation of up-down the slope, annual crop and mono-cropping are among the highlighted environmental problems.

Inadequate investment in the natural resource conservation and uncompetitive market for natural resources in Rural-urban Kebeles and Adama city has also caused more environmental degradation. Natural resources are viewed as free and of little value thereby lacking incentives. These consequently affect demand driven extension service approach to sustainably build the capacity of the rural populations to embrace sustainable farming systems and know-how on soil conservation measures. Besides, lack of adequate extension services has propagated adoption of farming systems harmful to the environment for instance increased overgrazing on available vegetation causing bio-diversity loss. Such problems among rural urban Kebeles Communities are not only resulting from lack of knowledge but also culturally instigated. Population growth rate is at 4% per year (CSA, 2008). This rapid increase in population has caused farmers to expand their agricultural activities into the hill sides and seasonal river banks resulting into more degradation of land, and forests and consequently, increasing risk levels (ACAEPO, Jan2012).

In the past experiences of developed countries, risk oriented planning focused on socio-economic consequences of flood and a flood hydrology which is based on a safety oriented approach inherited from structural flood protection are well practiced and established so far to minimize flood risks with less attention of the risk management that needs identification and assessment.

In Ethiopia it is mostly practiced response that conditional and temporary risk oriented protection measures without preconditions. As a first step in urban flood risk management, policy makers need to understand the flood hazard that can affect the urban environment. Understanding hazard requires a better comprehension of the types and causes of flooding, their probabilities of occurrence, and their expression in terms of extent, duration, depth and velocity.

This understanding is essential in designing measures and solutions which can prevent or limit damage from specific types of flood. Equally important is to know where and how often flood events are likely to occur, what population and assets occupy the potentially affected areas, how vulnerable these people and their settlements are, and how these are planned and developed, and what they already do towards flood risk reduction. This is critical in grasping the necessity, urgency and priority for implementing flood risk management measures.

Different countries protect flood based on various mitigation measure with respect to the environment situation. Flood control measures may be structural or nonstructural. Structural measures are those that alter the river system by means of structures in the watershed (extensive measures) or in the river (intensive measures) to prevent flood water overflowing into the flood plain. Non-structural measures are those in which the losses from flooding are reduced for the convenience of the population, using preventive measures such as flood warnings, zoning of risk areas, flood insurance, and individual protection measures ("flood proofing").

Urban flooding is often identified with insufficient drainage capacity via urban watercourses and piped systems, with interaction from an underground sewerage system and also perhaps from fluvial inputs.

Flood is one of the major hazards in Ethiopia. It has mostly negative socio-economic consequences in some parts of the country. The national topography of high land mountains and low land plains linked with natural drainage systems. When heavy rains fall in high land areas flood occurs in the adjacent low land areas.

There are mainly three cases that how flood occurs in the country. When we look at the first case as a result of prolonged heavy rain fall causing the river overflow and inundate areas along the river banks in low land plains. In the second case, when excessive heavy rainfall occurs around a certain area and the area natural drainage system interrupted by another natural and/or manmade factors. The third is a flash flood characterized by a sudden onset with little lead time for early warning and often resulting in significant damage on social and economic conditions. In all the cases different parts of the country are highly affected both urban and rural areas. Usually the above cases of flood conditions occur and affect cities, towns and rural kebeles.

When we see the damages in urban areas, loss of lives, social instability and crisis, public infrastructures damage and economic losses are uncovered and seek answer. Such effects especially for developing counties like Ethiopia create a huge damage of double jeopardy in each situation.

In Ethiopia, one of flood prone areas is the Rift Valley zone. The Rift Valley runs through Ethiopia from the Red Sea to the Kenya border. In northern Ethiopia it forms the Danakil Depression, an inaccessible and inhospitable desert that dips to the lowest point on the earth's surface. In this zone there are different types of urban and rural people settlements from highly densely populated area like Adama to sparsely populated rural kebeles. These areas relative to the other parts of the country are a low lying and highly likely flood prone area of the country. The weather condition is dominantly 'Kola' or arid.

Adama is one of the largest and populated cities of the country and located right at the western edge of the Great Rift Valley, it represents the gateway to the Arsi heartland to the south east and the Great Afar Triangle to the North West.

Basically there are two major watershed sites at Adama. Under these watersheds, 14 sub watersheds with varies characteristics and geographical features are already identified by the city administration environmental protection office.

The watershed sites are Awash and Migira macro-watersheds and climatically they are categorized under Kola agro-climatic Zone. Total area of the Awash main watershed 7141.8 ha, location north east. Total area of Migira main watershed is 4566.7 ha, west. Total watershed area covers 11708.5 ha and the rest about 2250 ha drained outside direction. The selected study sites Kersa and Golbategene catchments have 74ha and 159ha area coverage as source of water that accumulate in a big flat depression as there is no outlet because of the Migira ridges.

From the ten identified watershed location Boku Shenen sub watershed has unique feature characteristics. It has four catchments (Kersa, Golbategene, Haro Ya'a, and Birka Roba) having different watersheds that cause significant effect to the lives and nature of adjacent low lying environment. This low lying area has been susceptible to excessive rain fall flooding and flash flooding.

1.2 Problem Statement

Adama city is repeatedly affected by an extraordinary flood that becomes common in most parts of the region every year causing loss of lives and property damage as well. The City is situated in various land features such as gentle and steep slopes, gorges, plateaus, hills and mountain topography which is exposing the city to floods and gully erosion as the natural drainage runoff across the lower and virtually flat area which is with major land use of residential. Flood risk management of the city is not well organized and established and hence every year flood risks occurred and create social crisis. In January 2013, a significant and valuable work has been done by the city administration environmental protection office. The office tries to identify the major watershed areas of the city and their sub watershed areas of each kebeles. It also shows and recommends with respect to environmental protection different mitigation measures that must be carried out by the government, urban & rural communities and concerned partners. Those mitigation measures are indeed important and should be strengthened and supported by engineering aspects. Hydrological analysis, hydraulic analysis and structural designs together with risk management system are essential for better flood risk mitigation measure of the city.

Boku Shenen kebele is one of highly affected areas of the city by flood. Because of poor drainage system and unorganized mitigation measures or options, flood hazard is becoming series and hard that creates a social crisis in the kebele from time to time. On the other side population growth and urbanization in the kebele has been growing as highly populated area with social infrastructures, factories, housings, and government & private institutions.

The existing drainage system which is the drainage canals, different types of culverts like pipe and box culverts cannot accommodate when a maximum flood discharge comes from the upstream catchment. Thus the runoff flood discharge overflows and inundates the village, highways (Adama-Assela & the new access road), main and sub streets of the kebele. Such situations create different complications to the community every year; loss of lives, loss of the people's different assets, damages of social infrastructures and institutions. So such type of areas flood risk assessment and management based on studied mitigation measures is absolute and incontestable to minimize the damage. Nevertheless there are no well-organized and studied options in order to handle the risk. Particularly where watershed areas are different and more than one like Boku Shenen kebele, it needs well identified and assessed system for safe and sustainable flood risk mitigation measures.

Figure 1.0: Pictures showing small part of the early morning of May 10, 2015 damages

Figure1.1: GolbaTegene and Kersa streams during dry season

1.3 Objectives of the study

1.3.1 General objectives

The general objective of the study is

- Reducing flood risks and damages caused by frequent and extraordinary flood

1.3.2 Specific Objectives

- Estimate peak flood discharge
- Recommend flood risk mitigation option
- Recommend cost benefit analysis approach

1.4 Scopes of the Study

The scope of the study was limited to analysis of 62 years rainfall data and then computing the peak runoff discharge that caused damages by the help of selected soft wares and model. Despite of previous study of the area, assessment of flood risk mitigation option were also another important part of the study. Its design and benefit cost analysis approach was also essentially considered to achieve project objectives.

1.5 Outline of the Thesis

This thesis is divided into six chapters. Chapter one provides introduction, general background, problem of the study area, the objective and scope of the study. Chapter two is literature review of the study. General approaches of the methodology of the research are included in chapter three. Chapter four deals with results of data analysis and then chapter five is detention pond/dam analysis. Chapter six are about conclusion & recommendation. References and the appendix parts are also included at the last pages.

2 Literature Review

2.1 Overview of Flood

Flood risks can only be mitigated but not removed. Floods usually result from a combination of meteorological and hydrological extremes, such as extreme precipitation and flows. However they can also occur as a result of human activities: flooding of property and land can be a result of unplanned growth and development in floodplains, or from the breach of a dam or the overtopping of an embankment that fails to protect planned developments.

Descriptions and categorizations of floods vary and are based on a combination of sources, causes and impacts. Based on such combinations, floods can be generally characterized into river (or fluvial) floods, pluvial (or overland) floods, coastal floods, groundwater floods or the failure of artificial water systems. Based on the speed of onset of flooding, floods are often described as flash floods, urban floods, semi-permanent floods, and slow rise floods.

All the above-mentioned floods can have severe impacts on urban areas – and thus be categorized as urban floods. It is important to understand both the cause and speed of onset of each type to understand their possible effects on urban areas and how to mitigate their impacts. (Abhas K Jhaet.el. (2012))

In general the causes of floods are manifold, but they can be categorized broadly under the following :(Donald W. Knight, et.el. (2006)).

Natural causes:

- Precipitation (rainfall, hail & snowmelt)
- Landslides (slope instability, erosion, seismic activity)
- Storm surge (e.g. low pressure in sea raising tidal levels)
- High groundwater levels (hence quicker runoff in chalk catchments or saturated ground)
- Glacier melts or collapse (due to volcanic action)
- Climate change (affecting precipitation and sea level).

Manmade causes:

- Dam failures (catastrophic, overtopping, piping, etc.)
- Embankment failures (river & coastal flood defense embankments)
- Floodplain encroachment (building on floodplains, loss of storage)
- Change of land use (crop change, compaction of soil, deforestation, etc.)
- Inadequate planning controls within whole catchment area (local & national)
- Inadequate drainage capacity (urbanization) & siltation (natural)

- Inadequate integration (e.g. between river and underground sewer/drainage systems)
- Inadequate maintenance (urban watercourses, blockage of culverts, sewer systems).

It is also important to distinguish between different types of flood (flash floods & long period floods), as well the predominant setting (fluvial, urban and coastal flooding).

The most common causes of flood in Ethiopia are

- Excessive rainfall leading to extraordinary runoffs
- Poor drainage systems and drains of inadequate capacity
- Silting up of the natural drains and river beds from sediments due to erosion in the catchment area.
- Sudden failure of water retaining structures
- Inadequate integration of drainage systems & poor maintenance
- Damage of existing drainage, situation plugging with refuse and other solid waste.
- Lack of attention from all concerned bodies

Flooding affects every section of people; systems in a city, some of the effects are summarized below :(B2-36, PDF)

Economic effects

· Damage to Public buildings, Public utility works, housing and house –hold assets.

· Loss of earning in industry & trade

· Loss of earning to petty shopkeepers and workers

· Loss of employment to daily earners

· Loss of revenue due to Road, Railway Transportation Interruption

· High prices for essential commodities.

After flooding, government has to put many resources for aiding e.g., police force, fire control, aid workers and for restoration of flood affected structures, persons, live-stock etc. Flood cause a great economic loss to the country, individual and to the society.

Environmental effects Damage to surroundings, forests, ridges, wild-life, urban community-trees, farm lands, shrubs in go downs etc result imbalance of eco-system of the city.

Effect on Traffic Flooding results in the damages of roads, collapse of bridges causing traffic congestion which affect day-to-day life and other transportation system.

Effect on Human Beings

Human lives: Sometimes floods in Adama cause loss of life and people become homeless.

Psychological impact: The people of all ages who stranded in flooding suffer a great psychological impact disturbing their whole life and the society as whole.

Live Stock: The live stock is the most affected living being due to urban floods. It is difficult to care for them particularly when human being itself is in trouble.

Disease: Flooding usually brings infectious diseases, e.g. military fever, pneumonic plagues, dermatopathia, dysentery, common cold, Dengue, break bone fever, etc. Chances of food poisoning also become more where electric supply interrupted in food-storage area due to flooding.

Public Inconveniences: Flood causes impairment of transport and communication system due to which all people of all section get stranded e.g. school children, college students, office goers, vegetable & milk venders etc. The basic and essential commodities also do not reach to the common person. This result either starvation to poor persons or high priced.

Table 2.1: Factors contributing to flooding (WMO, March 2008)

Meteorological Factors	Hydrological factors	Human factors aggravating natural flood hazards
• Rainfall • Cyclonic storms • Small-scale storms • Temperature • Snowfall and snowmelt	• Soil moisture level • Groundwater level prior to storm • Natural surface infiltration rate • Presence of impervious cover • Channel cross-sectional shape and roughness • Presence or absence of over bank flow, channel network • Synchronization of run-offs from various parts of watershed • High tide impeding drainage	• Land-use changes (e.g. surface sealing due to urbanization, deforestation) increase run-off and may be sedimentation • Occupation of the flood plain obstructing flows • Inefficiency or non- maintenance of infrastructure • Too efficient drainage of upstream areas increases flood peaks • Climate change affects magnitude and frequency of precipitations and floods • Urban microclimate may enforce precipitation events

Various preventive measures are applied in different countries based on the flood, environment and social conditions. Some of them are mentioned below :(B2-36 PDF)

Construction of flood protection structures such as Marginal bunds on banks as barriers for flooding, Regulators on drain, banks raised, a large number of spurs, bed bars, studs and bund to protect the embankments, Regulators with mobile pumping arrangement can be made ready where there is frequent risk of main drain/river flowing at higher level than max out fall level of out falling drains, Supplementary drain and at any out falling drains into large streams, Channelizing, lining etc. can also be considered supplementary drain to cater for a large design discharge, Construction of detention or retention ponds/dams and barrages on/around streams, Construction of supplementary drain can reduce flood risks

Improvement of drainage efficiency such as Desilting, cleaning of road, bell mouth, gullies, removal of debris, solid waste materials from all drains(all drains are checked and cleaned before rainy time to ensure that they are not blocked or collapsed), Constructions of drain to a certain design capacity, Main drains, supplementary drain, out fall drains etc. carry lot of silt from domestic sewers and therefore continuous desilting of these drains is being carried out by deploying machines, Implementing properly networked drainage system

Rain water harvesting: plays a key role in holding floods and urban water scarcity. There are many ways of rain harvesting adopted which will go a long way in reducing floods. The ways are On-channel storage of rain water in storm drains, Artificial recharge trenches, Check dam, Development and deepening of village ponds, Providing detention basins, Creation/revival of water bodies, Rain water harvesting structures.

Flood-plain management: Floodplains are where the river naturally stores water during a flood. This needs well integration with concerned bodies in order to revive the course and flood plain

Planting sturdy trees sustaining draught as well flooding: This will result in reducing soil erosion and run-off coefficient of the area and in turn reduce the flooding

Preparing of Master Drainage plan: data shall be properly documented and shall be used in planning and construction of drainage system

Land use and development planning

It is noted that flood continues to cause the largest number of deaths in the poor and heavily populated countries of the world. The global factors that might govern the future prospects for flooding are related to population growth, pressure on land use, climate change and insurance market response.

A representative number of floods in different parts of the world are highlighted below (Donald W. Knight, et.el. (2006))

- Bangladesh, May 1997

Tropical cyclone with winds up to 200 km/hr occurred. The storm surge hit at low tide, which reduced the damage. Many lives saved by use of cyclone shelters. Estimated damage of 95 killed, 2 million affected and 500,000 houses ruined.

- China, June–September 1998

Maximum discharge of Yangtze River at Wuhan was 72,300 m3 s_1, with 8 flood peaks spread over 83 days. Alongside the Yangtze, 1320 people killed, 200 million people affected and 3 million ha inundated. During 1998 China experienced devastating floods that killed over3000 people, inundated 21 million ha of land, destroyed 5 million houses and caused $21 billion of direct economic loss. Around 175,000 soldiers and 8 million civilians worked to reinforce the Yangtze River embankments.

- Mozambique, January–February 1997

Heavy rain from mid-January caused severe floods in central and North-Western Mozambique, causing 35 deaths and affecting 400,000 people. Appeal for food, medicine, seeds and funds for transport of emergency items and rehabilitation of roads and bridges.

- USA, Mississippi, 1993

Once in 500 year flood event. Floods inundated 75 towns, causing 47 deaths, $11 billion worth of damage, 20 million acres of agricultural land damaged, 45,000 homes damaged or destroyed and 74,000 people evacuated. Whole towns are now being relocated – but not major cities like St Louis.

- Dire Dawa, Ethiopia, August 5/6 2006 (Tesema Habte, April2009)

On the night of 5/6 August 2006, the most severe and recent flood to date swept through Dire Dawa, resulting in over 256 fatalities, rendering nearly 10,000 homeless and significant damage to the flood defenses, public infrastructure, housing and livelihoods. The high death toll was largely due to the fact that the flood peak occurred at night when people were asleep.

The basic principles of flood alleviation are outlined as follows :(Donald W. Knight, et.el. (2006))

1. *Reduce flooding (mainly structural measures)*

. dams (storage in on-line reservoirs)

. flood detention basins (storage in off-line reservoirs)

. river training (improve conveyance, raise levees, lower floodplain)

. high flow diversions (add conveyance, by-pass channels)

2. *Reduce susceptibility to damage (mainly non-structural measures)*

. physical & mathematical models (use to optimize solutions, produce flood risk maps)

. flood forecasting & warning (real time, use of national website &flood line, etc.)

. flood plain development regulations (broad scale planning strategy)

. flood proofing of buildings (protect/modify lower floors & use upper floors)

3. *Reduction of impact of flooding*

. disaster preparedness (early warning)

. education (inform the public about flood issues and problems)

. insurance (linked to government expenditure on flood defense)

. improve emergency response (plan for worst case scenarios)

. post flood recovery (give generous assistance following disaster)

4. *Restoration of natural resources*

. zoning (restoration of rivers/floodplains, wetlands/forests)

. permanent evacuation (re-development of urban areas out of floodplain)

. tax structure (floodplain gain,)

. national flood insurance scheme (for those who adopt & enforce good practice, USA)

As it is seen there need further attention and well integrated risk assessment to minimize the effect of flood. It is noted that the study area flood type is described usually as a flash flood. A flash flood characterized by a sudden onset with little lead time for early warning and often resulting in significant damage on social and economic conditions.

Structural preventive measures have significant contribution in reducing flood risks especially flash flood. Flood water storage structures like detention ponds/dams might become feasible in some areas like Adama Boku-shenen. It is designed to hold storm water runoff and release the water slowly to prevent downstream flooding and stream erosion. These structures are an extremely effective water quality control measure and significantly reduce the frequency of erosive floods downstream.

2.2 Flood Estimation

Many hydrologic methods are available for estimating peak discharges and runoff hydrographs. Each method has a range of application and limitations. Some of the methods are such as rational method, SCS method, regional regression analysis, and analysis of stream gage data.

2.2.1 Rational Method

A rational approach is to obtain the yield of a catchment by assuming a suitable runoff coefficient. It estimates the peak runoff at any location in catchment area as a function of the area, runoff coefficient, and rainfall intensity for duration equal to the time of concentration. It is best suited to urban storm drain systems and rural ditches. It shall be used with caution if the time of concentration exceeds 30 minutes. This method is used for catchment areas less than 50 hectares (0.5km2) and expressed as below.

$Q = 0.00278CIA$ ………………………………………… (1)

Where: Q = maximum rate of runoff, m^3/sec

C = runoff coefficient representing a ratio of runoff to rainfall

I = average rainfall intensity for a duration equal to the time of concentration, for a selected return period, mm/h

A = catchment area tributary to the design location, ha

2.2.2 SCS and other Unit Hydrograph Methods

The unit hydrograph used by the SCS method is based upon an analysis of a large number of natural unit hydrographs from a broad cross section of geographic locations and hydrologic regions. This method can be used for catchment areas greater than 50 hectares (0.5km2).

This technique requires the same basic data as the Rational Method: catchment area, a runoff factor, time of concentration and rainfall. The SCS approach, however, 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 a storm. With SCS method, the direct runoff can be calculated for any storm either real or fabricated, by subtracting infiltration and other losses from the rainfall to obtain the precipitation excess.

The SCS runoff equation is therefore a method of estimating direct runoff from 24-hours or 1-day storm rainfall.

$Q = (P - I_a)^2 / (P - I_a) + S$ ……………………………………………..(2)

Where:

Q = accumulated direct runoff, mm

P = accumulated rainfall (potential maximum rainfall), mm

I_a = initial abstraction including surface storage, interception, and infiltration prior to runoff, mm

S = potential maximum retention, mm

The empirical relationship used in the SCS runoff equation for the precipitation excess, and hence the runoff will be zero until the accumulated rainfall exceeds the initial abstraction. Therefore, the cumulative excess at time *t* is:

$I_a = 0.2S$ ……………………………………..……………. (3)

Substituting, we get

$Q = (P - 0.2S)^2 / (P - 0.8S)$ …………………………….. (4)

S is related to the soil and cover conditions of the catchment area through the CN. CN has a range of 0 to 100, and S is related to CN by:

S= 25400/CN-254 ……………………… (5)

2.2.3 Regional Regression Analysis

Peak flow can be calculated by using regression equations developed for specific geographic regions. In the equations the dependent variable would be the peak flow, and the independent variables may be area, slope, channel geometry, rainfall and other meteorological, physical or site specific data. This method shall be used for all routine designs at sites where applicable. Regression Equations are a commonly accepted method for estimating peak flows at ungagged sites or sites with insufficient data. Also, they have been shown to be accurate, reliable, and easy to use as well as providing consistent findings. Regression equations are one of the preferred methods for estimating peak flows for larger catchment areas. A regional approach to estimating floods at ungauged sites can be adopted using this regression model to predict flood.

2.2.4 Analysis of Stream Gauge Data

Generally, the drainage designer will need to acquire a record of the annual peak flows for the appropriate gauging station. If adequate data are not available, the design peak discharge should be based on analyses of data from several stream flow-gauging stations. In some cases, a site requiring a design peak discharge is on the same stream and near an active or discontinued stream flow gauging station with an adequate length of record. For example, at least 8 years of continuous or synthesized record is for 10 years discharge estimates; and 25 years for 100-year discharge estimates. With at least 25 years of continuous or synthesized stream gage data the best fit frequency analysis is considered to be the most reliable method for estimating flood frequency relationships and shall be used for all designs. Having determined that a suitable stream gauge record exists, it is necessary to determine if any structures or urbanization may be affecting the peak discharges at the design site. (ERA Drainage Manual, 2013 & 2001)

2.3 Previous Studies of the Area

In the previous times flood study of Adama city had not been well done in an organized manner for safe flood risks protection. In 2013 ACAEPO developed a supportive macro-watershed study of environmental concern aimed at analyzing common themes and particular variations in the experience of the 14 watershed sites, with the ultimate goal of eliciting lessons learned on participatory and integrated watershed management. In the study the major problems that maximize flood risks are identified, 14 catchment areas identified. Possible mitigation measures both structural and non-structural mentioned but lack engineering points of view.

The other study was carried out by the Adama city master plan revision project through its partner consultant of PACE consulting Architects and Engineers PLC. The scope of the work has two major components, preparation of drainage plan and potential site selection for solid waste disposal.

This report summarizes the existing situation relating the physical condition of the city and its environment as a whole, the condition of the storm drainage and solid waste disposal systems. Some of the important comments given by the report as long term solution are:

- The first step would be to review the development plan road layout and to optimize these, where possible, to maximize drainage efficiency.
- Proper estimation of storm water volume for each interceptor is necessary.
- Election of site-specific types of roadside ditches, e.g. open channels or tunnels, channels with partly closed tops, or closed conduits dependent on road use and side access requirements.
- Structural designs should consider constructions using locally available materials.
- Detail design works also include development of a comprehensive and sustainable plan for catchments protection, land stabilization and follow-up works.

Even though that different studies and recommendation were took place, still study requires detail drainage network design for sustainable flood mitigation of the city.

3. Methodology

The methodology followed for the study has the following parts as described hereunder.

i. Establishing the basis of the research: aimed at defining the theoretical basis, and formulating the research questions through the following steps.

• Literatures were reviewed to obtain a theoretical basis for the study and formulating the research objectives and defining the scope. To this effect, the main authors of textbooks in the field of hydrology were identified. The books were then reviewed in order to get a general understanding of the research area. Thereafter relevant articles from other publications were searched to conceptualize urban flood risk management. The search was made by using the following keywords: urban flood management, flood estimation, flood mitigation measures, flood damage analysis, and design methods. Apart from searching in libraries, internet sources were used to obtain recent articles and research papers in the area.

. Any previous studies to the area were examined to know the extents of others researchers recommendation for the problem of the study area.

. In addition to my knowledge to the area, discussion with local people, Kebele administration experts, Woreda municipality experts, and Woreda environmental protection office experts were conducted about the study.

ii. Conducting the study: aimed at finding out how much discharge produced from the catchments and caused damages to the area and what mitigation options can be applied based on the area condition, identifying existing practices of flood estimation and risk mitigation were done with the following approaches:

• Survey the area and topography then take notes of existing situations

• A desk study to get a picture of flood prevention practices and outcomes, and

• Interviewing key informants to get in-depth understanding of flood risk management processes being practiced, and to explore their opinions on risk mitigation.

• Computing quantitative descriptions on flood discharge and volume using proper methods

• Analyze flood mitigation options with its benefit and prepare design approach.

3.1 Description of the study area

Adama city is the third largest urban center in Ethiopia and is located about 90 km southeast of Addis Ababa at latitude of 8°33’ and longitude 39°17’. The city is in the Great Rift Valley of East Africa on the flat lowland between two mountain ridges. The average elevation of the built up area is about 1,620m.a.s.l with maximum difference of 70m between the highest and lowest sections.

Figure 3.00 Location of the study area

3.1.1 Climate

Adama and its immediate surroundings lie on the border between various climate zones. In terms of the traditional division according to altitude and temperature, Weina-Dega (sub-tropical) or Qolla (tropical) climate zone classify the area. However, if judged according to the moisture balance and plant growth, semi-arid classifies the area.

The mean annual ambient temperature in Adama is between 19 and 22 ^{o}C. The Maximum temperatures usually occur between March and May; the maximum temperature in this period can exceed 30 ^{o}C. The temperatures are at their lowest in November.

- Mean Maximum 27.8 ^{o}C
- Mean Minimum 13.2 ^{o}C

3.1.2 Rainfall

Generally, there are three seasons at Adama; Kermt (main rainy season), Bega (dry season), Belg (small rains), The rainfall recorded at Adama metrological station for the past 62 years (1953- 2014) indicated that the mean annual rainfall is about 224.18 mm. The maximum monthly average rainfall is below 70.5mm. Most of the rain occurs between June to September .The wettest months are July and August. The average amount of rainfall in July and in August is about 45.93 mm. The proportion of the precipitation in these two months is about 55% of the annual total. Data on rainfall intensity for Adama city is not available.

The rainfall intensity data recorded at the nearby station of Bishoftu, Kulumsa and Methara is sometimes considered; accordingly, the average maximum rainfall intensity in the three stations for the past 27 years (1975-2002) is about 39.7mm/hr.

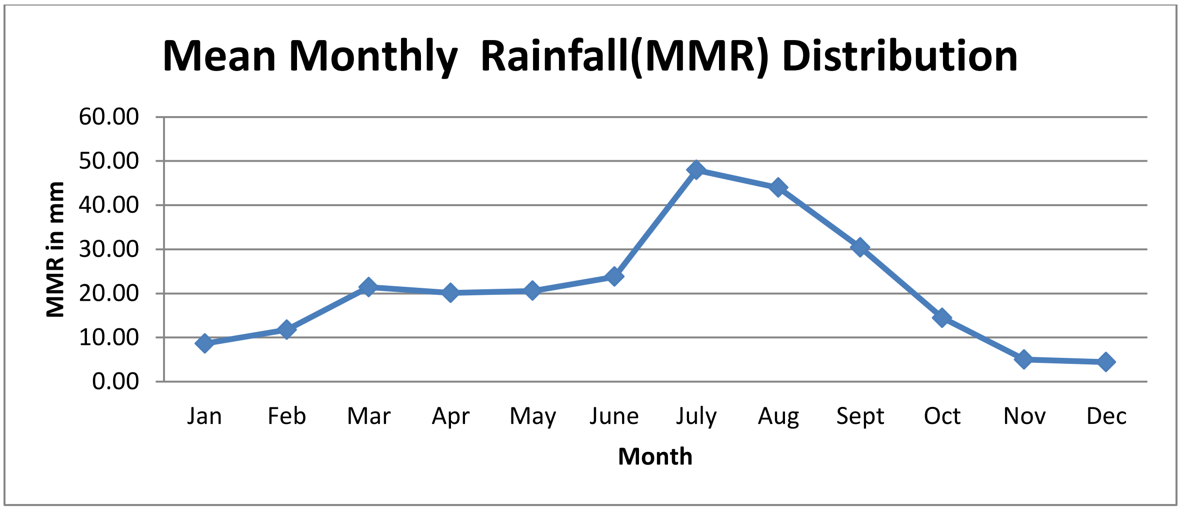


Figure 3.0: Adama MMR Distribution

3.2 Data Collection and Analysis

3.2.1 Data Collection

Primary data from field visit and secondary data from governmental and non- governmental organizations have been collected and analyzed for the study.

The primary data were collected at site. Surveying data like geographical coordinates of the pond/dam axis, gully cross sections were collected using GPS and become an input for the ArcGIS analysis.

The following table describes secondary data type and its purpose.

Table 3.1 Data type and purpose

Data Type	Purpose	Source
Topomap	generating town part and the gully feature geo-spatial data using Google map	Google
Kersa& GT Stream 0.5m contour interval map	for cross checking gullies cross section geo-spatial data using ArcGIS	MoWR
Maximum daily Rain-fall for Adama station	to compute hydrograph using SCS Method	Metrology Agency
ERA Intensity-frequency-duration curve for the region	to compute hydrograph using block method	ERA drainage manual
Soil type and land-use for Kersa& GT	For curve number computation	MoWR
1:50000 scale top-map of the area	for cross checking geo-spatial data using Global Mapper	Ethiopian Mapping Authority

3.2.2 Data Analysis

3.2.2.1 Basin Model

It is an important component in watershed physical description. Its principal purpose is to convert atmospheric conditions into stream flow at specific locations in the catchment. The basin model is responsible for describing the physical properties of the watershed and the topology of the stream network. This can be facilitated by a program like ArcGis basin model by entering required data. It contains the modeling components that describe catchment data, infiltration, surface runoff and channel routing. Outflow is computed from meteorological data by subtracting losses of basin and transforming excess precipitation through the basin. Sub-basins can be used to model the catchment.

Although Kersa and Golbategene catchments are relatively small, they can be analyzed and described in detail by such models.

a. Sub-basins

A sub-basin is an element that usually has no inflow and only one outflow. They can be connected together in a dendritic (branched extension or shape of a tree branch) network to form a representation of the stream system. While a sub-basin element conceptually represents infiltration, surface runoff, and subsurface processes interacting together, the actual subsurface calculations are performed by a base flow method contained within the sub-basin. For the study area, the base flow was taken as zero.

The Kersa and GT basin catchments were described below:

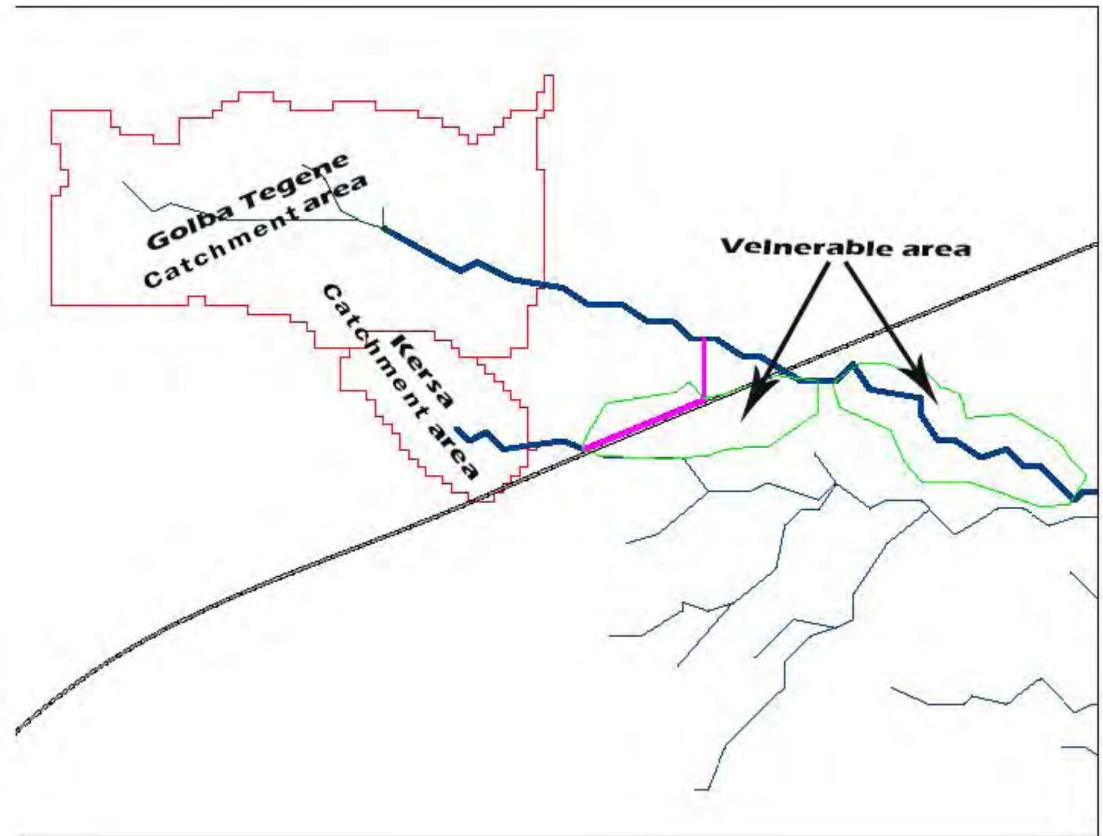


Figure 3.3 Golba Tegene and Kersa catchments

Loss

While a sub-basin element conceptually represents infiltration, surface runoff, and subsurface processes interacting together, the actual infiltration calculations are performed by a loss method. Due to the available data type SCS Curve Number Loss method is adopted for loss computation. After selecting the method, three parameters were required as shown below to compute the loss. These are:-

- Curve Number
- Initial Abstraction
- Impervious (%) (for this study, negligible)

Curve Number and Initial Abstraction, Ia

The relationship between I_a and S was developed from experimental catchment area data. It removes the necessity for estimating I_a for common usage. The empirical relationship used in the SCS runoff equation is:

$$I_a = 0.2S$$

S is related to the soil and cover conditions of the catchment area through the CN. CN has a range of 0 to 100, and S is related to CN by:

S= (25400/CN)-254

b. Reaches

A reach element conceptually represents a segment of stream or river; the actual calculations are performed by a routing method contained within the reach. Flow routing is a procedure to determine the time and magnitude of flow (i.e., the flow hydrograph) at a point on a watercourse from known or assumed hydrographs at one or more points upstream. In broad sense, flow routing may be considered as an analysis to trace the flow through a hydrologic system, given the input. For the study area routing was considered for GT pond spillway by using Muskingum method for the streams are ungagged.

c. Junctions

A junction is an element with one or more inflows and only one outflow. All inflow is added together to produce the outflow by assuming zero storage at the junction. It is usually used to represent a river or stream confluence. The junction element does not have any special data or properties.

3.2.2.2 Synthetic unit Hydrograph

The Soil Conservation Service (SCS) proposed a parametric UH model; this model is included in different programs and adopted for the transformation process. The model is based upon averages of UH derived from gauged rainfall and runoff.

There are three types of synthetic unit hydrograph:

- Those relating hydrograph characteristics (peak flow rate, base time, etc.) to watershed characteristics.
- Those based on a dimensionless unit hydrograph.
- Those based on models of watershed storage.

The SCS proposed model is relating the hydrograph characteristics to watershed characteristics and based on this the computation carried out in the later sections.

3.2.2.3 Meteorological Model

Based on the data available, two methods can be adopted as an option for precipitation modeling

I. Rain fall Frequency Analysis

II. Gage weights (different modes of storm)

I. Rain fall Frequency Analysis

Rainfall frequency analysis was used as a key input to the calculation of flood frequency and magnitude. 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 hydrologic data analyzed are assumed to be independent and identically distributed, and the hydrologic system producing them (e.g., a storm rainfall system) is considered to be space-independent, and time-independent. The hydrologic data employed carefully selected so that the assumptions of independence and identical distribution can be satisfied. In practice, this is often achieved by selecting the annual maximum of the variable being analyzed (e.g., the annual maximum rainfall, which is the largest instantaneous peak storm occurring at any time during the year) with the expectation that successive observations of this variable from year to year will be independent.

The rainfall frequency analysis has been carried out using the available 24 hour annual maximum rainfall data for Adama rain gauge station. ERA 2013 drainage manual also tries to develop the IDF curve for Adama region since there is no direct relationship between rainfall and flood frequency. Easy fit software was used for the analysis allowing fitting probability distributions.

II. Gage weights (different modes of storm)

Despite of the record, the actual duration of the storm is normally much less than 24 hours and will include all or nearly the entire recorded 24 hour rainfall amount. For this reason, three options were considered for meteorological modeling as additional approach of the rainfall analysis. These are:

a. Adopting the annual maximum daily rainfall as 24 hours duration storm
b. Adopting the annual maximum daily rainfall as 6 hours duration storm
c. Using ERA Intensity-frequency-duration curve

3.3 Peak Flood Estimation

There are different types of peak flood estimation methods. The choice of these methods depends on data available and the practical existing situations. Each method has a range of application and limitations. For this study SCS method was used because this method is most suited for computing flood peaks and run of volumes for catchments smaller than 65km2, with slopes of less than 30% and a time of concentration (Tc) less than 10 hours.

Regional regression equations are commonly used for estimating peak flows at ungaged sites or sites with insufficient data. Regional regression equations relate either the peak flow or some other flood characteristic at a specified return period to the physiographic, hydrologic, and meteorological characteristics of the watershed. In order for regression equations to be useful for Ethiopia, extensive study of drainage basins and hydrologic regions for the country is required.

Also analysis of stream gage data is not possible since the catchments are not gauged. Hence, the preferred methodology for this study is SCS method with the help of suitable Computer Programs ArcGIS and SCS models used to facilitate the calculations.

Table3.2: Application and limitation of flood estimation methods (ERA Drainage Manual, 2013)

Method	Input data	Recommended maximum area(km^2)	Return period of flood that could be determined (years)
Rational Method	Catchment area, watercourse length, average slope, catchment characteristics, rainfall intensity	<0.5	2 – 200, PMF
SCS Method	Catchment area, watercourse length, length to catchment centroid (center), mean annual rainfall, veg. type ,soil cover and synthetic regional unit hydrograph	0.5 to 65	2 – 200, PMF
Synthetic Hydrograph Method	Catchment area, watercourse length, length to catchment centroid (center), mean annual rainfall, veg. type and synthetic regional unit hydrograph	0.5 to 5000	2 – 200
Empirical Methods	Catchment area, watercourse length, distance to catchment centroid (center), mean annual rainfall	No limitation, large areas	2 – 200, PMF
Statistical Method	Historical flood peak records	No limitation, large areas	2 – 200 (depending on the record length)

SCS runoff equation is therefore a method of estimating direct runoff from 24-hour or 1-day storm rainfall. The equation is:

$$\mathbf{Q = (P - I_a)^2 / (P - I_a) + S}$$

Where:

Q = accumulated direct runoff, mm

P = accumulated rainfall (potential maximum runoff), mm

I_a = initial abstraction including surface storage, interception, and infiltration prior to runoff, mm

S = potential maximum retention, mm

The empirical relationship used in the SCS runoff equation for the precipitation excess, and hence the runoff will be zero until the accumulated rainfall exceeds the initial abstraction. Therefore, the cumulative excess at time *t* is:

$$I_a = 0.2S$$

Substituting, we get

$$Q = (P - 0.2S)^2 / (P - 0.8S)$$

S is related to the soil and cover conditions of the catchment area through the CN. CN has a range of 0 to 100 and S is related to CN by:

$$S = 25400/CN - 254$$

The ERA drainage Design hydrologic analysis procedure flowchart shows the steps for the hydrologic analysis and the designs that will use the hydrologic estimates.

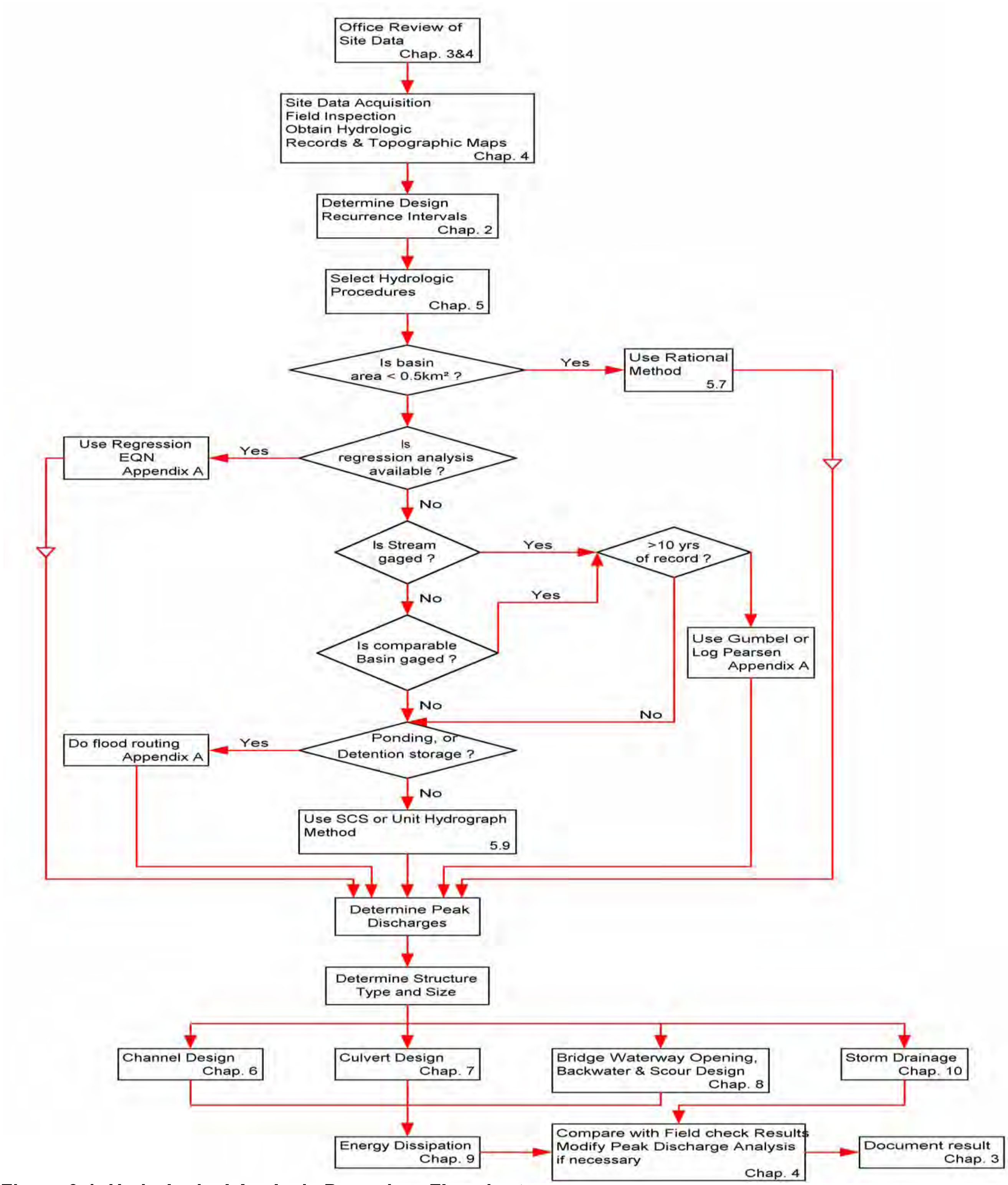


Figure 3.4: Hydrological Analysis Procedure Flowchart

Runoff Factors

The principal physical catchment area characteristics affecting the relationship between rainfall and runoff are land use, land treatment, soil types, and land slope.

Land use is the catchment area cover, and it includes both agricultural and nonagricultural uses. Items such as type of vegetation, water surfaces, roads, roofs, etc. are all part of the land use.

Land treatment applies mainly to agricultural land use, and it includes mechanical practices such as contouring or terracing and management practices such as rotation of crops. The SCS uses a combination of soil conditions and land-use (groundcover) to assign a runoff factor to an area. These runoff factors, called runoff curve numbers (CN), indicate the runoff potential of an area. The higher the CN, the higher is the runoff potential.

Soil properties influence the relationship between rainfall and runoff by affecting the rate of infiltration. The SCS has divided soils into four hydrologic soil groups based on infiltration rates (Groups A, B, C, and D). Care shall be taken in the selection of curve numbers (CN's). Use a representative average curve number, CN, for the catchment area. Selection of overly conservative CN's will result in the estimation of excessively high runoff and consequently excessively costly drainage structures. Selection of conservatively high values for all runoff variables results in compounding the runoff estimation.

For a watershed that consists of more than one soil types and land uses, a composite CN is calculated as:

$$CN_{Composite} = \Sigma A_i CN_i / \Sigma A_i \text{ .... (6)}$$

In which, $CN_{composite}$ = the composite CN used for runoff volume computations;

i = an index of watersheds subdivisions of uniform land use and soil type;

CN_i = the CN for subdivision i; and

A_i = the drainage area of subdivision i.

3.4 Flood vulnerable area delineation

Flood area delineation tried to be put by direct surveying and measurement of the area and relating with the Google map. Also in this study it is done by using AutoCAD and Google map.

3.5 Preventive measures and damages analysis of the area

Different types of flood damage mitigation methods structurally based on the flood, environment and social conditions. And although data limitations are a challenging, damages analysis were tried to be considered with the cost benefit analysis. Both points have been seen in more detail way in the next sections.

3.5.1 Preventive Measures/Options

When we consider the catchment situation and the size and position of the gulley, there exist feasible solutions. Hence the flood discharge produced by Kersa catchment (74ha) can be mitigated in two ways: the first option is creating a pond area so as to provide a lag time before getting in to inundating area. Another option is size modification that can incorporate channel size reducing factors. As can be observed, formation of sand domes, dead bodies, different dry wastes and gulley bank slid in the channel cause overflowing and inundation of the village.

In the case of Golbategene catchment, it is a bigger catchment (159ha) with large flood discharge that needs special treatment. In a general view measures to mitigate the impact of flooding in suburban areas can largely be divided into two groups - structural and non-structural.

Structural measures are those that involve physical works to lessen the effects of flooding, such as improvements to drainage infrastructure (property and flood modifications). These might otherwise be described as "engineered" solutions.

Non-structural measures are typically passive measures (response modification) and linked with town planning policies and building codes and involve longer-term consideration. These might include, for example, restrictions on where construction can take place, limitations on fill in floodplains and specification of minimum habitable floor levels for buildings.

This section considers structural measures, or "engineered" works to mitigate the impacts of flooding to the study areas.

Structural measures can be further subdivided into active and passive measures. Passive measures require no operation whereas active measures are those that require some form of operation or movement to provide a benefit. Active measures add a layer of complexity when considering the costs and benefits, because they add a level of risk.

Most strategies to mitigate the impact of flooding come with a cost. This can be a direct cost, such as the capital and operating costs associated with structural measures, or an indirect cost such as loss of social amenity or a reduction in property value. Inevitably, these costs need to be balanced against the benefits that are derived.

Possible flood preventive measures for Kersa and GT floods:

For Kersa; a retarding pond and/or channel modification.

For GT;

- Catchment treatment work
- Gulley/stream training work
- Levee & flood walls construction
- Detention pond/dam construction

Advantages and disadvantages of the preventive measures based on the current environmental and social conditions of the study area:

- **Catchment treatment work**

Advantages;

- maintain the ecology
- reduce sediment transport
- reduce surface flow & increase subsurface flow
- increase groundwater table

Disadvantages;

-High initial cost

-need integrated response

-may take many years

- **Gully/stream training work**

Advantage;

- maintain natural or existing channel
- reduce new route formation
- do not use extra land

Disadvantage

-very expensive for loose land formation

- **Levee and flood wall construction**

Advantage;

-economical for strong land formation

-simple & safe if well designed

-simple for construction & maintenance

-protect large area with relatively less investment cost

Disadvantages

-uneconomical for loose formation

-If overtopped, severe damage may occur and then pumping to the channel or basin needed; and an emergency plan should be put as an option

-sensitive to the design

-needs continues inspection and maintenance

- **Detention pond/dam construction**

Detention ponds have some inherent disadvantages that should be carefully evaluated for example;

-It requires a substantial area to achieve the necessary storage

- If overtopping occurs, result in severe damage at the d/s especially if large ponds/dams

Consequently it is important that the pond/dam should be properly designed, constructed, and maintained.

Primary use: To control release rate of storm water to receiving streams.

Additional use: Remove sediment and other pollutants from storm water.

The main reasons for use of dry detention basins are

- Reducing peak storm water discharges,
- Controlling floods
- Preventing downstream channel scouring.
- It is also probable that it will remove a limited amount of pollutants.

In selection of this preventive measure to the study areas, environmental, geological formations of the area and economic conditions are taken in to consideration.

3.5.1.1 Environmental Conditions of the Area

Less than 1.5ha which is currently used for cereal crop production once a year is selected for the storage. The storage area has insignificant effect to any inhabitants as it has been serving only for crop production. This piece of land owners can be compensated by city administration (CA). Relatively less polluted or clean steam flow will be created at d/s of the dam. The soil on the reservoir area can be used as major source of construction material for the dam body in turn increase its capacity with insignificant effect to the environment.

3.5.1.2 Geological Formations of the Area

Adama and its surrounding area are made of trachyte, ignimbrite, tuff (pumesious tuff), basalt and lacustrine sediments. The rocks are belonging to Pliocene and to quaternary age. Trachyte, rhyolite (obsidian rich) and ignimbrite are available on the hilly part of the area and pumesious tuff and lacustrine sediments are in the plain area.

In general, lacustrine soils have covered the immediate vicinity of Adama, except for some limited areas along the major riverbanks where alluvial deposits are the prominent cover sediments. The lacustrine sediments are mainly composed of clays and silts. The thickness of the deposits ranges from 30-40m. (ACA construction dept)

Geological formation of Kersa & GT stream courses, both are formed along the pumesious tuff and lacustrine sediment soil formation area. These soils are natural easily erodible and sliceable while flood discharge passes through it. Because of the fact that gullies of the study area are changing their route from year to year and the responses have become ineffective & uneconomical mitigation measures that were carried out by the city administration such as catchment treatment works, gully training works and levees construction works. According to the kebele administration environmental office report, six (6) thousand peoples are participating by labor for the preventive works.

The soil nature around u/s and d/s of the dam axis can be used as a primary choice for the construction. It is a long time practice to be used for water harvesting pond construction by the local community.

Topography of Golbategene site is convenient and accessible for construction. The maximum crest length the dam is not more than 170m which is in between two natural ridges. For Kersa, for a limited storage area, the local type of pond could be designed.

3.5.1.3 Economic Conditions

When options are very limited and unthinkable, economical advantage consideration may not be important. Adama city administration (ACA) has been deploying an average of six thousand people with an estimated equivalent cost of 360000 to 400000Br every year for only the study area flood prevention action.

One of the best options to reduce this cost and for better sustainable solution based on the environmental and geological conditions of the area was that provision of detention dam/pond. This detention dam consideration was worked out in detail in the next chapter.

3.5.2 Cost Benefit Analysis

A Cost benefit Analysis (CBA) assessment was generally complicated since the methodology implies a comprehensive assessment and it was hard to establish generalized costs for all the impacts on the society. Due to data limitation, it was early found that full CBA assessment was unable to perform but to still was able to test and evaluate the method. It is very important to note that data collection is a very important fundamental step, or even the most difficult step in flood damage analysis.

Average annual cost (C)

This cost was taken as the capital investment cost (construction cost) and maintenance & operation cost. The capital cost has to be recovered during a certain period equal to the life of the structure or less than that for conservative design along with a certain minimum interest rate. From depreciation and compound rate interest calculations, the capital recovery factor should be multiplied in order to get the equivalent annual recovery cost and it is given by:

Cost recovery factor, $C_f = (i(1+i)^N)/((1+i)^N - 1)$ ……………………(7)

Where i= the annual interest rate which is the minimum attractive rate of return, depending up on the need of the project (i can be specified by the concerned offices as per the economic advantage)

N = estimated life of the project in years.

Annual recovery cost, $X = C_f * R$ ………………………… (8)

Where R = the capital cost

Therefore;

The total annual cost, $C= X + M\&O_c$………………………….. (9)

Where $M\&O_c$ = maintenance & Operation cost

Annual benefits (B)

For any flood control project, the annual benefits can be worked out by calculating the average annual damages (AAD) that occur to the community due to devastations caused by flood, and then subtracting from it the average annual average damages that will still continue to occur even after the project completion.

Benefit, B = AAD (before the project) – AAD (after the project) ………………. (10)

So a damage table with generalized damage costs per unit area was tried to define as input to the damage assessment, see Table. These generalized damage costs was supposed to reflect the societal costs for certain flood depths.

The effects of vulnerability and capacity on flood risk can be represented as the increase or decrease of the loss rates of assets varying with water depth.

Risk = Probability × Consequence …………………… (11)

For flood risk, probability refers to the return period of flood event and consequence refers to the total flood losses/damages. In order to calculate the flood risk for a certain area, the annual average expected flood damage was considered by the integration of the above equation, which can indicate evolving trend of flood risk continuously in a long term.

For the damage assessment, building damage cost, street damage cost, farm land damage cost and livestock damage cost were considered for the analysis. These assets were taken as a sample for estimating the general direct and indirect damage costs of the vulnerable areas.

Table 3.3: Generalized costs values for the damage assessment

Flood depth [m]	Damage cost in Birr/m²				Total damage cost Birr/m²	Total damage area m²	Total Damage Cost in Birr
	Buildings	Streets	Farm land	Livestock			
0-0.05	0	0	0	0	0	150000	0
0.05-0.10	86	0	0	0	86	200000	17200000
0.10-0.20	133	115	0	0	248	250000	62000000
0.20-0.50	250	100	10	20	380	300000	114000000
0.50-1.00	1500	350	60	95	2005	350000	701750000
>1	1500	550	100	150	2300	350000	805000000

To establish the generalized building damage cost, it was very difficult since there was no any reliable data that can be measured for the study. A damage data of the past minimum of 30yrs should have been collected by the ACA, but it was a difficult job to obtain such reliable data. For the study, it was tried to estimate the total damage cost by random sampling and analyzing the risk area. Interview with local people was also important for damage estimation.

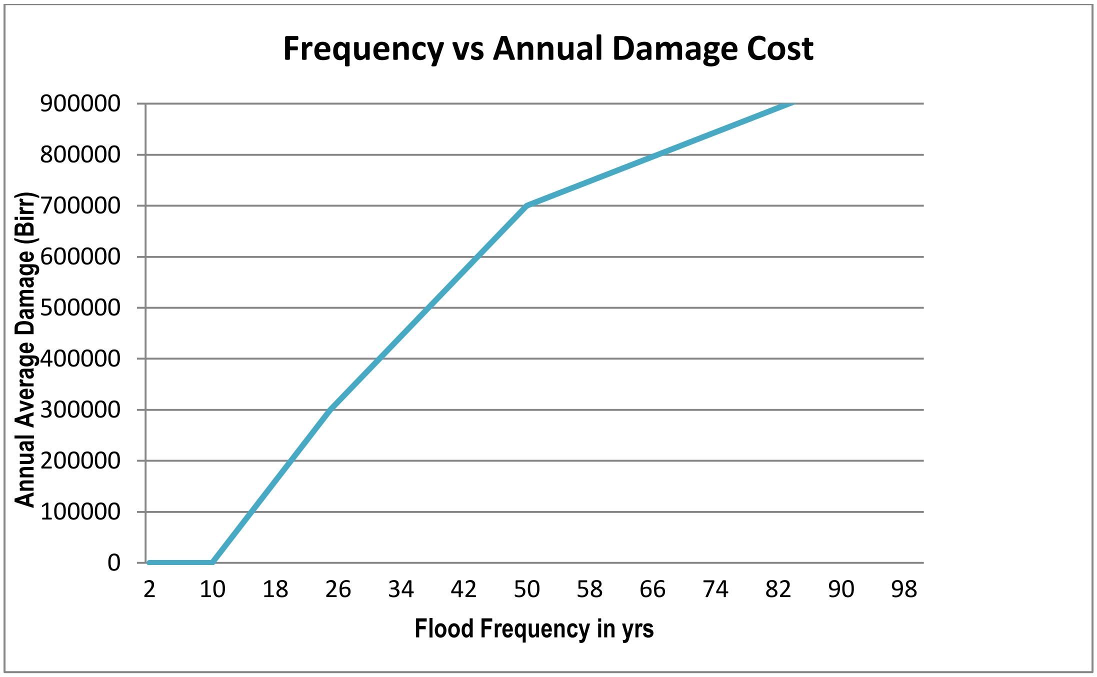
The generalized damage cost for the streets was also considered from an estimation of societal costs for certain flood depths. These costs reflect the damage to the street caused by the water but also the impact the flood has on the street users in terms of accessibility, time losses etc.

The generalized damage costs for farm land and livestock areas have been estimated with the defined building and street costs. They were chosen to be fair as it is reasonable that a farm lands & livestock can be flooded similarly with high costs as it would still be worth something for the residents.

Table 3.4: Average annual Damage

Return Period T_r (yrs)	P(xi)	AAD
100	0.01	1000000
50	0.02	700000
25	0.04	300000
10	0.10	0
5	0.20	0
2	0.50	0

Figure 3.5: Frequency Vs Annual Damage Cost



Then after the benefit cost ratio can be computed so as to justify that the scheme is economically viable, and is likely produce more benefits than the costs

Benefit cost ratio = Annual Benefits (B) / annual Cost (C) ……………….. (12)

For flood control projects, the ratio needs to be greater than one.

4. Results of Data Analysis

4.1 Frequency analysis

The most important hydro meteorological analyses for this study include estimation of:

- The 24-hour maximum annual rainfall values recorded Design flood discharge of those rives for Design of hydraulic structure and flood protection works.

With a lack of information with reference to the annual peak flow regime in the vicinity of project area, the only feasible alternative was to apply design rainfalls as the rainfall-runoff model inputs, thereby obtaining the design discharge values at various return periods at the hydraulic structure site.

The basic rainfall data applied for that purpose was the 24-hour maximum annual rainfall values recorded at the Adama rainfall station during the period 1953 – 2014. Appendix 5 shows the Annual 24hours peak rainfall of Adama rainfall station.

The Adama rainfall analysis initially carried out by EasyFit software which is used for the analysis allowing fitting probability distributions. And also the best fit probability distribution again analyzed by the histogram and plotting methods. The results as shown below:

Table 4.1 Descriptive Statistics

Statistic	Value
Sample Size	62
Range	74.8
Mean	60.49
Variance	349.87
Std. Deviation	18.705
Coef. of Variation	0.30922
Std. Error	2.3755
Skewness	0.78698
Excess Kurtosis	0.13901

Table 4.2 Adama Goodness of Fit - Summary

#	Distribution	Kolmogorov Smirnov		Anderson Darling		Chi-Squared	
		Statistics	Rank	Statistics	Rank	Statistics	Rank
1	Exponential	0.415	11	14.399	11	90.703	11
2	Exponential(2P)	0.22629	10	4.7957	10	19.32	10
3	Gamma	0.08217	7	0.51804	7	1.8987	7
4	Gamma(3P)	0.06215	4	0.36266	5	0.36608	4
5	Gumble Max	0.05958	1	0.29628	1	0.24697	3
6	Log-Pearson 3	0.06046	3	0.31833	3	0.23487	1
7	Lognormal	0.06375	5	0.3405	4	0.42153	5
8	Lognormal(3P)	0.05999	2	0.31743	2	0.23635	2
9	Normal	0.12335	9	1.2994	8	4.1246	8
10	Weibull	0.114	8	1.6755	9	4.4831	9
11	Weibull 3P	0.07015	6	0.51226	6	1.255	6

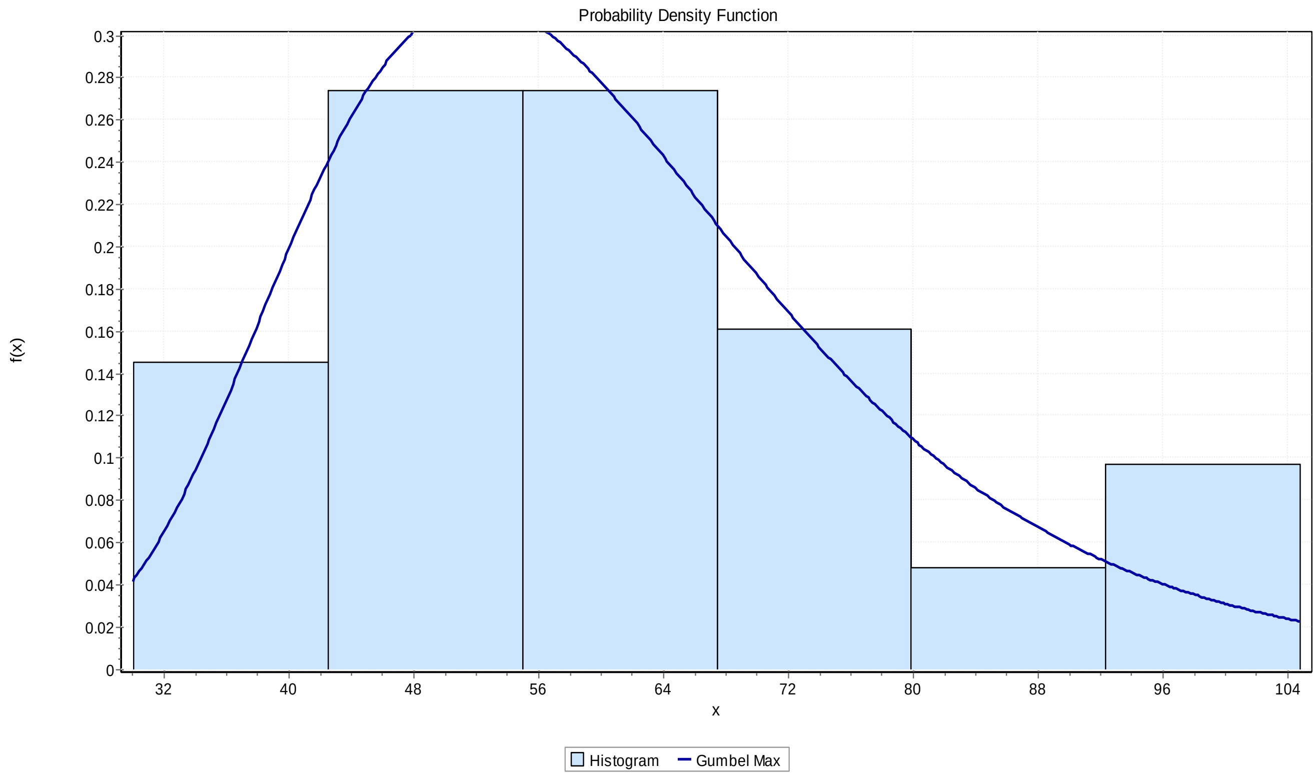


Figure 4.1: Histogram & Gumble Max Probability Function

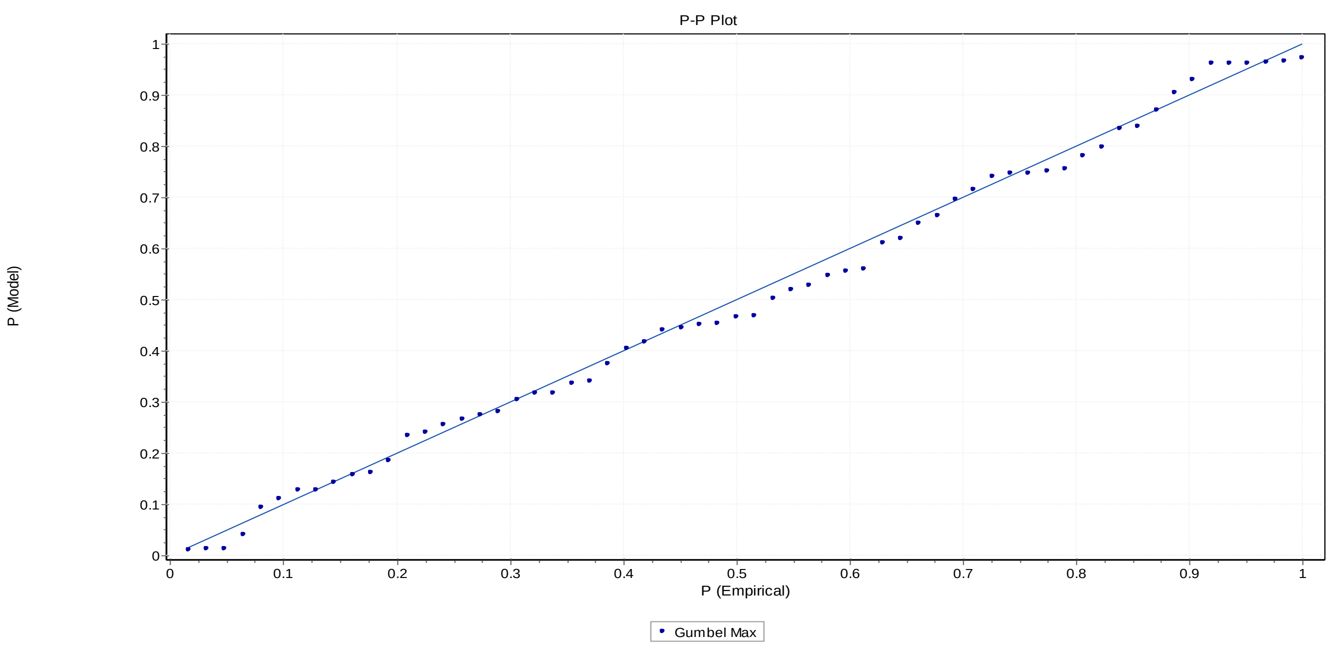


Fig 4.2: Probability Plotting Position of Gumble Max (EV1) Distribution

Using the weighted best fit test ranking from the above three common tests (Kolmogorov Smirnov, Anderson Darling and Chi Square tests), Adama rainfall data fits Gumble Max parameter distribution.

Table 4.3: Adama station rainfall T year's extreme rainfall magnitude ($X_T$in mm)

Return Period T (year)	X_T (Gumble Max distribution) in mm
2	57.41
5	73.94
10	84.88
25	98.70
50	108.96
100	119.14
500	142.66
1000	152.78
2500	166.14
5000	176.25
7500	182.16
10000	186.36

Table4.4: Summary of Adama annual maximum daily rainfall (mm) for selected return periods

Mean	60.49
Max	104.80
Min	30.00
St. Dev	18.70
T-2	57.41
T-5	73.94
T-10	84.88
T-25	98.70
T-50	108.96
T-100	119.14
T-500	142.66
T-1000	152.78
T-2500	166.14
T-5000	176.25
T-7500	182.16
T-10000	186.36

- ERA has developed the below using SCS method and based on a 24-hour storm event which has a Type II time distribution. The Type II storm distribution is a 'typical" time distribution which the SCS has prepared from rainfall records.

Table4.5 : 24hr Rainfall Depth Vs Frequency (ERA Drainage Design Manual – 2013,P5-61)

24 hr Rainfall Depth (mm) vs Frequency (yr)								
Return Period Years	2	5	10	25	50	100	200	500
RR-A3(Adama)	47.54	59.61	67.66	77.92	85.62	93.34	101.13	111.58

RR- Rainfall Region

4.2 Using Different Mode of Storms

a. Adopting the annual maximum daily rainfall as 24 hours duration storm

The 50 years return period daily maximum rainfall, 108.96mm, is changed to 24hrs incremental rainfall.

Using, p = M * square root (T), ► M = p / square root (T), ► M = 22.24mm/hr

Table4.6: 24hrs incremental rainfall for Adama station 50 years return period daily maximum rainfall

Time	Hourly Distributed Cumulative P P = M * sqrt (Ti)	Incremental depth	Time interval	Precipitation
hr	mm	mm	hr	mm
1	22.24	22.24	0-1	2.29
2	31.45	9.21	1-2	2.40
3	38.52	7.07	2-3	2.52
4	44.48	5.96	3-4	2.66
5	49.73	5.25	4-5	2.83
6	54.48	4.75	5-6	3.03
7	58.85	4.37	6-7	3.28
8	62.91	4.06	7-8	3.61
9	66.72	3.82	8-9	4.37
10	70.33	3.61	9-10	5.25
11	73.77	3.43	10-11	7.07
12	77.05	3.28	11-12	22.24
13	80.19	3.15	12-13	9.21
14	83.22	3.03	13-14	5.96
15	86.14	2.92	14-15	4.75
16	88.97	2.83	15-16	4.06
17	91.70	2.74	16-17	3.82
18	94.36	2.66	17-18	3.43
19	96.95	2.59	18-19	3.15
20	99.47	2.52	19-20	2.92
21	101.92	2.46	20-21	2.74
22	104.32	2.40	21-22	2.59
23	106.67	2.34	22-23	2.46
24	108.96	2.29	23-24	2.34

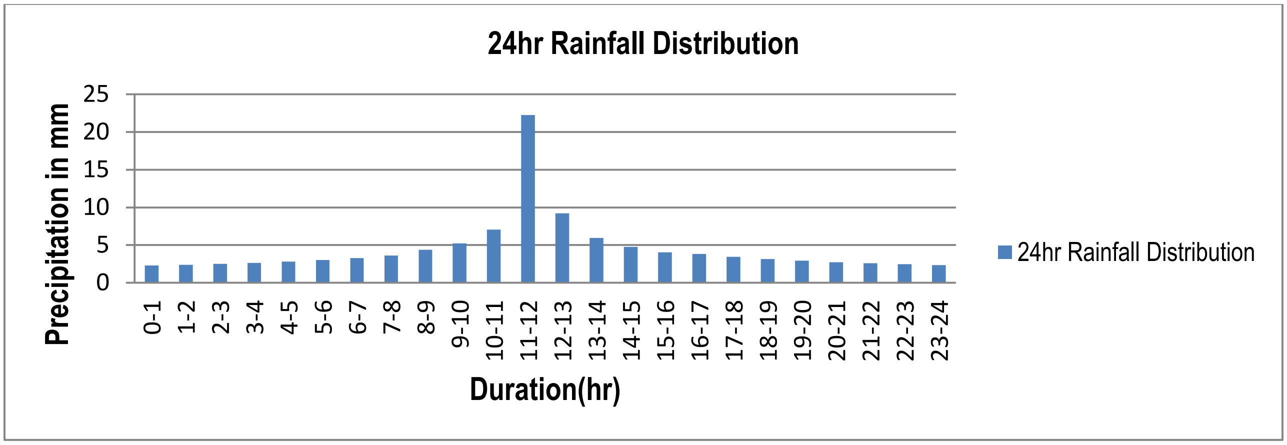


Fig 4.3: Adama 24hr Rainfall distribution

b. Adopting the Annual Maximum Daily Rainfall as 6 hours Duration Storm

The 50 years return period daily maximum rainfall, 108.96mm, is changed to 6hrs incremental rainfall.

Using, p = M * square root (T), ► M = p / square root (T), ► M = **44.48**mm/hr

Table 4.7: 6hrs incremental rainfall for Adama station 50 years return period daily maximum rainfall

Time	Hourly Distributed Cumulative P P = M * sqrt (Ti)	Incremental depth	Time interval	Precipitation
hr	mm	mm	hr	mm
1	44.48	44.48	0-1	10.50
2	62.91	18.43	1-2	14.14
3	77.05	14.14	2-3	44.48
4	88.97	11.92	3-4	18.43
5	99.47	10.50	4-5	11.92
6	108.96	9.49	5-6	9.49

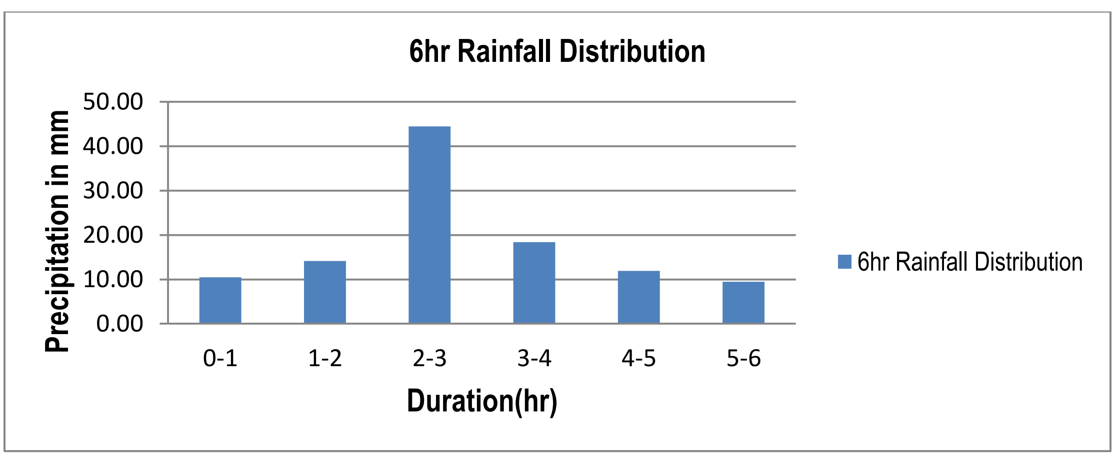


Fig 4.4: Adama 6hr Rainfall distribution

C. Using ERA Intensity -duration-frequency curve

ERA has developed rainfall intensity-duration curves by dividing the country into different hydrological regions as shown below.

Table4.8: ERA Hydrological Regions Meteorological Stations(Yr of Record via 2010)

Meteorological region	Station	Yr of Record	Meteorological region	Station	Yr of Record
	Axum	17		Bedele	39
	Mekele	46		Gore	56
A1	Maychew	32		Nekemte	40
	Gonder	52		Jimma	54
	Debretabor	15		Arbaminch	23
	Bahir Dar	45		Sodo	49
	DebreMarkos	55	B	Hawasa	37
	Fiche	44		Kombolcha	56
	Addis Ababa	57		Woldya	29
A2	Debrezeit	55	C	Sirinka	27
	Nazreth	46		Gode	33
	Kulumsa	43		Kebridihar	40
A3	Robe/Bale	29	D1	Kibremengist	33
	Metehara	24		Negele	51
	Diredawa	58		Moyale	29
A4	Meiso	42	D2	Yabelo	34

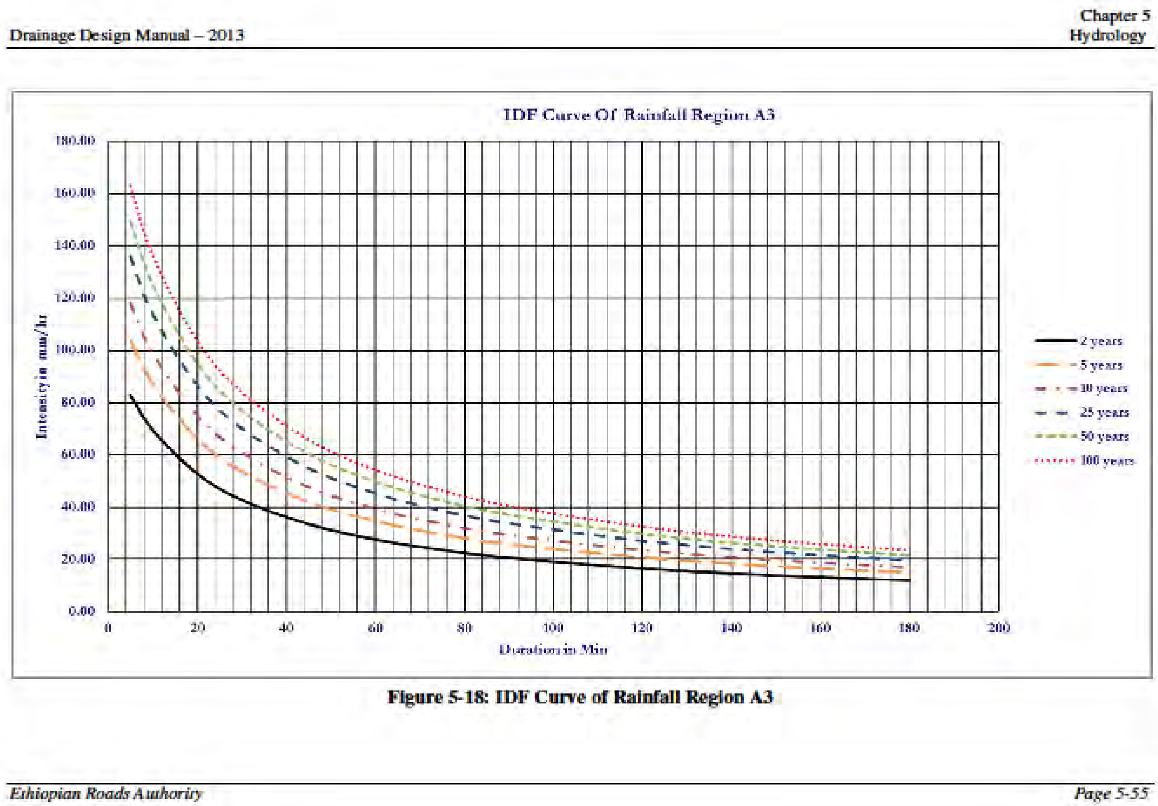


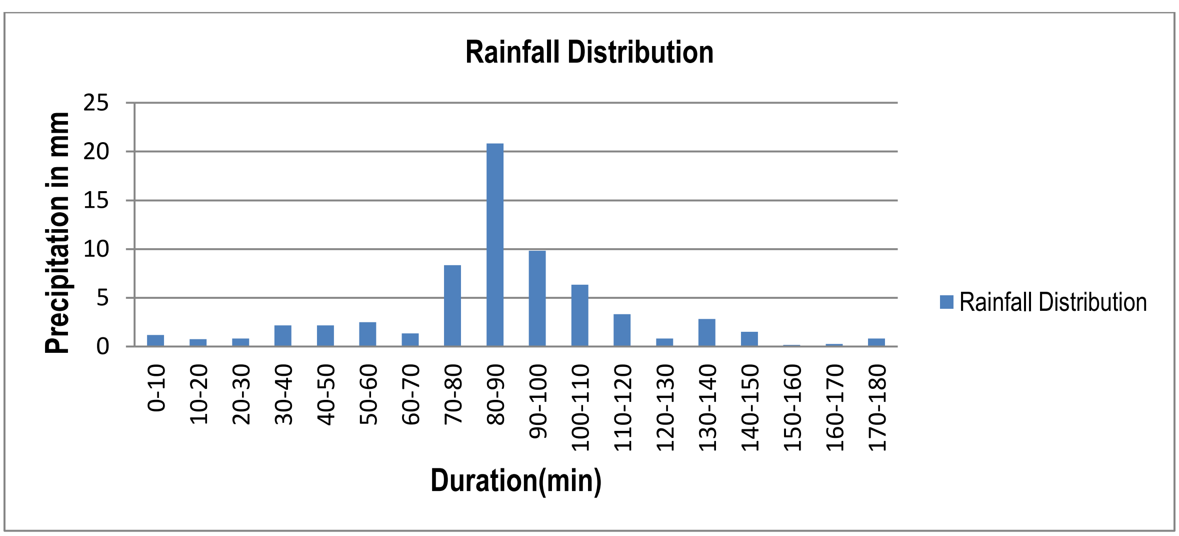
Figure 4.5: ERA drainage manual rainfall intensity-duration curves

The hyetograph produced from the ERA IDF curves using the alternate block method was as shown below.

Table4.9: Hyetograph using alternate block method for T =50yrs

Duration	Intensity	Cumulative Depth P= I*time	Incremental Depth	Time	Precipitation
Min	mm/hr	mm	mm	Min	mm
10	125	20.83	20.83	0-10	1.17
20	92	30.67	9.84	10-20	0.75
30	78	39.00	8.33	20-30	0.83
40	68	45.33	6.33	30-40	2.17
50	56	46.67	1.34	40-50	2.17
60	50	50.00	3.33	50-60	2.50
70	45	52.50	2.50	60-70	1.34
80	40	53.33	0.83	70-80	8.33
90	37	55.50	2.17	80-90	20.83
100	35	58.33	2.83	90-100	9.84
110	33	60.50	2.17	100-110	6.33
120	31	62.00	1.50	110-120	3.33
130	29	62.83	0.83	120-130	0.83
140	27	63.00	0.17	130-140	2.83
150	25.5	63.75	0.75	140-150	1.50
160	24	64.00	0.25	150-160	0.17
170	23	65.17	1.17	160-170	0.25
180	22	66.00	0.83	170-180	0.83

Figure 4.6: Adama Rainfall distribution using ERA IDF



4.3 Flood estimation

As it is described in section 3.2, the SCS method with the help of suitable Computer Programs ArcGIS and SCS excel models used to facilitate the calculations. The flood of the two catchments Golbategene and Kersa are computed as follows:

For Golbategene Catchment

Table 4.10: Land use of the Catchment area and CN Value

Land use/cover class	Area (Km2)	Percent
Cultivated and managed area	0.73	55.28
Cropland / shrub and/ or Grass	0.59	44.72
Total	1.3223	100.0

Sub-watershed area (km2)	Land Cover	Hydrologic Soil Group	CN
0.73	cultivated and managed area	B	76.00
0.59	Cropland / shrub and/ or Grass	B	56.00

	Area by Soil Type (Km2)	Catchment Area (Km2)	Land Cover	Hydrologic Soil Group	CN by soil type	Catchment CN
GolbaTegenCatchmt	0.731	1.322	cultivated and managed area	B	76.00	67.06
	0.591	1.322	Cropland / shrub and/ or Grass	B	56.00	

- Golbategene catchment estimated CN = 67, but need the antecedent moisture content (AMC) consideration.

The Time of concentration

The time of concentration computed as follows

Table 4.11Topography data

Elevation, m	Distance, m	Distance Diff.	Ele. Diff.	Slope
1744	0	0	0	0.00000
1731	290.9	290.9	13	0.04469
1726	474	183.1	5	0.02731
1704	764	290	22	0.07586
1686	1195	431	18	0.04176
1675	1441	246	11	0.04472
1668	1668	227	7	0.03084
1666	1874	206	2	0.00971
1664	2007	133	2	0.01504

The time of concentration computation recommended by SCS method related with length (L) & Slope(S) of the given catchment as:

$$T_c = (1/3000) * (L/S^{1/2})^{0.77} \ldots\ldots\ldots\ldots\ldots\ldots \text{(13)}$$

Table 4.12 Time of concentration for GT

Length, L(m)	**Slope, S**	Time of Concentration, T_c(hr)
0.0	0.00000	0.000
290.9	0.04469	0.087
183.1	0.02731	0.074
290.0	0.07586	0.071
431.0	0.04176	0.121
246.0	0.04472	0.076
227.0	0.03084	0.083
206.0	0.00971	0.120
133.0	0.01504	0.072
L=2007m		T_c= 0.704hr

Based on the above GIS date and computation, the probable maximum flood to the area was computed using the SCS excel model as shown in the following steps:

Table 4.13 Peak Discharge Estimation

Step	Parameter	Unit	Value
1	Catchment Area	Km^2	1.322
2	Length of main river	m	2007.00
3	Time of concentration, Tc	hr	0.704
4	Rain fall excess duration D = Tc/6	hr	0.117
5	Time to peak, Tp Tp = 0.6 Tc + 0.5 D	hr	0.481
6	Time to base, Tb Tb = 2.67 Tp	hr	1.285
7	Peak rate of discharge created by 1mm runoff excess on whole of the catchment, Tp p = (0.278* A) / Tp	m^3/sec/mm	0.764
8	Lag time, te Te = 0.6 Tc	hr	0.42

Read from tables (in the appendix) based on step9

1.32 Km^2	
Duration	Arial to pt. RF ratio
0.12	97.90
0.23	98.03
0.35	98.15
0.47	98.28
0.59	98.40
0.70	98.52

Duration	Rainfall Ratio
Hr	%
0.12	**18.21**
0.23	**23.54**
0.35	**28.86**
0.47	**31.89**
0.59	**35.78**
0.70	**39.54**

9	10	11		12	13	14	15	16
Duration	**Daily point rainfall(100 yr Return per.)**	**Rainfall profile**		**Areal to point R.F ratio**	**Areal-Rainfall**	**Incremental rainfall**	**Descending order**	**Descending order**
hr	**mm**	**%**	**mm**	**%**		**mm**	**mm**	**No.**
0 - 0.117		18.21	21.67	97.90	21.22	21.22	21.22	1
0.117 - 0.235		23.54	28.01	98.03	27.46	6.24	6.25	2
0.235 - 0.352	119.00	28.86	34.34	98.15	33.71	6.25	6.24	3
0.352 - 0.470		31.89	37.95	98.28	37.30	3.59	4.60	4
0.470 - 0.587		35.78	42.58	98.40	41.90	4.60	4.46	5
0.587 - 0.704		39.54	47.05	98.52	46.36	4.46	3.59	6

17	18	19	20	21	22	
Rearranged order	Rearranged incremental R.F	Cumulative rainfall	Time of incremental hydrograph			
			Time of beginning	Time to peak	Time to end	
No.	mm	mm	hr	hr	hr	
6	3.59	3.5896	0	0.48	1.29	
4	4.60	8.1928	0.12	0.60	1.40	
3	6.24	14.4315	0.23	0.72	1.52	
1	21.22	35.6495	0.35	0.83	1.64	
2	6.25	41.8990	0.47	0.95	1.75	
5	4.46	46.3559	0.59	1.07	1.87	

23	24	25	26	27	28	29	30	31	32	
Land use or cover	**Area ratio**	**Land treatment practice**	**Hydrologic Soil group**	**Soil Family**	**Textural Class**	**Stream Flow Potential**	**Curve No. CN'**	**Weighted "CN"**	**Sum weighted "CN"**	
cultivated and managed area	0.5528351		B				76.00	42.02	AMC	CN
Cropland / shrub and/ or Grass	0.4471649		B				56.00	25.04		
								0		
								0		
								0		
									III	83.06
								67.06		

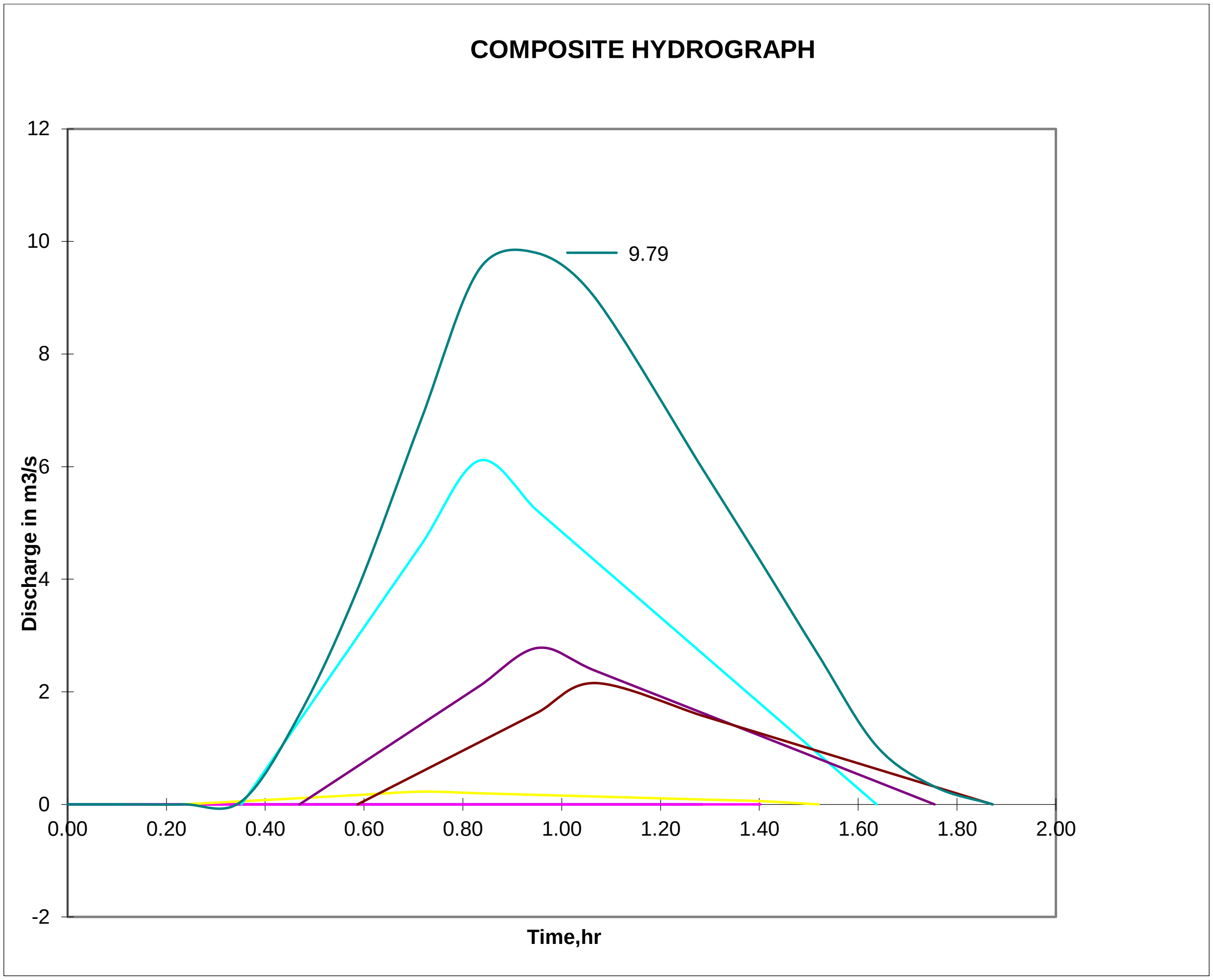
S can then be computed using the AMC of CN 83.06.

33			
The maximum potential difference b/n Rainfall (p) and direct runoff (Q)	mm		
$S = \frac{25400}{CN} - 254$, CN = corresponding to AMC III	mm	51.82	
		19	33
$Q = \frac{(p - 0.2S)^2}{(p + 0.8S)}$	mm	p (mm)	Q(mm)
		3.5896	0.00
		8.1928	0.00
		14.4315	0.30
		35.6495	8.29
-		41.8990	11.93
		46.3559	14.75

34	35	36	37	38		
Duration	Cumulative run off	Incremental run off	Peak run off incremental	Time of begin	Time to peak	Time to end
hrs	mm	mm	m^3/sec	hrs		
0 - 0.117	0.00	0	0.00	0.000	0.481	1.285
0.117 - 0.235	0.00	0.00	0.00	0.117	0.599	1.402
0.235 - 0.352	0.30	0.30	0.23	0.235	0.716	1.520
0.352 - 0.470	8.29	8.00	6.11	0.352	0.833	1.637
0.470 - 0.587	11.93	3.64	2.78	0.470	0.951	1.755
0.587 - 0.704	14.75	2.82	2.16	0.587	1.068	1.872

39							
Time	Ordinate of Hydrograph (m³/Sec.)						
(hr)	1	2	3	4	5	6	Total
0.00	0						0.00
0.12	0	0					0.00
0.23	0	0.00	0				0.00
0.35	0	0.00	0.06	0			0.06
0.47	0.00	0.00	0.11	1.49	0		1.60
0.59	0	0.00	0.17	2.98	0.68	0	3.82
0.72	0	0.00	0.23	4.62	1.42	0.58	6.85
0.83	0	0.00	0.20	6.11	2.10	1.10	9.51
0.95	0	0.00	0.17	5.22	2.78	1.63	**9.79**
1.07	0	0.00	0.14	4.32	2.37	2.16	8.99
1.29	0	0.00	0.09	2.68	1.62	1.57	5.96
1.40		0	0.06	1.78	1.22	1.26	4.32
1.52			0	0.89	0.81	0.94	2.65
1.64				0	0.41	0.63	1.04
1.75					0	0.31	0.31
1.87						0	0.00

Figure 4.7: Composite Hydrograph



Peak discharges for different return periods can be computed in same way

Return Period,T	2	5	10	25	50	100
Max RF in mm	57	74	85	99	109	119
Q in m^3/sec	1.47	3.23	4.63	6.62	8.16	9.79

Similarly for Kersa catchment

Tables 4.14: Kersa Land use of the Catchment area

Land use/cover class	Area (Km2)	Percent
cultivated and managed area - Agriculture	0.25	100.00
Total	0	100.0

Sub-watershed area (km2)	Land Cover	Hydrologic Soil Group	CN
0.25	cultivated and managed area - Agriculture	B	76.00

	Area by Soil Type (Km2)	Catchment Area (Km2)	Land Cover	Hydrologic Soil Group	CN by soil type	Catchment CN
Kersa Stream	0.25	0.25	cultivated and managed area - Agriculture	B	76.00	76.00

For the time of concentration;

Table 4.15 Kersa Topography Data

Ele	Dist. M	Dist.Diff, m	Elev Diff. m	Slope
1721	0	0	0	0.0000
1716	116	116	5	0.0431
1712	236	120	4	0.0333
1702	362	126	10	0.0794
1697	463	101	5	0.0495
1690	642	179	7	0.0391
1684	709	67	6	0.0896
1682	811	102	2	0.0196

Equation (13) is used similarly.

Table 4.16 Time of concentration for Kersa

Length(m)	Slope	Time of Concentration
0.0	0.00000	0.000
116.0	0.04310	0.043
120.0	0.03333	0.049
126.0	0.07937	0.037
101.0	0.04950	0.037
179.0	0.03911	0.063
67.0	0.08955	0.021
102.0	0.01961	0.053
811.0		0,304

Table 4.17 Kersa T_C and Lag time

Step	Parameter	Unit	Value
1	Catchment Area	Km^2	0.255
2	Length of main river	m	811.00
3	Time of concentration, Tc	hr	0.304
4	Rain fall excess duration D = Tc/6	hr	0.051
5	Time to peak, Tp Tp = 0.6 Tc + 0.5 D	hr	0.208
6	Time to base, Tb Tb = 2.67 Tp	hr	0.555
7	Peak rate of discharge created by 1mm runoff excess on whole of the catchment, Tp p = (0.278* A) / Tp	m^3/sec/mm	0.341
8	Lag time, te Te = 0.6 Tc	hr	0.18

Computed peak discharge for different return periods;

For Kersa

Return Period, T in years	2	5	10	25	50	100
Max RF in mm	57	74	85	99	109	119
Q in m3/sec	0.58	1.19	1.65	2.31	2.81	3.34

5. Detention Pond/Dam

Detention ponds/dams are constructed to retard flood runoff and minimize the effect of sudden floods. It falls into two main types. In the more common type, the water is temporarily stored and released through an outlet structure at a rate that will not exceed the carrying capacity of the channel downstream. In other types, the water is held as long as possible and, during the growing season, released through a gated outlet and travels through a dike system that irrigates the vegetation within the dike system.

In the event of failure, detention dams would cause only minor incremental damage over-and above the imminent failure flow to the stream channel downstream. They do not threaten buildings, streets, prime agricultural land or people's lives. Pond/dam controls the discharge of surface water flowing into or down a 'dry watercourse' in a storm so the discharge can be carried by the stream without overflowing and significant bank erosion or bed scour.

There are several types of detention devices, the most common being the dry detention basin and the extended dry detention basin. These are structures which hold a certain amount of water from a storm and which release the water through a controlled outlet over a specified time period based on design criteria.

5.1 Input Data

The input data for a flood protection design are, generally:

• Description of the place (city/town) (location, number of inhabitants, monuments, relics, nature reserves, springs, wetlands, etc.). (fig3.00, fig5.2, fig5.3)

• Character of the site (description of the catchment, land use, inclinations, length of the stream, objects on the stream, etc.), (fig3.3, fig5.4, table4.14)

• map data (fig1.1, fig5.3)

• Hydrologic data (rainfall, runoff and temperature data. The design flood wave was calculated from information about the catchment, land use, temperatures, rainfall and runoff. (In the Appendix)

• Geological, pedologic and morphological data (information about the bedrock, landslides, types of soil, etc.)(Table 3.1)

• Geodetic data (information about existing geodetic points, topographical survey of the area) (Table3.1)

• Hydraulic data (roughness of the stream, slope, capacity of any structures) (Table 3.1)

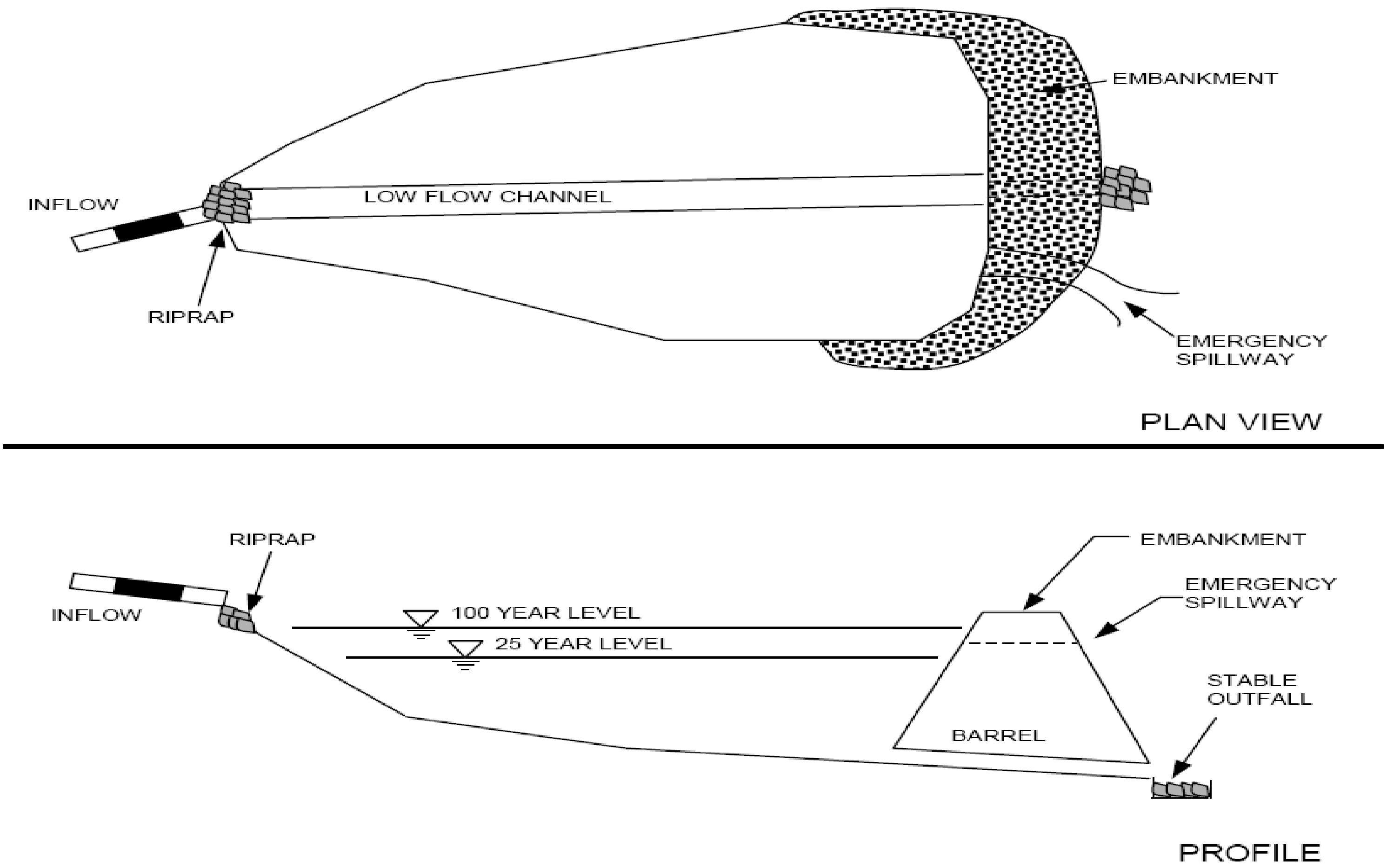


Figure5.1: Schematic of dry detention basin (Part2g.pdf, Iowa; 2G-1)

5.2 Design Criteria

The basic design parameters for a detention dam are its storage volume and detention time. A detention dam must have the correct combination of storage volume and outflow capacity to contain and slowly discharge the design runoff volume over a prescribed period of time. Details of these and other design parameters are presented below.

The design criteria for the storage facilities include:

- Release rate,
- Storage volume,
- Depth requirements,
- Safety considerations and landscaping,
- Outlet works, and location.

5.2.1 Release Rate

Control structure release rates shall approximate pre-developed peak runoff rates for the 2-yearthrough 25-year storms, with emergency overflow capable of handling the 100-year discharge.

5.2.2 Storage

Storage volume shall be adequate to attenuate the post-development peak discharge rates to pre-developed discharge rates for the 2-year through 25-year storms. Routing calculations must be used to demonstrate that the storage volume is adequate.

5.2.3 Depth

A minimum freeboard of 0.30m above the 100-year design storm high water elevation shall be provided for impoundment depths of less than 6m. Impoundment depths greater than 6m are subject to the requirements of the safety consideration unless the facility is excavated to this depth.

Other considerations when setting depths include flood elevation requirements, public safety, land availability, land value, present and future land use, water table fluctuations, soil characteristics, maintenance requirements, and required freeboard. Aesthetically pleasing features are also important in urbanizing areas.

5.2.4 Safety Consideration

A dam is an artificial barrier that does or may impound water. It can be standardized considering different factors like in relation to its objective, site condition, etc. A number of exemptions are allowed from the safety consideration and any questions concerning a specific design or application should be addressed.

5.2.5 Outlet Works and Locations

Outlet works selected for storage facilities typically include a principal spillway and an emergency overflow, and must be able to accomplish the design functions of the facility. The principal spillway is intended to convey the design storm without allowing flow to enter an emergency outlet. The minimum flood to be used to size the emergency outlet is the 100-year flood. The sizing of a particular outlet works shall be based on results of

hydrologic routing calculations. If several storage facilities are located within a particular basin it is important to determine what results a particular facility may have on combined hydrographs in downstream locations.
The water outlet of every detention dam must be hydraulically solved and shape-designed so that:

• The discharge is automatically secured until the water level touches the spillway crest

• The flood discharge is safely carried away without overflowing the dam

• The outlet contributes to the transformation of the flood wave and a decrease of the culmination discharge

5.3 Design Considerations and Outputs

From the two catchments Kersa and GT, both have similar and different catchment characteristics that should be considered in the design. For Kersa, pond design was similar to the locally practiced pond construction which is like longitudinally lied dome shape though the outlet and spillway sections design should consider and balance the design inflow outflow. Design recommendation of the pond, outlet and spillway was as follows:

- Pond capacity required = 26421m^3

  (Where: Delineated pond area = 8584m^2 and average pond height = 3m)
- Bed level = 1682m
- Out let level = 1683m
- Safe channel discharge = 0.78m^3/s

  Safe channel discharge computations were estimated by direct measurements of the existing channels (Gully) sizes and taking its hydraulic parameters.

For Kersa
A trapezoidal Channel

V= 1/n *R^3/2 *S^1/2 ….. (Manning's equation)
A= b_1*Y + my^2
P= b_1 + 2*Y*(1+m^2)^0.5
R= A/P
Q= A*V
Given
n= 0.023
S= 0.01
m= 1.5
Y= 0.5m
b1 = 1.2m
Result
P= 3.00m
A= 0.975m^2

R= 0.32m
V= 0.80m/s
Q= $0.78m^3/s$

- Rectangular Sluice way size; B = 1m, h=1m; Emergency Spillway; B=1m, L=3m, h = 0.5m
- Max height of the pond with FB (height of the weir crest) =3m

For GT, a standard dam design needs to be considered and basic design parameters are mentioned below:

- **Study area aerial photograph**

Figure 5.2: GT study area aerial photograph

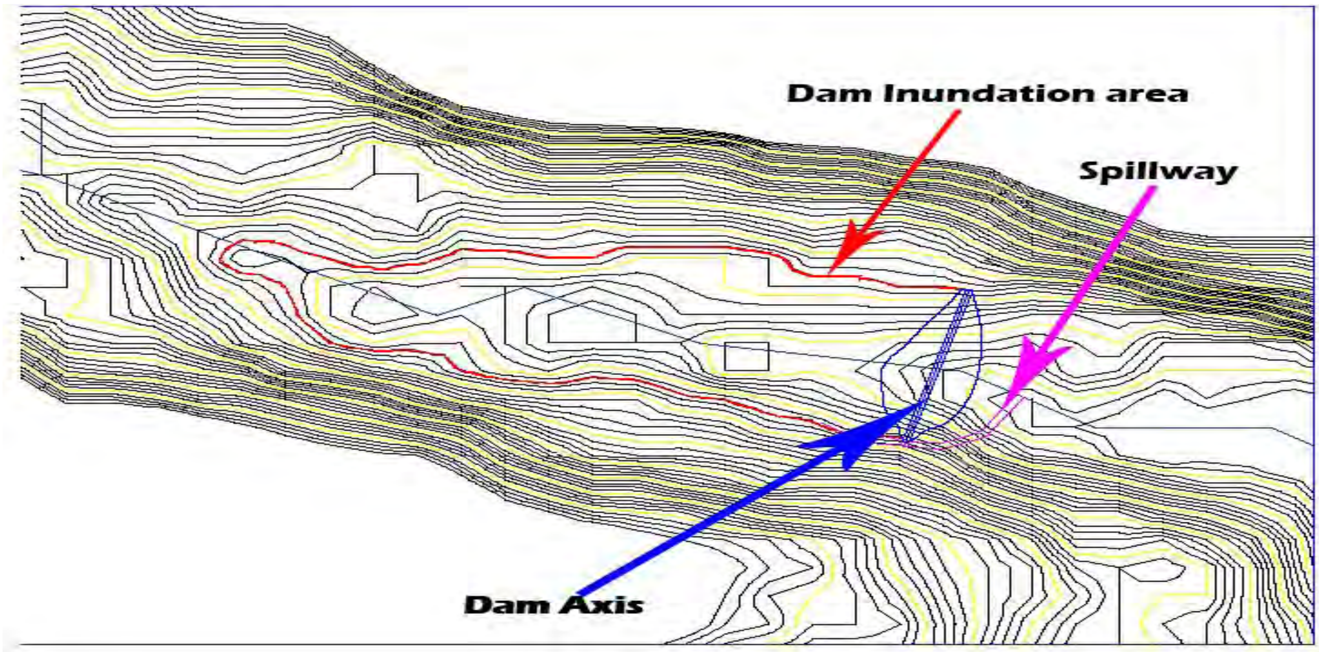


Figure 5.3: Golbategene pond/dam site

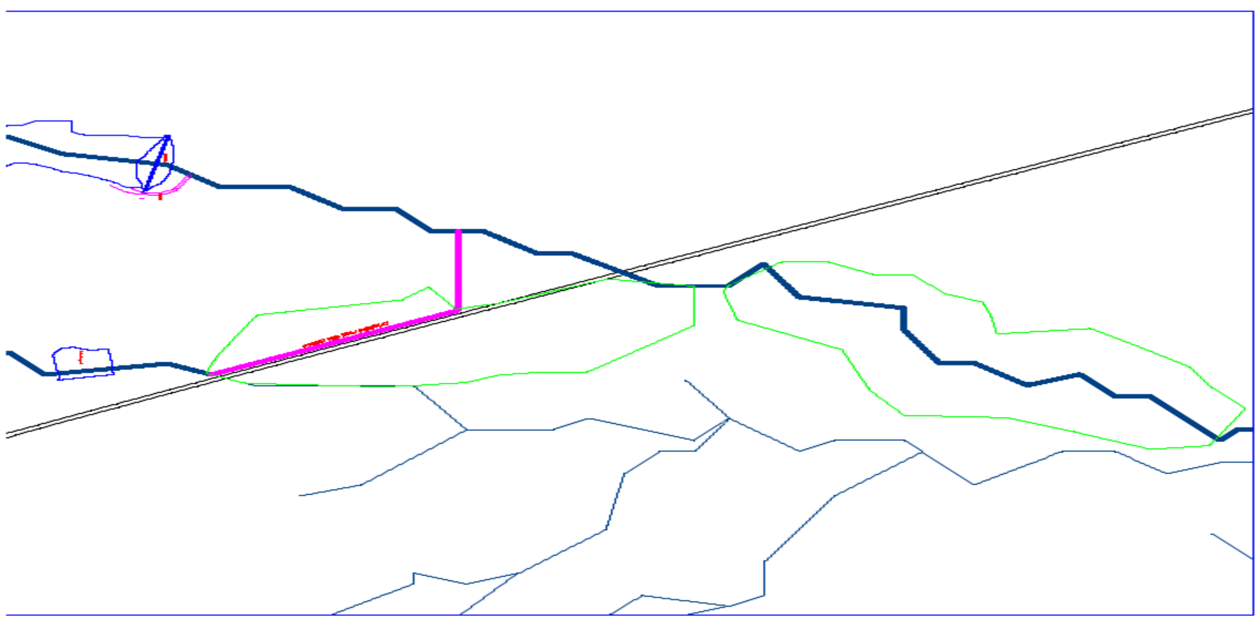


Figure 5.4: GolbaTegene & Kersa pond/dam layout (in blue) and flood vulnerable areas (in green); artificial drainage channel (in pink)

- Bottom level of the dam = 1662m
- Crest length =158m
- Crest width = 5m
- Surface areas and calculated reservoir capacities

Table 5.1: GT Reservoir Data

	Level(m)	Height y(m)	Area(m)	Volume(m)
	1662	0	0	0
	1663	1	797	398.50
	1664	2	2441	2,017.50
	1665	3	7898	7,187.00
	1666	4	13292	17,782.00
	1667	5	19600	34,228.00
	1668	6	25966	57,011.00
	1669	7	37658	88,823.00
	1670	8	46620	130,962.00

Table5.2: Catchment Characteristics For GT

	Catchment Area A(km^2)	1.32	
	Highest Point in Catchment RL(m)	1744	
	Height at Catchment Outlet RLs (m)	1662	
	Max. Height Difference H_{max} (m)	82	RL-RLs
	Longest Watercourse L (km)	2.007	
	Average Slope Sa	0.03	
	Lag Time t_L (hr)	0.42	
	Time of concentration tc (hr)	0.704	

Stage-storage curve and Stage-discharge curve for the proposed Golbategene storage facility

Table 5.3: Stage-Volume curve

Stage in m	Volume, m3
1662	0.00
1663	398.50
1664	2017.50
1665	7187.00
1666	17782.00
1667	34228.00
1668	57011.00
1669	88823.00
1670	130962.00
1671	181833.50
1672	241364.50
1673	309415.00
1674	385678.50
1675	469861.00

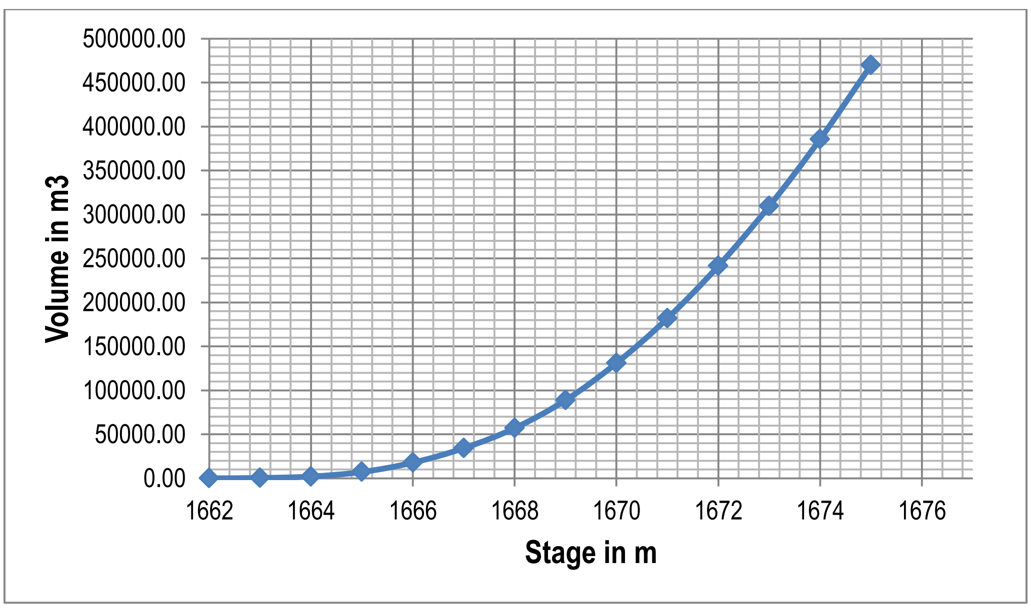


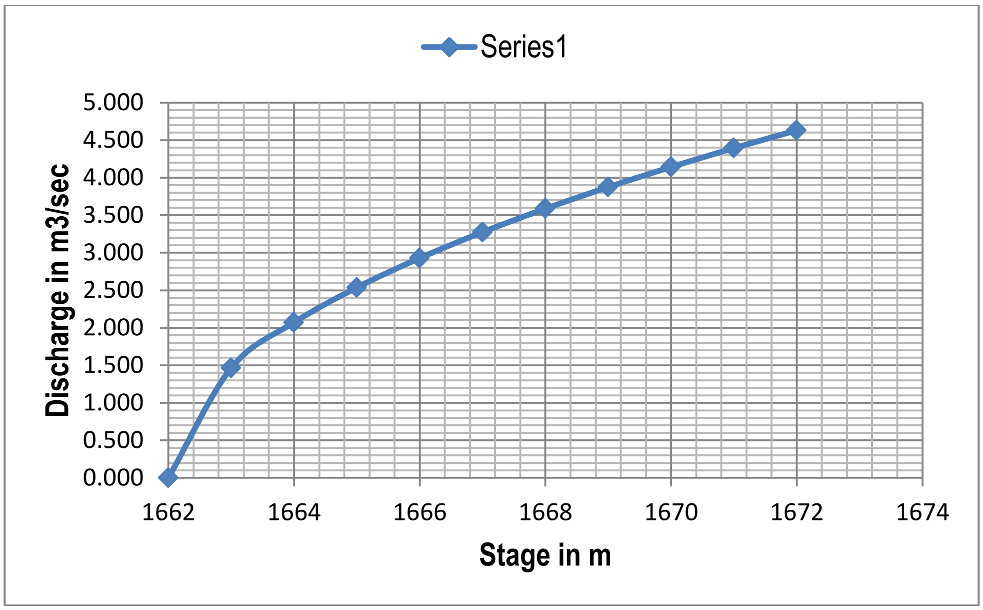
Figure 5.5: Stage-Volume curve

- Q from kersa weir out =0.78m³/s
- Q spill from GT pond/dam = 0.738m³/s (by reservoir routing)
- Q at vulnerable areas = 5.66m³/s (the existing channel/gully size computed)
- Q at Dam outlet = 4.142m³/s
- Velocity, V = 12.59m/s and then Area outlet pipe = 0.329m²
- Pipe diameter = 0.65m
- Q Safe at vulnerable areas = 6.36m³/s (modifying the existing channel size)

Table 5.4: Stage-discharge curve

Stage in m	Height (m)	Outflow Q in m3/sec
1672	10	4.631
1671	9	4.393
1670	8	4.142
1669	7	3.874
1668	6	3.587
1667	5	3.275
1666	4	2.929
1665	3	2.536
1664	2	2.071
1663	1	1.464
1662	0	0.000

Figure 5.6: Stage-Discharge Curve



Design Parameters

Design flood = 25 years flood

Levels:

Bottom Level = 1662m

Out let Level = 1663m

Overflow Crust Level =1670m

Non-Overflow Crust Level =1671m

Outlet Pipe Diameter = 0.65m

Spillway crest level& length: 1670m& 5m

Pond/Dam and Spillway Routing for GT

Table: 5.5 log area & volume Curve

VOLUME: MCM	Log of Area	Log of Volume		***Reservoir Area*** *(1000) m2*	***VOLUME:*** (1000) m3
0.000	0	0.00		0.00	0.00
0.000	0.00	-3.40		0.80	0.40
0.002	0.30	-2.70		2.44	2.02
0.007	0.48	-2.14		7.90	7.19
0.018	0.60	-1.75		13.29	17.78
0.034	0.70	-1.47		19.60	34.23
0.057	0.78	-1.24		25.97	57.01
0.089	0.85	-1.05		37.66	88.82
0.131	0.90	-0.88		46.62	130.96
0.182	0.95	-0.74		55.12	181.83
0.241	1.00	-0.62		63.94	241.36
0.309	1.04	-0.51		72.16	309.42
0.386	1.08	-0.41		80.37	385.68
0.470	1.11	-0.33		88.00	469.86
0.562	1.15	-0.25		96.80	562.26
0.663	1.18	-0.18		105.50	663.41

Fig 5.7 Reservoir area-Capacity

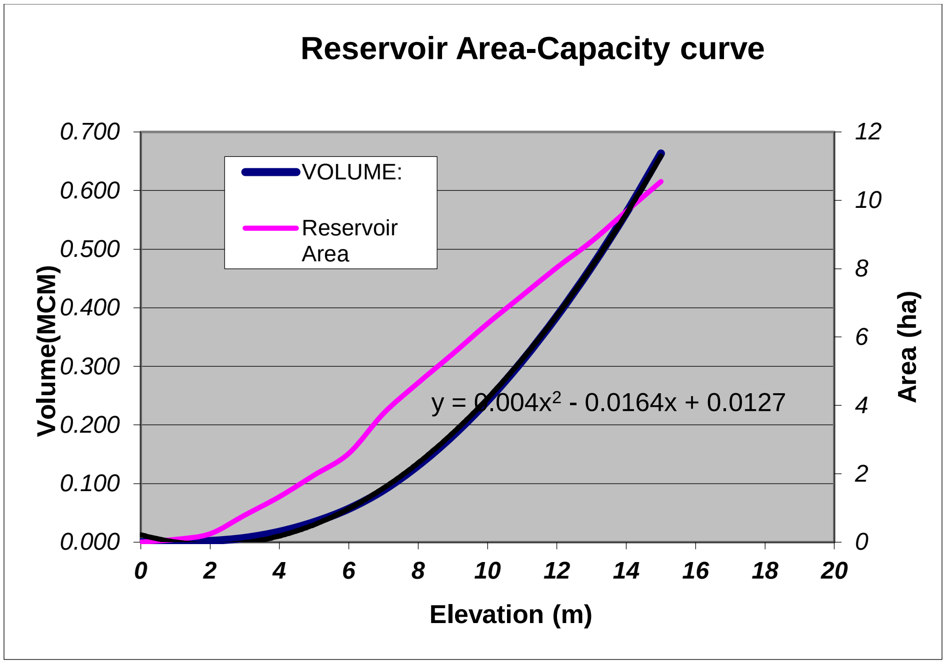


Fig 5.8 Elevation-Area-Volume Curve

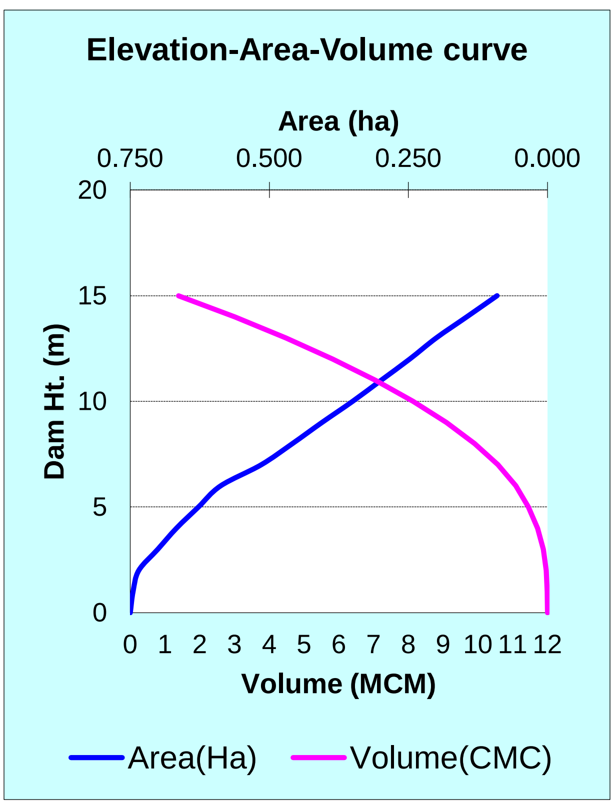
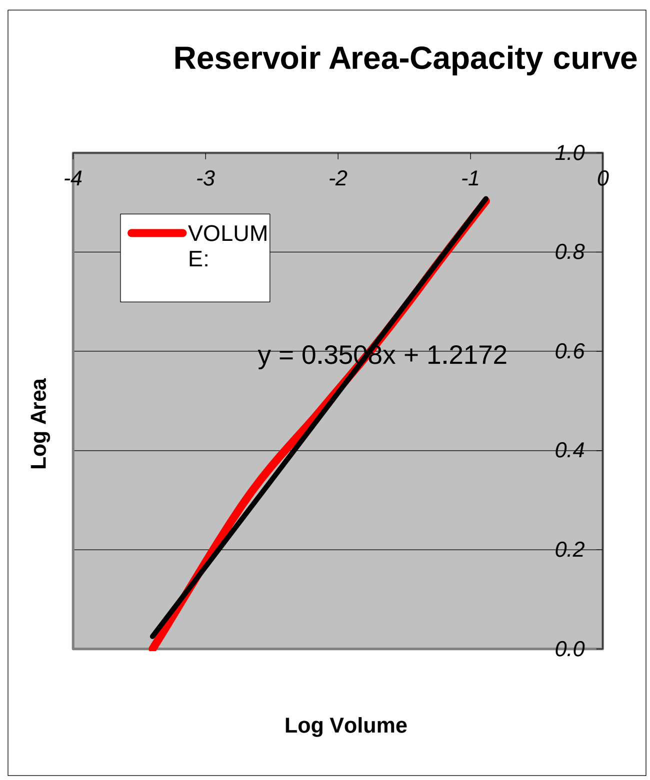


Fig 5.9 A-V log curve



Muskingum Method

The Muskingum method was selected due to its preference when there is no gauged data and if the flood will go out of bank, into floodplain which is similar to Boku shenen case. This routing method is based on the combination of the conservation of mass and the diffusion representation of the conservation of momentum. It is sometimes referred to as a variable coefficient method because the routing parameters are recalculated every time step based on channel properties and the flow depth.

A flood hydrogaph when propagating through a river reach is subjected to deformation. If lateral and tributary inflow can be neglected, comparision of an upstream flood hydrogaph with the related downstream hydrograph, the later will show a reduced peak flow and increase time base.

Muskingum developed a method for routing of floods which is based on the assumption of wedge storage in the river reach. The storage S is as follows defined as a function of the inflow I and the outflow Q in the considered reach of the river

$$\mathbf{S = K\,[xI+(1-x)Q]} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(13)$$

the Muskinggum formula the parameter x is a dimensiionless weight factor indicating the relative importance of I and Q in the determining the storage in the each. The value of x is limited between 0 and 0.5. the parameter K has the dimension of time. Both K and x are constans for a cetain river reach. Neglecting lateral inflow, K and x can determined if inpute and output hydrographs of the river reach ae known.

In the below spillway routing, it was basically analyzed by considering risky conditions using the Muskingum linear relationship function of storage for inflow and outflow i.e. assuming that what if the dam/pond main out let might be clogged during the peak flood flow occurred.

Table: 5.6 Spillway routing - Muskingum Method

Level spillway crest =	**8**
L =	**5**

Maximum water level	**8.83**
Maximum water depth	**0.83**

Time, t hr	Inflow m3/s	Outflow, Q* m3/s	V* MCM	H* m	Q** m3/s	V** MCM	H** m	Q m3/s	V MCM	H m
0.000	0.0								0.1310	8.00
0.117	0.010	0.000	0.131	8.091	0.073	0.131	8.089	0.071	0.1309	8.089
0.235	0.010	0.200	0.131	8.084	0.191	0.131	8.084	0.191	0.1306	8.084
0.352	0.055	0.182	0.130	8.080	0.176	0.130	8.080	0.177	0.1305	8.080
0.470	1.600	0.171	0.132	8.122	0.241	0.132	8.120	0.238	0.1323	8.120
0.587	3.823	0.312	0.137	8.221	0.527	0.137	8.215	0.513	0.1368	8.215
0.716	6.846	0.748	0.145	8.385	1.232	0.144	8.372	1.191	0.1444	8.373
0.833	9.510	1.706	0.155	8.582	2.472	0.154	8.561	2.394	0.1540	8.564
0.951	9.793	3.173	0.163	8.734	3.920	0.162	8.715	3.835	0.1621	8.717
1.068	8.992	4.558	0.168	8.829	5.099	0.167	8.815	5.032	0.1674	8.817
1.285	5.960	5.539	0.168	8.828	5.592	0.168	8.826	5.586	0.1679	8.826
1.402	4.319	5.634	0.166	8.794	5.467	0.166	8.798	5.488	0.1663	8.797
1.520	2.648	5.339	0.163	8.729	5.002	0.163	8.738	5.044	0.1631	8.737
1.637	1.035	4.745	0.158	8.642	4.294	0.159	8.654	4.349	0.1586	8.652
1.755	0.315	3.952	0.154	8.558	3.529	0.154	8.569	3.578	0.1542	8.567
1.872	0.000	3.206	0.150	8.483	2.853	0.150	8.492	2.892	0.1503	8.491
2.000	0.000	2.580	0.147	8.422	2.311	0.147	8.429	2.339	0.1472	8.428
3.000	0.000	2.101	0.144	8.371	1.894	0.145	8.377	1.914	0.1446	8.376
4.000	0.000	1.729	0.142	8.328	1.568	0.142	8.333	1.583	0.1424	8.332
5.000	0.000	1.438	0.141	8.293	1.310	0.141	8.296	1.321	0.1407	8.296
6.000	0.000	1.207	0.139	8.262	1.105	0.139	8.265	1.113	0.1392	8.265
7.000	0.000	1.021	0.138	8.236	0.939	0.138	8.238	0.946	0.1379	8.238
8.000	0.000	0.871	0.137	8.213	0.804	0.137	8.215	0.809	0.1368	8.215
9.000	0.000	0.748	0.136	8.194	0.694	0.136	8.195	0.698	0.1358	8.195
10.000	0.000	0.647	0.135	8.177	0.602	0.135	8.178	0.605	0.1350	8.178
11.000	0.000	0.563	0.134	8.162	0.525	0.134	8.163	0.528	0.1343	8.163
12.000	0.000	0.493	0.134	8.149	0.461	0.134	8.150	0.463	0.1337	8.150

13.000	0.000	0.434	0.133	8.137	0.407	0.133	8.138	0.409	0.1331	8.138
14.000	0.000	0.384	0.133	8.127	0.361	0.133	8.127	0.362	0.1326	8.127
15.000	0.000	0.341	0.132	8.117	0.321	0.132	8.118	0.322	0.1322	8.118
16.000	0.000	0.304	0.132	8.109	0.287	0.132	8.110	0.288	0.1318	8.110
17.000	0.000	0.272	0.131	8.102	0.258	0.131	8.102	0.258	0.1315	8.102
18.000	0.000	0.245	0.131	8.095	0.232	0.131	8.095	0.233	0.1312	8.095
19.000	0.000	0.221	0.131	8.089	0.210	0.131	8.089	0.210	0.1309	8.089
20.000	0.000	0.200	0.131	8.083	0.190	0.131	8.084	0.190	0.1306	8.084
21.000	0.000	0.181	0.130	8.078	0.173	0.130	8.079	0.173	0.1304	8.079
22.000	0.000	0.165	0.130	8.074	0.158	0.130	8.074	0.158	0.1302	8.074
23.000	0.000	0.151	0.130	8.070	0.144	0.130	8.070	0.144	0.1300	8.070
24.000	0.000	0.138	0.130	8.066	0.132	0.130	8.066	0.132	0.1298	8.066

Where:

Q*,Q** and Q = outflow trials

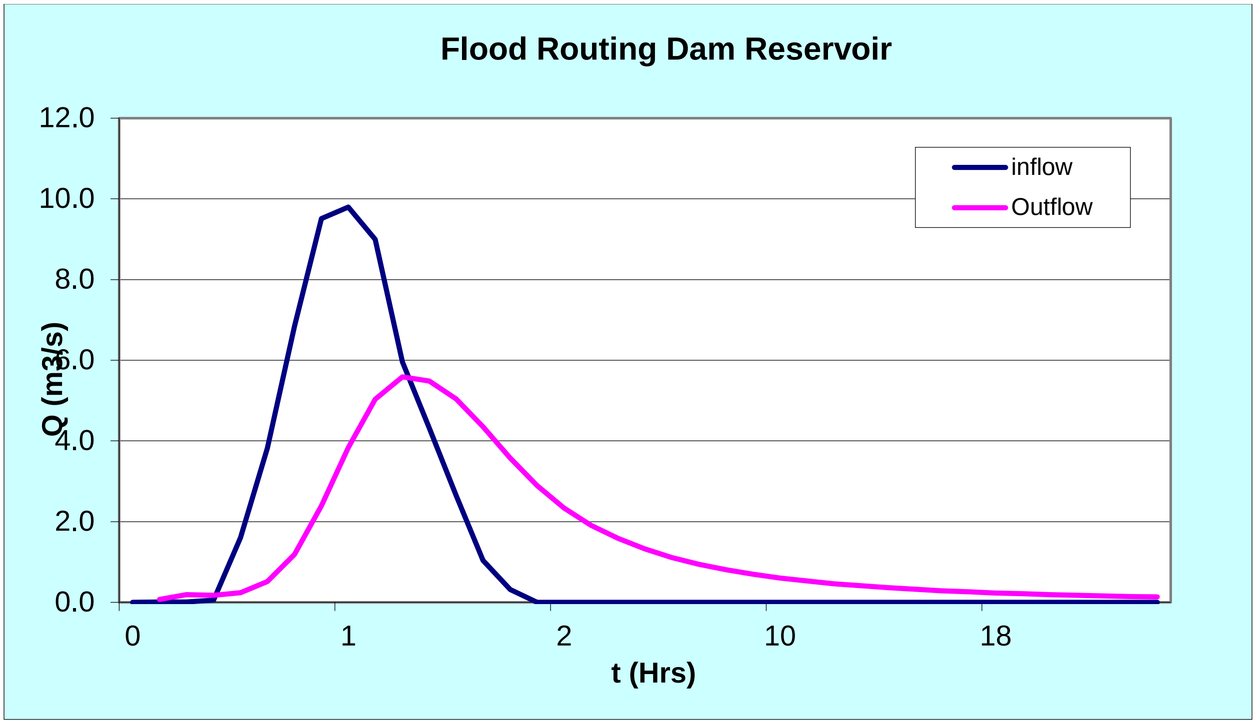
= $CLH^{3/2}$;in m^3/s

S*,S** and S = Volumes

= V+Q*t ;in million meter cubic(MCM)

H*,H** and H= Depths of water in the dam/pond

= 10^(0.35*log(S)+1.217) ; in meter



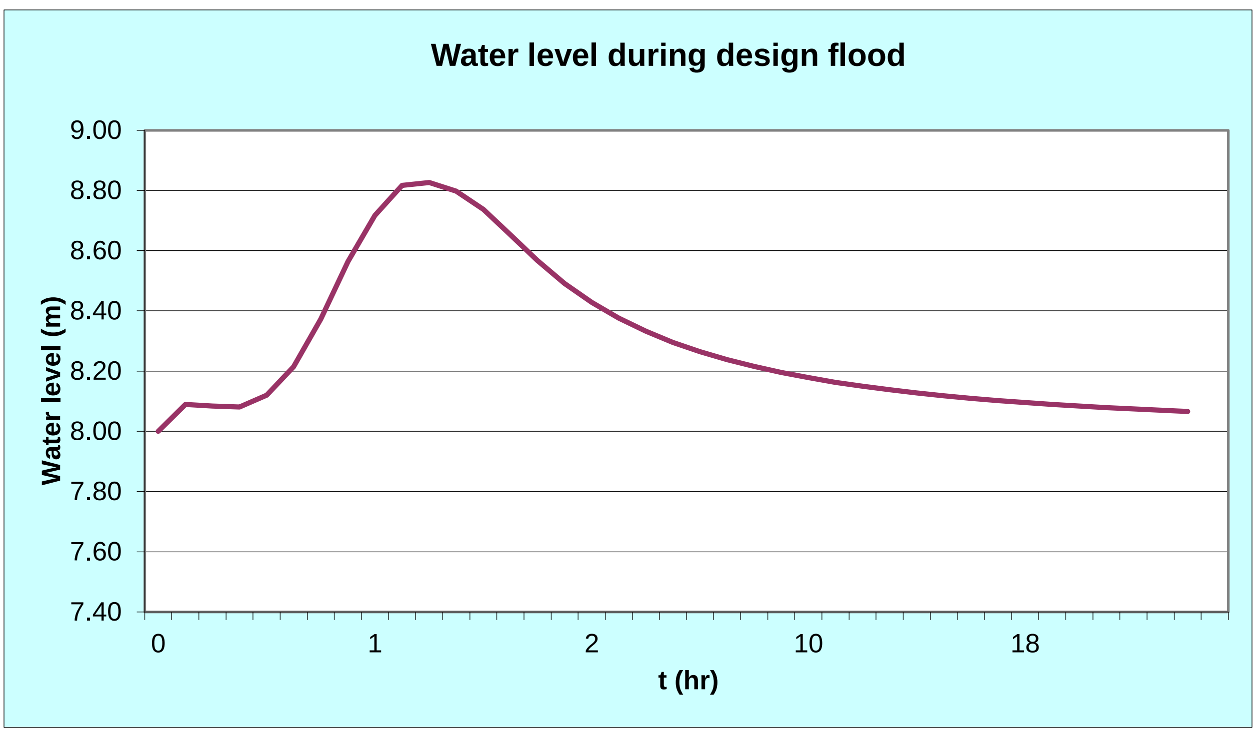


Fig 5.10 Flood Routing GT Pond/Dam

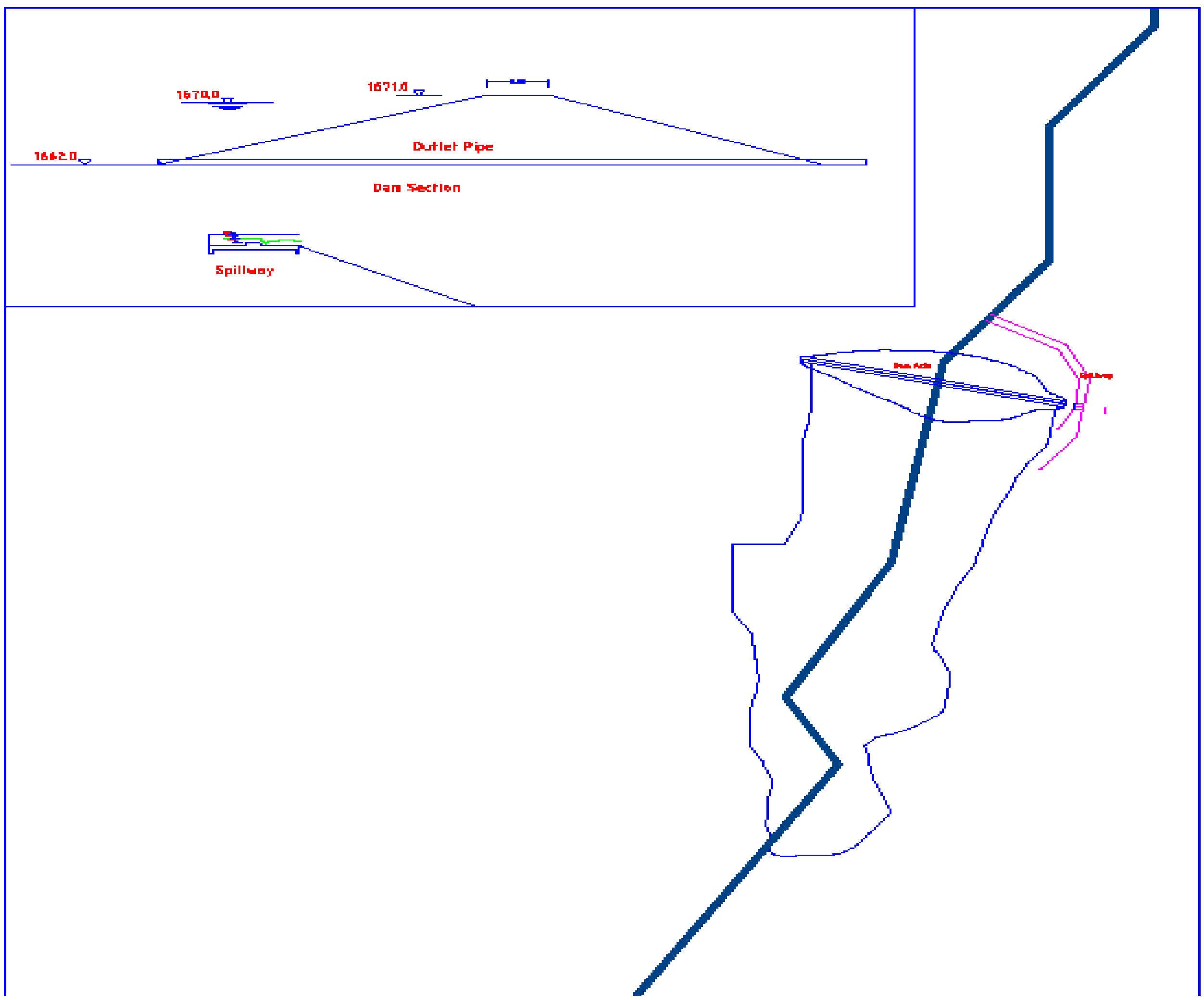


Figure 5.11 Cross section and layout of the dam/pond

6. Conclusions and Recommendations

6.1Conclusions

- The annual daily maximum rainfall data analyzed by the three goodness of fit test of different probability distributions and then Adama rainfall data fit Gumble's or extreme value type I distribution. Hence the probable maximum precipitation for different return periods computed and for the 100 years return period 119mm obtained.
- From the annual daily maximum rainfall the different years return period peak flood discharge estimated using SCS method and used for the design of structural flood preventive measures. The 25 and 100 years return period peak discharge 6.62m^3/s and 9.79m^3/s, respectively are used for the pond/dam design whereas for Kersa pond, it is 2.31m^3/s and 3.34m^3/s.
- Implementing reliable detention pond/dam is a viable alternative using onsite storm water controls and is able to meet the larger goals of economic and community development. Locally constructed ponds with simple technical modifications can also be used for storm water control. Golbategene detention pond/dam reservoir capacity is 130962m^3 and Kersa pond capacity is 26421m^3.
- An overall strategy is needed when aiming on mitigate flood consequences. To move a flood from risk area safely, demands careful surface water planning over the whole risk area which also means that different organizations and departments within the municipality needs to work together. Flood risk management is an integrated approach to the development of flood risk strategies that involve engineering, settlement, development, public administration, community-based strategies and land use planning with environmental consideration. Only constructing the ponds cannot be a solution for the flood hazard problem of the study area but those integrated approaches are very essential for the sustainability.
- Cost benefit analysis assessment of flood mitigation measures demands much time resources and large data of damages to realize the evaluation.

6.2 Recommendations

- ACA or concerned parties should give primary attention and work on spatial and temporal data production using recent technologies.
- It is obvious that detention pond is a better option while other options are unthinkable because of environmental, geological and economic factors. The CA should take necessary action with further justification for the implementation without investing for other measures.

- Integrated flood preventive measures are very essential for sustainable development. The city administration should and must coordinate all parties that concern flood mitigation in order to bring real solution.

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Appendix

1.Rainfall Profile

Time	Rain Fall Ratio of Daily R.Fall
hr.	%
0.000	0
0.000	5
0.000	10
0.081	15
0.138	20
0.275	25
0.375	30
0.500	32.5
0.563	35
0.719	40
1.094	45
1.250	50
1.500	53.125
1.744	55
2.188	60
2.500	62.5
2.794	65
3.625	70
4.625	75
6.563	80
10.000	85
14.000	90
18.688	95
23.000	100

2- Arial to Point Ratio (%)

Area	Duration I (hrs)	0.5	1	2	3	4	5	6	9	12	15	18	21	24
0		100	100	100	100	100	100	100	100	100	100	100	100	100
25		68	78	82	85	87	88	88	91	92	93	93	94	94
50		61	71	78	82	84	85	87	89	90	91	92	92	93
75		57	67	75	79	82	84	83	87	89	90	91	91	92
100		54	65	73	78	80	82	83	86	88	89	90	91	91
125		52	63	72	76	79	81	82	85	87	88	89	90	91
150		50	61	70	75	78	80	61	84	86	88	89	89	90
250		44	55	66	71	74	77	78	82	84	86	87	88	89
275		42	54	65	70	74	76	78	81	84	85	86	87	88
300		41	53	54	70	73	75	77	81	83	85	86	87	88
325		40	53	63	58	72	73	77	80	83	84	86	87	87
350		38	52	63	68	72	74	76	80	82	84	85	87	87
375		39	51	62	68	71	74	78	80	82	84	85	86	87
400		38	50	61	67	71	73	75	79	82	83	85	86	87
425		37	50	61	67	70	73	75	79	81	83	84	85	86
450		36	49	60	66	70	72	74	79	81	83	84	85	86
475		36	48	60	66	69	72	74	78	81	83	84	85	86
500		35	48	59	66	69	72	74	78	80	82	84	85	86
525		34	47	59	65	68	71	73	78	80	82	83	85	85
550		34	47	58	64	68	71	73	77	80	82	83	84	85
575		33	46	58	64	68	71	73	77	80	82	83	84	85
600		33	45	57	63	67	70	72	77	80	81	83	84	85
725		31	45	55	62	69	69	71	75	78	80	82	83	84
750		30	43	55	61	68	68	71	75	78	80	82	83	84

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3 - Runoff CN

CN for AMC		
II	I	III
100	100	100
98	94	99
96	89	99
94	85	98
92	81	97
90	78	96
88	75	95
86	72	94
84	68	93
82	66	92
80	63	91
78	60	90
76	58	89
74	55	88
72	53	86
70	51	85
68	48	84
66	46	82
64	44	81
62	42	79
60	40	78
58	38	76
56	36	75
54	34	73
52	32	71
50	31	70
48	29	68
46	28	67
44	26	65
42	25	64
40	22	60
38	21	58

36	19	56
34	18	54
32	16	52
30	15	50
25	12	48
20	9	37
15	6	30
10	4	22
5	2	13
0	0	0

4: Rainfall Data

Adama Monthly Rainfall

R.N	Year	Dec	Nov	Oct	Sept	Aug	Jul	Jun	May	Apr	Mar	Feb	Jan	Total	Max
1	1953	15									43.1		0	58.1	43.1
2	1954		0	25.5	31.5	31	41.5	35	0	0	0			164.5	41.5
3	1955				33		49.5	15	18.5	13				129	49.5
4	1956			0			30	18		25	5			78	30
5	1957	0	0			30	100	22	8	30				190	100
6	1958			6	35.5	46	28.5	22.5		40	70	0	0	248.5	70
7	1959	0	0	2	20.5	48	18	6	0	0	0	35.5	0	130	48
8	1960		0	0	25	25	35	8	20	7	0	0	0	120	35
9	1961													200	62.38
10	1962													200	64.28
11	1963													200	81.00
12	1964													200	90.50
13	1965	0	8.5	16.7	15.8	37	22.8	50	0	15.5	7.5	0	13.1	186.9	50
14	1966	0	0	5.6	22.1	66.9		48.5	32.5		14.5	30		220.1	66.9
15	1967	0						30.7	27.2	25	25.4	0	0	108.3	30.7
16	1968							57.5	3.5	8.5	25	9.7		104.2	57.5
17	1969	0	7.2	0.4	42	46.2	101.2	6.8	35.8					239.6	101.2
18	1970	0	0	2.4	19.2	60	22.7	6.4	5.7	13.1	23.8	19.8	25	198.1	60
19	1971	19.1	4.3	1	47.6	50.8	36.5					0	1.3	160.6	50.8
20	1972	0	0	6.1	27.1	47.2	38.4	58.2	13.6	26.5	51	4.7	1.3	274.1	58.2
21	1973	0	0	57.1	19.5	85.8	36	4.9	53.1	0.1	0	0	0	256.5	85.8
22	1974	0	0	1.5	31.3	27.7	38.5	40.5	40	0	23.7	7.3	0.6	211.1	40.5

R.N	Year	Dec	Nov	Oct	Sept	Aug	Jul	Jun	May	Apr	Mar	Feb	Jan	Total	Max
23	1975		0		22.2	35.2	100.8	47.5	32.7	56.5	1.1	3.3	1.9	301.2	100.8
24	1976	4.5	12.5	0.2	33	53.5	40	21.9		25.7	33		0	224.3	53.5
25	1977	0	33.7	50.9	18.9	28.4	55.4	52.3	21.5	38.6	42.7	10.6	47.9	400.9	55.4
26	1978	5.5		49.5	22.3	70.6	49.4	25.5	11.2	9	2.4	24	3.2	272.6	70.6
27	1979			17.4	7.8	19.1	17.2	33.2	56	4.8	20.3	20.6	49.2	245.6	56
28	1980	0	3.7	33.2	100									136.9	100
29	1981	0	0	2.1	36.6	65.1	45	1.1		19.2	48	25.3	0	242.4	65.1
30	1982	10	8.6	38.3	13.9	52.3	30.5	18.1	15.5	10.5	17.8	19.6	5.8	240.9	52.3
31	1983	0	0	7.7	14.3	57	58.6	14	28.7	38.5	16.8	25.7	3.5	264.8	58.6
32	1984	17.2	0	0	11.5	45	68	25	31	0.4	2.4	0	0	200.5	68
33	1985	0	0	0	55	57	55.5	4	50	36.4	7.5		3	268.4	55.5
34	1986	0	0	0	15.8	28.9	50	40.2	18.3	4	25	30.5	0	212.7	50
35	1987	0	0	0	8.4	37.6	44.4	0	43	27	30	3.1	0	193.5	44.4
36	1988	0	0	30.2	31.2	25.6	23.4	13.6	9.4	21	6.8	16.9	17.6	195.7	30.2
37	1989	2.3	0	3.2	18.8	36.3	55	40.1	0	27.7	11.1	15.5	0	210	55
38	1990	0	0	6.3	34.2	30.3	77	5	11	48.7	40.7	25.8	0.7	279.7	77
39	1991	1	0	10.1	32	30	73.8	28.8	7	8.8	30.6		0	222.1	73.8
40	1992	2.5	0	14.2	29.1	38.5	39.5	23.8	4	26.8	0	19.4	27.2	225	39.5
41	1993	0	0	8.2	23.5	33	70	26.4	21.4	28.6	0	17.4	6	234.5	70
42	1994	51	20.6	13.8	48.9	47.2	47.6	17.5	13.5	29.8	1.4	0	0	291.3	51
43	1995	1.8	0	14.1	31.4	42.3	77.5	30.3	6	22.4	14.5	12.4	0	252.7	77.5
44	1996	0	3.9	0	30.5	39.6	42.4	30.9	47.5	18	26.4	0	14.1	253.3	47.5

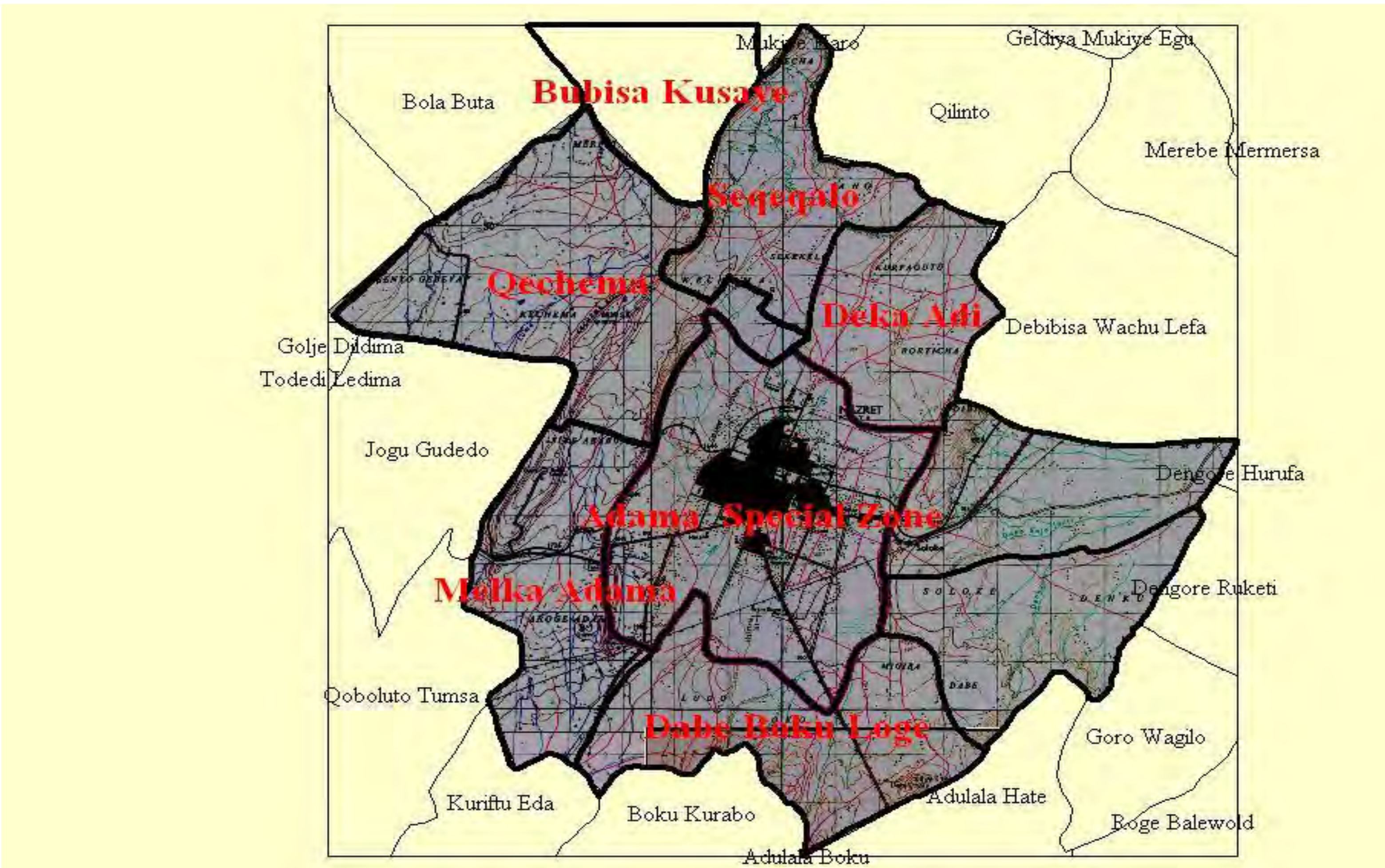
R.N	Year	Dec	Nov	Oct	Sept	Aug	Jul	Jun	May	Apr	Mar	Feb	Jan	Total	Max
45	1997	0	11.7	48.6	28.2	35.8	28.6	17.6	3.6	22.5	27.4	0	14.4	238.4	48.6
46	1998	0	0	49.3	24.6	39.5	59.8	30.5	13.5	13	50.3	19.2	7.5	307.2	59.8
47	1999	0	1.6	30	30.5	32.1	41.5	24.2	9.2	1.2	16.4	0	8.2	194.9	41.5
48	2000	7.2	31.2	43.8	20.4	82.2	99.8	42.5	14.7	3.6	19	0	0	364.4	99.8
49	2001	6.3	0	1.7	40.6	34.6	44.3	18.3	104.8	12.6	34.2	4.2	0	301.6	104.8
50	2002	10.9	0	0.9	28.4	48.3	30.8	15.3	15.7	17.2	13	11.1	14.8	206.4	48.3
51	2003	40	4.2	0	41.7	70.4	46.7	17.9	3.6	40.6	58.3	36	37.8	397.2	70.4
52	2004	1	8.8	37.3	16.1	43.3	25.6	36.1	1.4	23.5	25.9	2.9	12.4	234.3	43.3
53	2005	0	5	5	20.2	39.7	25.8	23.3	25.8	12.5	42.3	4.3	39.6	243.5	42.3
54	2006	13.5	0.5	3.8	51.3	41.5	18.4	22.5	13	28.8	47	62.8	17.6	320.7	62.8
55	2007	0	4.3	25.5	52	55.1	40	19	31.7	33.5	30.7	15.8	10.6	318.2	55.1
56	2008	0	29.2	11.1	36.1	40	49.1	15.4	24.1	72.5	0	0	8.4	285.9	72.5
57	2009			45.5	9.5	12.7	54	12	14.4	2.3	1.8	0	37	189.2	54
58	2010	0	17.6	0	34.9	56.1	36	28	28	18.4	22.6	26.7	0	268.3	56.1
59	2011	0	19.1	0	40.3	45.5	46.9	17.2	23.4	7.6	5.6	0	0	205.6	46.9
60	2012	3.9	0.5	1.4	35.7	54.8	69.6	16.3	11	33.3	0	0	0	226.5	69.6
61	2013	0	2.2	8.3	59.4	35	57.9	14	9.2	11	26.8	0.2	1.8	225.8	59.4
62	2014						46.6	7.3	15.5	4.9	46.4	3.7	0	124.4	46.6
Max		**51.00**	**33.70**	**57.10**	**100.00**	**85.50**	**101.20**	**58.20**	**104.80**	**72.50**	**70.00**	**62.80**	**49.20**	**400.90**	**104.80**
Mean		**4.43**	**4.98**	**14.43**	**30.37**	**43.93**	**47.93**	**23.76**	**20.55**	**20.1**	**21.42**	**11.75**	**8.63**	**224.18**	**60.49**
Min		**0.00**	**0.00**	**0.00**	**7.80**	**12.70**	**17.20**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**58.10**	**30.00**
St.Dev		**9.94**	**8.70**	**17.35**	**15.70**	**15.14**	**21.35**	**14.51**	**18.96**	**15.70**	**18.03**	**13.13**	**13.33**	**68.28**	**18.11**

5. Adama Annual Daily Maximum Rainfall

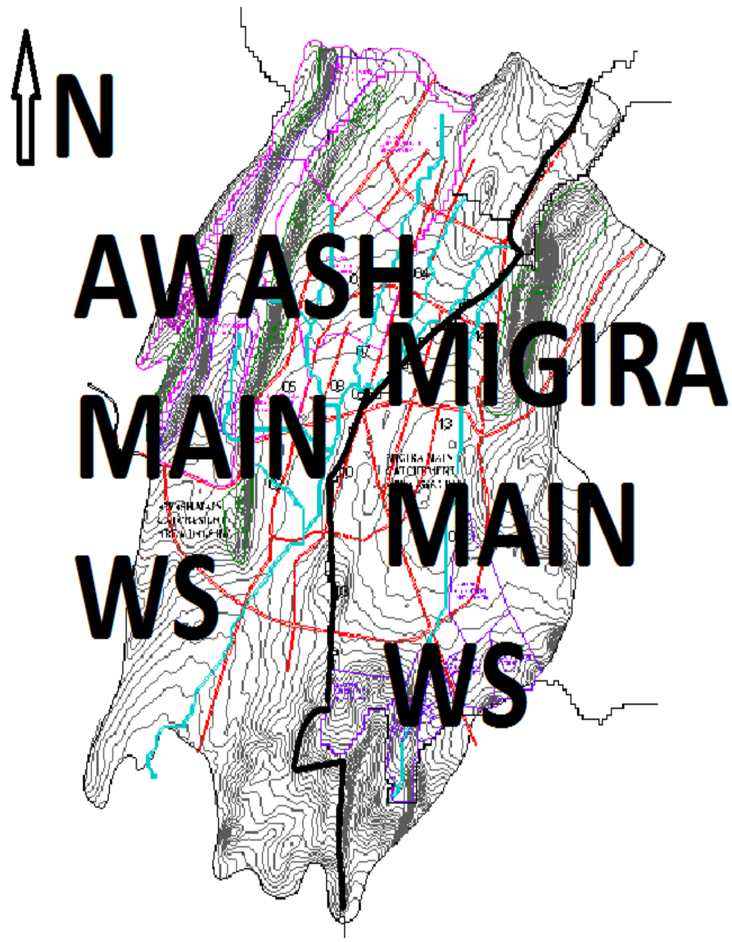
R.N	Year	Max	R.N	Year	Max
1	1953	43.10	36	1988	30.20
2	1954	41.50	37	1989	55.00
3	1955	49.50	38	1990	77.00
4	1956	30.00	39	1991	73.80
5	1957	100.00	40	1992	39.50
6	1958	70.00	41	1993	70.00
7	1959	48.00	42	1994	51.00
8	1960	35.00	43	1995	77.50
9	1961	62.38	44	1996	47.50
10	1962	64.28	45	1997	48.60
11	1963	81.00	46	1998	59.80
12	1964	90.50	47	1999	41.50
13	1965	50.00	48	2000	99.80
14	1966	66.90	49	2001	104.80
15	1967	30.70	50	2002	48.30
16	1968	57.50	51	2003	70.40
17	1969	101.20	52	2004	43.30
18	1970	60.00	53	2005	42.30
19	1971	50.80	54	2006	62.80
20	1972	58.20	55	2007	55.10
21	1973	85.80	56	2008	72.50
22	1974	40.50	57	2009	54.00
23	1975	100.80	58	2010	56.10
24	1976	53.50	59	2011	46.90
25	1977	55.40	60	2012	69.60
26	1978	70.60	61	2013	59.40
27	1979	56.00	62	2014	46.60
28	1980	100.00			
29	1981	65.10			
30	1982	52.30			
31	1983	58.60			
32	1984	68.00			
33	1985	55.50			
34	1986	50.00			
35	1987	44.40			

6.Figures

Fig a) Adama City and surrounded rural Kebeles

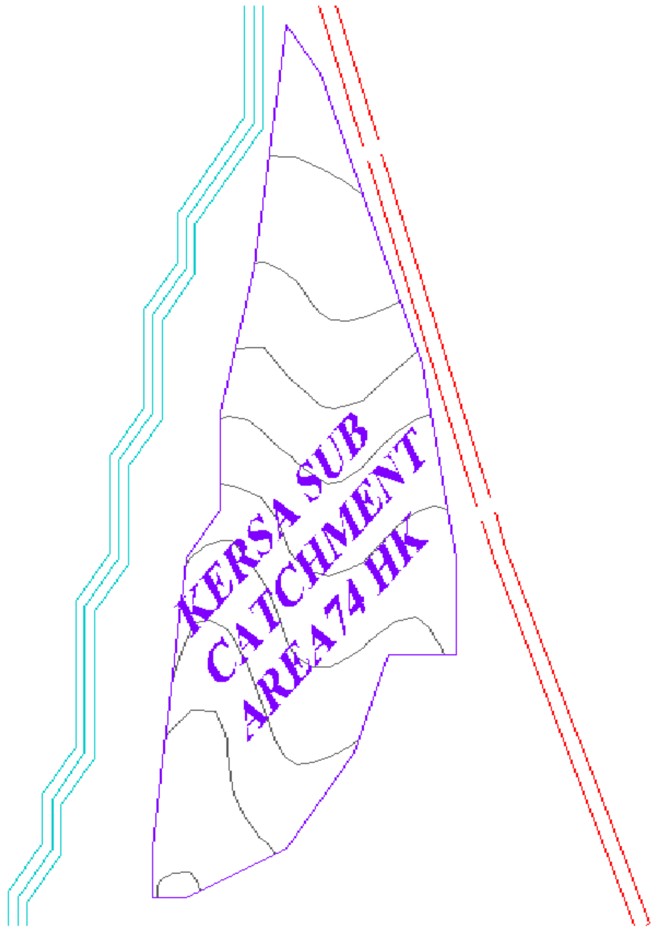


b) Main catchments

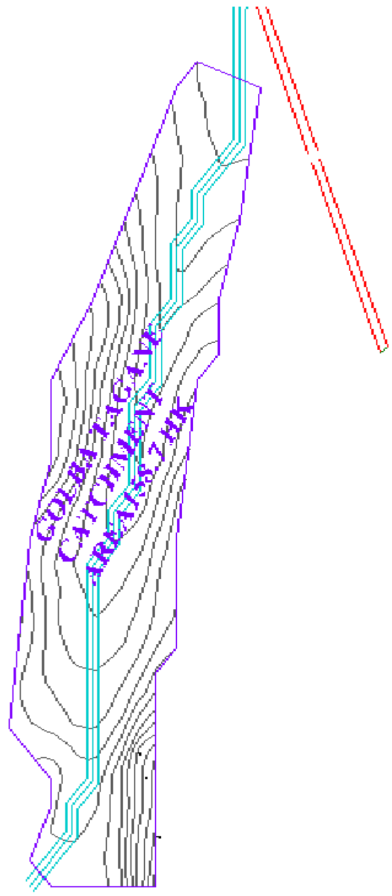


MIGIRA MAIN WATERSHED

c) Kersa Sub Watershed



D) Golba Tegene sub Watershed



7. Sub-Watershed that needs treatments

Name of Genda	Name of sub catchment	Area in the watershed (ha)	Closed area	Treated area
Ganda Melka Adama	Sire aba bune	229.5	79.6	
	Lebu	800.6	74.5	
Ganda Dekaadii	Sekekelo	747.4	87.9	
	Cheri	393.2	78	
Genda 01	Chokonu	728	95.7	
Genda 05	Dinegde degage	348.5	98.1	
Genda 09	Chemo	79.2	79.2	
Genda 03	Lugo	166.5		
Genda 14	Gendegara	403.4	403.4	
Genda 02	Dabe	191.9		
Genda 04	Water way	0		
Ganda Bokku Shenen	Kersa	74		
	Golbetegene	158.7		
	Haroya'a	125.8		
	Birkaroba	96.3		
Total		4535	996.4	5531.4

8. Primary Data Collected at Site

Kersa and Golbetegene Catchments

GPS data

Kersa Catchment

St.1 = Starting Point = Kersa gully, around 30m from the main access road (at a curve)

- Elevation = 1630m
- N = 08.50298
- E = 039.28370
- Accuracy = 3.5m

St.2 = at the side of the Asphalt (Erob gebeya crossing-at box culvert)

- Elevation = 1635m
- N = 08.49849
- E = 039.28569
- Accuracy = 3.7m

St.3 = at junction of Gorotafus & Kersa Gullies

- Elevation = 1639m(top bank)
- N = 08.49740
- E = 039.28621
- Accuracy = 4.8m

St.4 = at tip of hill kersa side (Adama view)

- Elevation = 1674m
- N = 08.49359
- E = 039.28681
- Accuracy = 4.6m

St.5 = Junction of Kersa gorge (three gully junction)

- Elevation = 1663m
- N = 08.49220
- E = 039.28460
- Accuracy = 5.5m

St.6 = Kersa tip

- Elevation = 1719m
- N = 08.48673
- E = 039.28089

- Accuracy = 5m

Golbetegene Catchment

St.1 = Axis of the D.dam – right side top

- Elevation = 1676m
- N = 08.49162
- E = 039.28014
- Accuracy = 5.1m

St.2 = Right GL

- Elevation = 1662m
- N = 08.49165
- E = 039.27992
- Accuracy = 4.9m

St. 3 = Top of the main gully- right side top bank

- Elevation = 1662m
- N = 08.49167
- E = 039.27979
- Accuracy = 4.2m

St.4 = Gully bed level

- Elevation = 1651m
- N = 08.49162
- E = 039.27975
- Accuracy = 5m

St.5 = Left top bank

- Elevation = 1665m
- N = 08.49165
- E = 039.27962
- Accuracy = 4.4m

St.6 = Left intermediate point

- Elevation = 1667m
- N = 08.49175
- E = 039.27911
- Accuracy = 3.8m

St.7 = Left top of the D.dam axis

- Elevation = 1678m
- N = 08.49196
- E = 039.27824
- Accuracy = 5.6m

St.8 = U/s point of the reservoir area

- Elevation = 1672m
- N = 08.49005
- E = 039.27877
- Accuracy = 4.3m

St.9 = Near the asphalt junction (Marked point 277-2)

- Elevation = 1630m
- N = 08.50190
- E = 039.28209
- Accuracy = 5.5m

St.10 = Kersa & Golbetegene junction (inlet point)

- Elevation = 1623m
- N = 08.50252
- E = 039.28224
- Accuracy = 4m

St.11 = Top of Kersa & Golbetegene junction

- Elevation = 1627m
- N = 08.50260
- E = 039.28225
- Accuracy = 4.0m

Additional Coordinates

For the GT (159ha) catchment

Left side abutment at the dam axis:

E39016’42”

N8029’32”

Right side abutment at the dam axis:

E39016’48”

N8029’30”

Center of the stream bed at the dam axis:

E39016’47”

N8029’30”

For Kersa(74ha) Catchment

A coordinate at the center of the stream bed:

E39017’10”

N8029’51”

For Kersa(the 74ha) & GT(the 159ha) catchments

A point that divides Kersa(the 74ha) & GT(the 159ha) catchments:

E39016’56”

N8030’9”