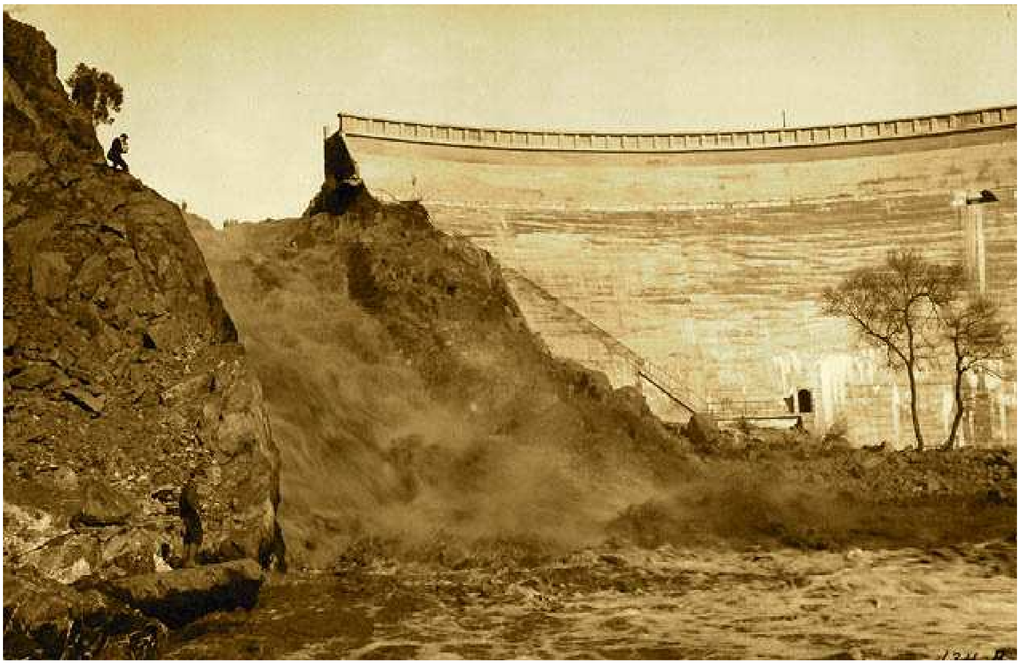
Dam Break Analysis Using HEC-RAS Model
The Case Study of Gilgel Gibe 1

A thesis

Submitted to the School of Post Graduate –Arba Mich University in partial Fulfillment of the Requirements for the Degree of Masters of Science in Hydraulic and Hydropower Engineering

October 2010



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CERTIFICATION

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I, Hamdi Sebit Eisa, declare that this is my own original work and that it has not been presented and will not be presented to any other University for similar or other degree award.

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The Case Study of Gilgel Gibe 1

A thesis Submitted in Partial in partial fulfillment of the requirements for the degree of Masters of Science in Hydraulic and Hydropower Engineering.

Date defended: October 08, 2010

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Acknowledgement

Two years were the time, for completed a master of science in Hydraulic and Hydropower Engineering , from Arba Minch University. During this time I meat and worked with great people, who contribute directly or indirectly in my success. It is a good chance to thanks some of them if not all of them.

First thanks to God for giving me this chance and power to finish my study .

I would like to express my deep felt gratitude to my advisers, Dr. Adane Abebe and Dr. Nigussie Tekle for their advises , encouragement and patience.

My thanks goes to Nile Basin Initiative in general and applied training project in particular for offering this scholarship , and also my thanks goes to Engineer Isaac liabwel Undersecretary of Ministry of water resources and Irrigation for his effort to bring chances to the Government of southern Sudan, in the line of organization and institutes, I would like to give special thanks to Dr.Mekonna Ayana NBI coordinate (Ethiopia) for helping us all this time, I cannot forget to thanks Dr.Yosif Ibrahim from Easter Nile Technical Regional office , for following my progressing, I would like to thanks all institutes where I collected the data.

Am grateful to my family , my father Sebit Eisa for taking care of my family, my wife Rita Stephen for taking the responsibility of the home, my mother Salaha Khamis , my brother Faisal Sebit ,indeed my thanks to my colleagues during the study ,because of you I never miss home.

Abstract

Floods due to failure of dams induce widespread damages to property and losses of life and owing to its high magnitude and unpredictable sudden occurrence.

The objectives of the study is to analyze , and evaluate the impact of 100 year flood , and probable maximum flood as inflow scenarios as well full reservoir pool sunny day failure , the objectives of the study is achieved by using two computer models .

Upstream of the study dam , the catchment has been divided into three subbasins, namely upper subbasin, upper left subbasin, and upper right subbasin, Hydrological Engineering Center- Hydrological Model System developed for study area after determining the initial parameters, such as probable maximum precipitation , and one percent precipitation, the objective of hydrological model was to estimate probable maximum flood , one chance flood , and the result of the two scenarios applied to the dam in case of overtopping.

Hydrologic Engineering Center -River Analysis System used to determine water surface profile, and simulate and predict peak flow , and outflow hydrograph for the three scenarios, model is used for dam breach analysis , dam failure were analyzed for probable maximum flood , one chance flood , and sunny day failure, breach parameters are determined by using two methods ,Bureau of reclamation (1982) and Thun Von & Gillette (1999).

The result of the hydrological models showed that , there is no overtopping for the two scenarios , probable maximum flood , one chance flood, meanwhile the result of hydraulic model indicate that peak flow , and outflow hydrograph depend on the additional inflow , nature of the reservoir, and breach paraeters.

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Abbreviation and Acronyms

Assenda	Assendabo station
Asl	Above sea level
DEM	Digital elevation Model
D.B	Dam bottom
E	East direction
GIS	Geographic information system
HEC-HMS	Hydrological Engineering Center-Hydrological model System
HEC-RAS	Hydrological Engineering Center-River analysis system System
M	Meter
M^3	Cubic meter
M^3/s	Cubic meter over second
m.a.s.l	Meter above sea level
MCM	Million cubic meter

MW	Mega watt
Max WL	Maximum water level
Min WL	Minimum water level
N	North direction
PMF	Probable maximum Flood
PMP	Probable Maximum precipitation

Chapter One

1.0 Introduction

1.1 Background

The construction of dams ranks with the earliest and most fundamental hydraulics and civil engineering activities. Most of great civilization has been identified with the construction of storage reservoir appropriate to their needs.

Dam plays a very important role in the economy of the country by providing numerous benefits such as water supply, hydropower, irrigation, navigation, and flood control. Generally dam is contributing to human positively most of the time; on the other hands it has contributed negatively in case of dam break and its impact on downstream infrastructure.

Embankment dams are subject to possible failure from either overtopping or piping which erodes a breach through the dam. The breach formation is gradual with respect to time and width. The majority of embankment dam failures are generally the result of inadequate design, poor construction methods, deteriorated pipe, Hydraulic fractures , or significant environmental occurrence (www.prinsco.com).

The flood resulting from Dam break or failure has great impact to downstream area through damaging community properties, public infrastructure and environment. The percentage of damage depend on the nature of flood and human interaction on flood such as flood protection ,and flood management .And also the nature of life downstream , in case of urban area(centers), whose failure may cause damage to homes , agriculture , industry and commercial facilities , important public utilities , main high -ways or rail- way and roads .But in case of rural downstream area failure of dams may cause damage to houses , or town-ship and country road . Indeed there are losses of life for the two cases but this depends on the degree of hazard. When the degree of hazard is lower or in term of risk is class (C) there will be less losses, when you compare with the high hazard or class (A),(Antonio,2002).

1.2 Problem Statement

Dam safety has received considerable attention through all the process of dam, starting from designing, construction, and operation, even in maintenance and repair, to protect the dam itself and reduced risk downstream area. But for unexpected reasons resulting from excessive rainfall or any other reason which may change the characteristic of the watershed in general and meteorological and hydrological parameters in particular, or changing of geological condition of the area from non earthquake zone to earthquake zone, dam failure might occurs. The result having parameters different than design parameters lead to excess discharge and excess level.

Flood resulting from the failure of constructed dams produced some of the most disasters. When dams fail, property damage, and loss of life can vary dramatically with the extent of the inundation area.

1.3 Objectives of the study

The research's general aims will be achieved through the following objective

1.3.1 Main objective

To analyze, and evaluate the impact of 100 –year (1% chance) flood and probable maximum flood (PMF) as inflow scenarios as well as the full reservoir pool sunny day dam failure scenario (Three scenarios).

1.3.2 Specific objectives

In addition, some **specific** but not less in importance has to achieved

- ❖ Determine water surface profile in case of unsteady flow resulting from overtopping in downstream channel.
- ❖ Determine the effect of additional inflow to peak discharge and outflow hydrograph.
- ❖ Determine the effect of breach parameters and nature of the reservoir to peak flow and outflow hydrograph.

1.4 Scope of the study

The research is concerning dam break analysis through two modes of failure overtopping and piping as are common problems in rockfill embankment dam through three scenarios: 1 percent chance exceeding flood, probable maximum flood (PMF), and sunny day dam failure. Using hydrological model HEC-HMS, breach parameter, and hydraulic model HEC-RAS .Is not including flooded area.

1.5 Significance of the study

- ❖ There is presently both a need and opportunity to achieve significant improvement in technology to be used for analyzing embankment dam breach process .The potential benefits to be achieved from this research aid risk assessment studies, in which thresholds of dam failure, probability of failure, and the consequence of failure are all of the prime importance.
- ❖ When property located downstream of the dam, accurate prediction of dam break is helpful in development of effective emergency action plan, and designing of early warning.

- ❖ Much valuable information will be available from embankment failure that can help in further studies , such as Environmental impact assessment .which required for any dam failure greater than 15m.

1.6 Description of the basin characteristic

1.6.1 Location of the basin

Omo –Gibe River Basin which is considered as one of most important basin in Water resources potential, is located in South-west part of the country .It lies between 4˚30 and 9˚ 30 North Latitudes and 35˚00 and 38˚ 00 East longitudes. The basin has an estimated area of 79000 km^2.

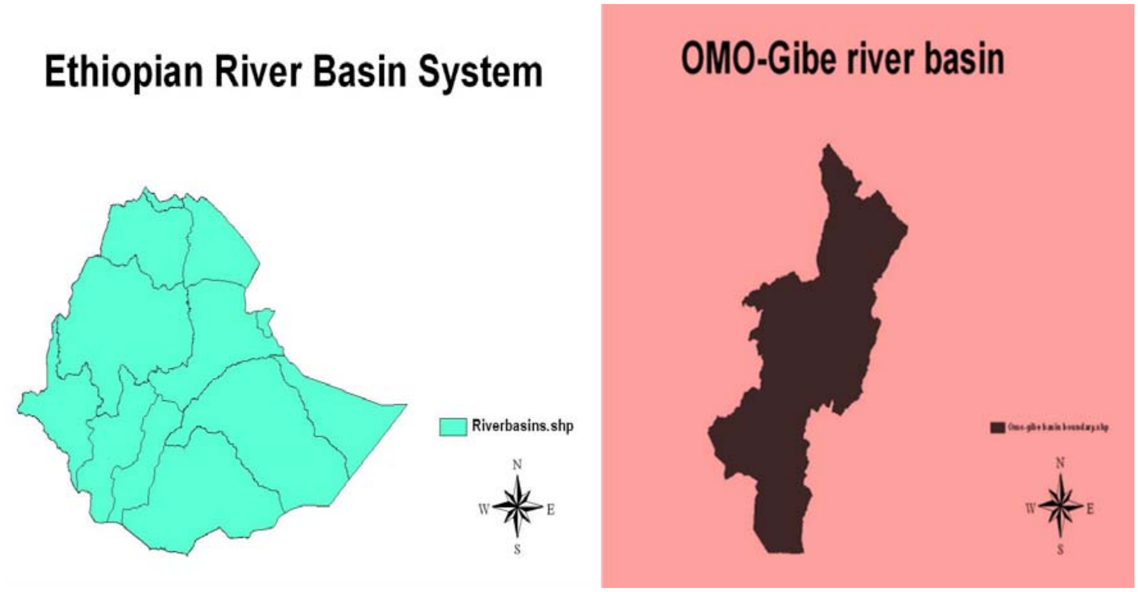


Figure 1.0 : Ethiopia and Omo-Gibe River Basin map

1.6.2 Climate

The climate of Omo-Gibe River valley varies from a hot arid climate in most southern part of the plain to the tropical humid one in the highlands .That include the extreme north near Bako. The areas near Jimma and around the head waters of the Gojeb river intermediate between these extremes and for greater part of the basin is tropical sub humid.

1.6.3 Topography

Generally the topography of Omo-Gibe River Basin is characterized by its physical variation from the north side .Two third of the basin is mountainous to hilly terrain cut by deeply incised gorges of Omo , Gojeb , and Gigle Gibe River.

While one third of the southern basin is flat alluvial plain punctuated by hilly areas.
The head waters of the Great –Gibe River are at an elevation of about 2200 m.a.s.l, indeed there are different tributaries from different directions , but generally the direction of the flow in Gibe river is southwards to Omo River , to lake turkana .

1.6.4 Soils

Soils in Omo-Gibe Basin are for the most part permeable and well drained .In the upper and middle parts of the basin is true of valley slopes near the catchment divide . While the valley bottoms have significant areas of less permeable soils with impeded drainage, in lower Omo the sedimentary floodplain is mostly characterized by poor drainage.

1.6.5 Land cover

In a very broad term, most of the northern catchments of the Omo-Gibe Basin is under extensive cultivation with increased land pressure, meaning the expansion of cultivated areas in to increasingly marginal lands at the expense of wood lands. Deforested areas are now confined to areas too steep and inaccessible to farm. The flatter poorer drained bottom lands of the northern catchments are usually not cultivated but are used for dry season grazing and eucalyptus tree plantations. The main gorges of the basin are relatively unpopulated and support a cover of open wood-land and bush-land with grasses, the eastern part of the basin has some of the most densely populated and intensively farmed areas in the country let alone the basin. The south of the basin is more sparsely populated with a greater population of natural vegetation, through even here the forest is decimated at an alarmingly rate(Kemal,2007).

1.6.6 Description of Gilgele Gibe I

Gilgle Gibe I is rock fill dam with asphalt concrete face, constructed for hydropower purpose with installed capacity 183.9 MW. The dead and live storage are 171 Mm^3 and 668 Mm^3 respectively .The Crest length is 1700 M .The name of the dams comes from the Gilgel river which is the right-hand tributary of great Gibe which becomes Omo River downstream where is a tributary to turkana lake (EELPA,1995) . Feature of the dam are list in the Table 3.2 .

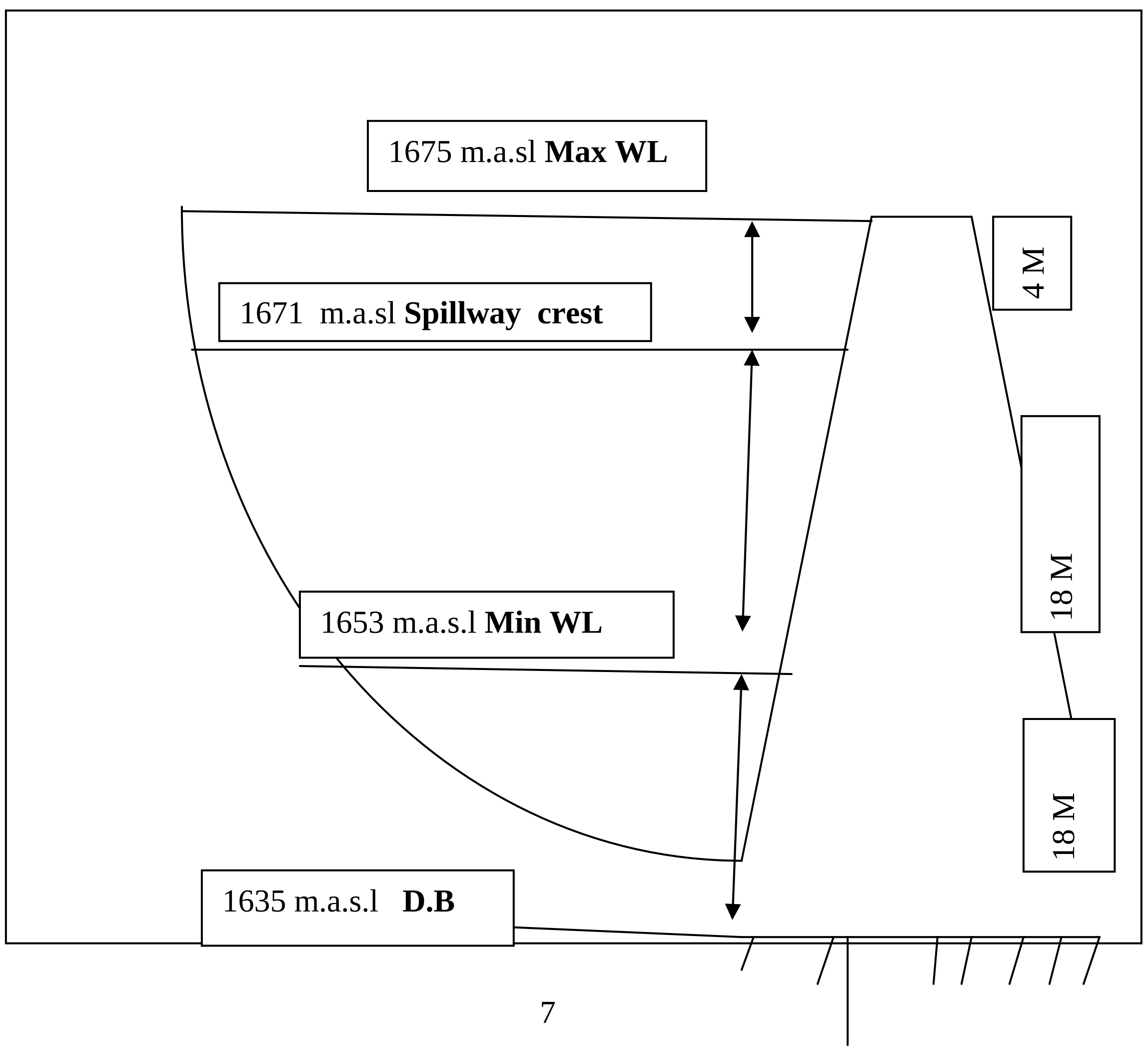


Figure 1.1: Reservoir description

Chapter Two

2.0 Literature Review

2.1 General

Benefits provided by dams include water supply for drinking, Irrigation, industrial, flood control, hydropower, recreation, and Navigation. Dams require maintenance, monitoring and rehabilitation if needed to continue their service.

Dams are water storage, control, or diversion structure that impounds water in the upstream reservoir. Dam failure can take several forms including collapse, or breach in the structure. Most dams storing large amount of water can cause significant flooding downstream. Dam breaks result from any one or combination of the following (G.B.R.A, 2001).

- Prolonged period of rainfall and flooding which cause most of failure.
- Inadequate spillway capacity resulting in overtopping of the embankment
- Improper maintenance, including failure to remove trees, repairing internal seepage problem of maintaining gates, valves, and other appurtenant structure.

- Improper design such as filter, cor or use of improper construction materials.

- Landslides into reservoir, which cause surge that result in overtopping.

- Earthquake, which typically causes longitudinal cracks at the tops of the embankment leading to the structure failure.

- The first filling of the reservoir leads to failure of dam complete or partial in case of gradually system is not applied.

In the event of a dam break the energy of the water stored behind the dam is capable of causing rapid and unexpected flooding downstream resulting in the loss of life, great property damage, and shut down of facilities.

2.2 Embankment Dam

The two principal types of embankment dams are earth and rock-fill dams. Depending on the predominant fill material used. Some generalized sections of earth dams showing typical zoning for different types and quantities of fill materials and various methods for controlling seepage are presented. When practically only one impervious material is available and the height of the dam is relatively low, a homogeneous dam with internal drain may be used (Singh and Varshney, 1995). The inclined drain serves to prevent the downstream slope from becoming saturated and susceptible to piping and/or slope failure and to intercept and prevent piping through any horizontal cracks traversing the width of the embankment. Earth dams with impervious cores, are constructed when local borrow materials do not provide adequate quantities of impervious material. A vertical core located near the center of the dam is preferred over an inclined upstream core because the former provides higher contact pressure between the core and foundation to prevent leakage, greater stability under earthquake loading, and better access for remedial seepage control. An inclined upstream core allows the downstream portion of the embankment to be placed first and the core later and reduces the possibility of hydraulic fracturing. However, for high dams in steep-walled canyons the overriding consideration is the abutment topography. The objective is to fit the core to the topography in such a way to avoid divergence, abrupt topographic discontinuities, and serious geologic defects.

In Pervious foundations, seepage control is necessary to prevent excessive uplift pressures and piping through the foundation. The methods for control of under seepage in dam foundations are horizontal drains, cutoffs (compacted backfill trenches, slurry walls, and concrete walls), upstream impervious blankets, downstream seepage breams, toe drains, and relief wells. Rock-fill dams may be economical due to large quantities of rock available from required excavation and/or nearby borrow sources, wet climate and/or short construction season prevail, ability to place rock fill in freezing climates, and ability to conduct foundation grouting with simultaneous placement of rock fill for sloping core and decked dams (Bharat Singh and R.S.Varshney ,1995)

2.3 Dam failure

From 1946-1955 a total of 12 major dam failures were recorded, and during the same time more than 2000 dams were constructed worldwide. And from year 1956-1965 a records of 24 failures and more than 2500 new dams were constructed during the same period of time (Jansen ,1988).Failure is common for all the dams but mode of failure is different for each type.

2.3.1 Cause of failure of earth Dam

Earth dams are subjected to mainly three groups of failures

2.3.1.1 Hydraulic failure

This may occur due to one or more of the following causes

(i) Overtopping

(ii) Erosion of upstream face

(iii) Erosion of downstream face

(iv) Erosion of downstream toe

(v) Frost action (Arora,2002).

2.3.1.2 Seepage failure

This may occur due to following cause.

(i) Piping through the dam.

(ii) Piping through the foundation.

(iii) Conduit leakage.

(iv) Sloughing of downstream toe. (Arora, 2002).

2.3.1.3 Structural failures

Are generally shear failure leading to sliding of the embankment or the foundation and it consist.

(i) Slides in embankment.

(ii) Foundation slides.

(iii) Liquefaction slides.

(iv) Failure by spreading.

(v) Failure due to Earthquake.

(vi) Failure due holes caused by burrowing animals .

(vii) Failure due to holes caused by leaching of water-soluble salts. (Arora, 2002).

2.3.2 Cause of Failure in Rockfill embankment dam

The principal mechanisms and failure can be within one of the following

(i) Overtopping leading to washout, less cohesive materials.

(ii) Internal erosion and piping with migration of fines from the core, also in the foundation.

(iii) Embankment and foundation settlement.

(iv) Instability when downstream slope too high or too steep in relation to shear strength of the shoulder materials.

(v) Instability when upstream slope failure following rapid drawdown of water level .

(vi) Failure of downstream foundation due to overstress, weak horizons. (Arora, 2002).

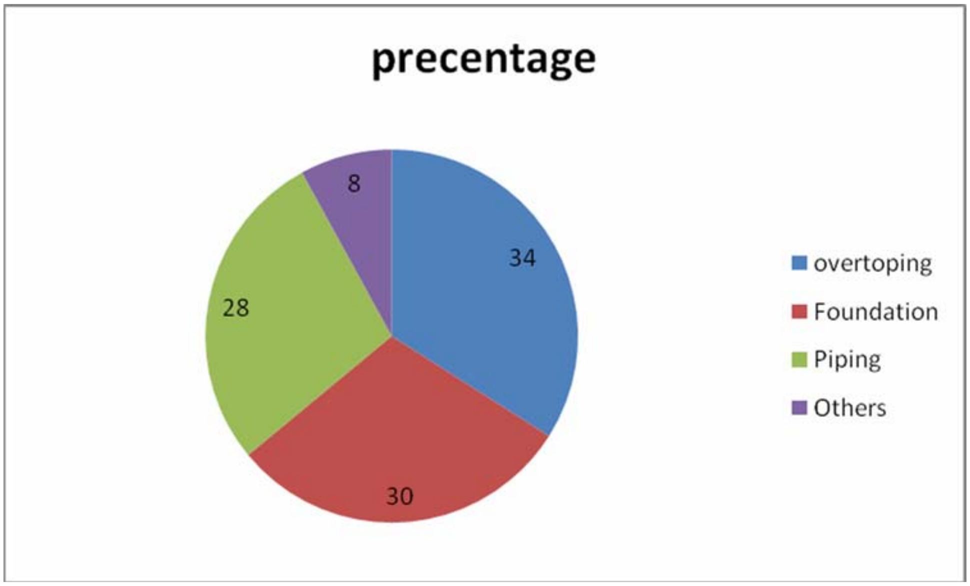


Figure 2.0: Modes of failure in percentage
Source: Dam Safety office (1998)

Based on figure 2.1 , showed that one third (34%), of the dam failure resulting from overtopping mode . Cause of overtopping is Extreme rainfall beyond the planned design . This might lead to assume that, most of those dams are designed for a certain return period ,rather than probable maximum flood. Dams failure resulting of foundation mode is 28 %, and the causes of the failure are Poor maintenance and error in design.

Table 2.0 : Some examples of Dam failure over the world
Source : Wikipedia(2010)

Dam/incident	Year	Location	Details
Buffalo Creek Flood	1972	West Virginia,United States	Unstable loose constructed dam created by local coal mining company, collapsed in heavy rain
Canyon Lake Dam	1972	South Dakota,United States	Flooding, dam outlets flooded with debris.

Banqiao and Shimantan Dams	1975	China	Extreme rainfall beyond the planned design capability of the dam
Teton Dam	1976	Idaho, United States	Water leakage through earthen wall, leading to dam failure.
Laurel Run Dam	1977	Pennsylvania,United States	Heavy rainfall and flooding that over-topped the dam.
Lawn Lake Dam	1982	Rocky Mountain National Park,United States	Outlet pipe erosion; dam under-maintained due to location
Val di Stava Dam collapse	1985	Italy	Poor maintenance and low margin for error in design; outlet pipes failed leading to pressure on dam.
Peruća Dam detonation	1993	Croatia	Not strictly a dam failure as there was a detonation of pre-positioned explosives by retreating Serb Forces.
Saguenay Flood	1996	Quebec, Canada	Problems started after two weeks of constant rain, which severely engorged soils, rivers and reservoirs. Post-flood enquiries discovered that the network of dikes and dams protecting the city was poorly maintained.
Opuha Dam	1997	New Zealand	Heavy rain during construction caused failure, dam was later completed
Vodní nádrž Soběnov	2002	Soběnov, Czech Republic	Extreme rainfall during the 2002 European floods
Hope Mills Dam	2003	North Carolina,United States	Heavy rains caused earthen dam and bank to wash away
Big Bay Dam	2004	Mississippi,United States	A small hole in the dam, grew bigger and eventually led to failure.
Camará Dam	2004	Brasil	
Shakidor Dam	2005	Pakistan	Unexpectedly extreme rain
Taum Sauk reservoir	2005	Lesterville, Missouri, United States	Computer/operator error; gauges intended to mark dam full were not respected; dam continued to fill. Minor leakages had also

			weakened the wall through cavitation
Campos Novos Dam	2006	Campos Novos,Brazil	Tunnel collapse
Ka Loko Dam	2006	Kauai, Hawaii	Heavy rain and flooding. Several possible specific factors to include poor maintenance, lack of inspection and illegal modifications.
Situ Gintung Dam	2009	Tangerang,Indonesia	Poor maintenance and heavy monsoon rain
Kyzyl-Agash Dam	2010	Kazakhstan	Heavy rain and snowmelt
Hope Mills Dam	2010	North Carolina,United States	Sinkhole caused dam failure

2.4 Dam safety

Through designing of dams major effort has been directed to determine the safety of the dam, the reason behind explained by the fact the water retained by a dam has a potential to cause extensive loss of life and considerable economic damage in the downstream flood plain in the event of failure. Rock fill dam safety can be achieved through two phases.

2.4.1 Design consideration

The design and construction of rockfill dam is governed by more or less the same principle as that used in an earth dam. During the design of rockfill dam the following points must be considered.

2.4.1.1 Top width

Should not be less than 3.0 M for rockfill dams up to 60 M height, and 4.5 M for greater height

Because the rock fill dams are less vulnerable to wave action , a minimum free board of 1.5 to 3.0 above the MWL is usually recommended. Sometimes rockfill are built higher, with allowance for settlement. The camber varies from one section to the other and it's proportional to the height of the dam at that section. Thus the maximum allowance is made at the central section, which decreases towards the abutments (Arora,2002)..

2.4.1.3 Slope and Section

The rock fill dams usually have the upstream and downstream slope equal to the angle of repose of rock, which is between H:V 1.3:1 and H:V 1.4:1 .However, the upstream slope may be flatter up to 2:1 to facilitate construction of the upstream membrane .The overall base width of the most rock fill dams are about 2.5 to 3.5 times the height. The shear strength of the rock fill is estimated from angle of repose. In case of very high Rock fill dam, sometimes large scale field tests are conducted by using the slope stability method.

The analytical methods using the stress-strain relationships of the materials have also been used to compute the displacements and the stresses in the rock fill dam. The finite element method is also being increasingly used to determine the stresses and deformation in the rock fill. However because of the difficulties of obtaining the correct stress-strain relations of the materials and their incorporation in the analysis, these methods are not able to replace the conventional methods based on the limiting equilibrium concept or the rigid – plastic analysis (Arora, 2002).

2.4.1.4 Settlement

Care shall be taken while placing the rock fill to minimize the settlement and the possibility of damage to the impervious membrane. The major settlement in the rock fill dam occurs during the construction stage before the membrane is placed. When the membrane is placed concurrently with the rock fill dam. The settlement during the construction stage should be considered in the design of the membrane. Substantial settlements also occur in the rock fill dam when the reservoir is filled and the trust due to water load is transmitted to the rock fill (Arora, 2002).

2.4.1.5 Axis of the dam

It is desirable to keep the axis of the rock fill slightly curved in plan, with its convexity towards the upstream. Is because of the movements of rock fill downstream due to water load will close the vertical construction joint in the impervious membrane (Arora,2002).

2.4.2 Flood approach

Based on the statistics of the of dam failure one –third of the failure can be traced to inadequate spillway capacity .Different methods are available for estimating spillway flood.

2.4.2.1 Flood Frequency Analysis

(Where estimation of spillway design is based upon the probability of the events being exceeded in any given year) . This statistical approach has been widely used , despite some major drawbacks .One of the problem is that the selection of design flood can vary considerably , depending upon the method of statistical analysis adopted .Kit (1977) has stated that the flood records usually available yield 100-year flood estimates with significant uncertainty .To estimate the design flood for a dam spillway , the statistical curves have always to be extrapolated into a realm of considerable uncertainty and the analyst has often to rely upon data lacking either quality or quantity or both.

When long stream flow records are available . It is possible not only to obtain from the data information about the choice of distribution , but also to have some reasonable good estimates (often distributed as at-site estimates) of a flood with an associated return period . On the other hand, if the along record is not available the process becomes more complex and the estimates obtained are neither so accurate nor so realistic , especially when the statistical function has several parameters .In this situation the knowledge from grouped stations may be used in the form of regionalization (Kevin ,2008).

2.4.2.2 Probable Maximum Flood approach (PMF)

An alternative approach used to estimate spillway design flood involves the concept of probable maximum precipitation (PMP), and probable maximum Flood (PMF). They are based upon the principle that since every physical process has a nature limit, there is equal to an upper limit to the amount of precipitation that can fall over a specific area in a given duration (Fordland , 1989).

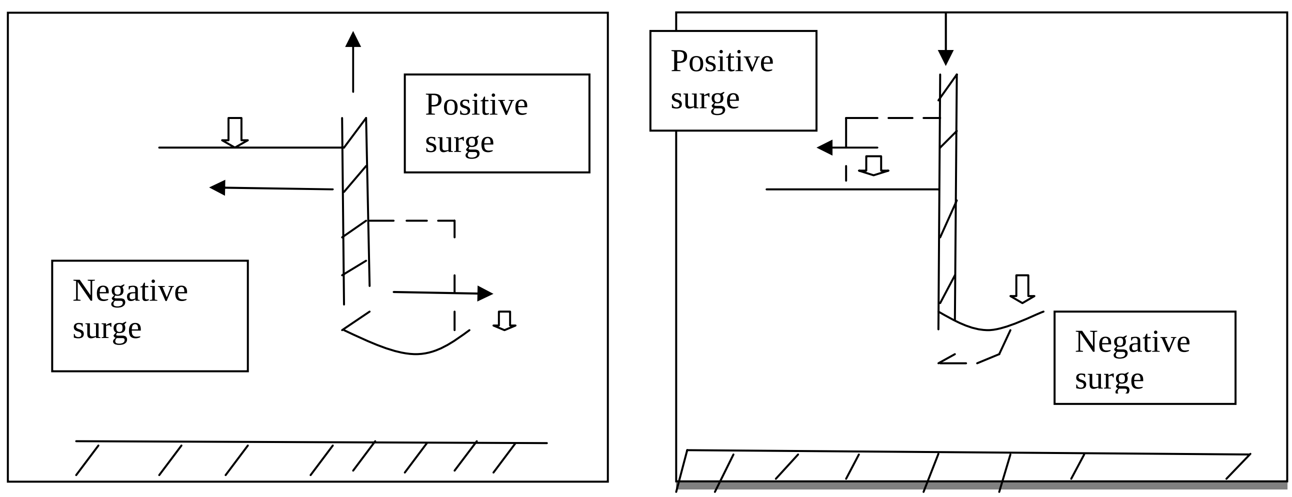
2.5 Unsteady flow

Unsteady flow denotes the flows that change with time .All flows in nature are unsteady because it is almost impossible to maintain all parameters invariant with time. Even for seemingly steady flows, where the mean properties do not change with time, there would be turbulent fluctuations, through generally of a much smaller magnitude than the mean.

Some of the unsteady flow problems can be simplified to an equivalent steady flow problem by a simple change of the reference frame. For example a wave travelling at a uniform velocity and maintaining its shape would be unsteady with respect to a stationary frame of reference since the flow depth and velocity at a cross section would be different before, during, and after the passage of the wave. However, if we move with the wave at the same speed as that of the wave, it would appear to be steady flow with respect to this moving frame of reference(Rajesh, 2009).

2.6 Conversion to equivalent steady flow

If a disturbance in the channel is moving with a constant velocity and retains its shape, it can be converted to a steady –state situation with respect to a moving frame reference. Depending on the nature of variation of depth, we can call it a wave (showing a gradual variation) or a surge (abrupt change in depth).The wave can be a small –amplitude wave in which its amplitude is very small compared to the flow depth or it can be finite-amplitude wave. On the other hand, a surge can be a positive surge, causing an increase in flow depth or a negative surge, causing a decrease in the flow depth. Waves and surge are created in open channels due to various causes, e.g., sudden opening or closing gates, lateral inflow due to runoff, tidal action in an estuary, discharge in hydropower canal (Rajesh, 2009).



(a)Raising of the gate (b) Lowering or closing of the gate

Figure 2.1: Positive and negative surges at gate

2.7 Overtopping

Overtopping failure relates to a dam's inability to cope with an extreme rain event. Presumably, very heavy rainfall which causes a build-up of water capable of overwhelming a dam is not likely to be confined solely to the catchment above the dam and entirely held back until the moment of failure. Rather, a dam-failure flood is likely to be accompanied, before and after failure actually occurs, by flooding downstream and in adjacent, possibly tributary, catchments. In such a situation it would be expected that the emergency services would already be active by the time the failure took place and that some people would already have been evacuated from low-lying areas. Equally, though, evacuation routes may have already been lost (Goran, 2009).

2.8 Sunny day failure

Sunny-day failures, which may occur in many cases have rather different features from an emergency management point of view .Worldwide, Many dams have collapsed through internal erosion (piping), but failure may also result from "impact" events such as massive landslides, earthquakes or even terrorist activity. With the possible exception of piping or landslide-related problems (the latter of which presumably would take place as a consequence of rain-induced rotational slumping along well-lubricated shear planes in mountainous country), all these kinds of failure would be likely to occur with very little if any warning.

2.9 Hazard classification

Hazard potential classification system for dam is based on the probable loss of human life , and the potential for economic loss, and environmental damage , or disruption to the life lines caused by failure of the dam or its appurtenance .This hazard potential classification system for dams recognizes that the failure of any dam or water retaining structure , no matter how small it is , but the consideration given to the a potential of danger in downstream life and property , whenever there is an controlled release of stored water there is hazard (Dam safety, 2004).

Based on Federal Guideline for dam for dam safet , hazard has classified into three groups.

1. LOW HAZARD POTENTIAL

Dams assigned the low hazard potential classification are those where failure or miss -operation results in no probable loss of human life and low economic and/or environmental losses. Losses are principally limited to the owner's property.

2. SIGNIFICANT HAZARD POTENTIAL

Dams assigned the significant hazard potential classification are those dams where failure or miss-operation results in no probable loss of human life but can cause economic loss, environmental damage, disruption of lifeline facilities, or can impact other concerns. Significant hazard potential classification dams are often located in predominantly rural or agricultural areas but could be located in areas with population and significant infrastructure.

3. HIGH HAZARD POTENTIAL

Dams assigned the high hazard potential classification are those where failure or miss-operation will probably cause loss of human life.

2.10 Dam break

Dam failures are often caused by overtopping of the structure due to inadequate spillway capacity, seepage or piping through the dam or long internal conduits, slope failure, earthquake damage, liquefaction of earthen dams or waves generated within reservoir by landslides. Failure of dams are depend on the type of dam. Constructed earth dams do not fail completely nor do they fail instantaneously. While concrete gravity dams also tend to fail by partial breaching as one or more of the monolithic concrete section formed during construction are forced apart by the escaping water , but in case of concrete arch type dams , the assumption of complete and instantaneous failure may closely approximate the actual situation.

A significant and critically important example of rapidly varied, unsteady flow is flood wave generated by the failure of dam .When a dam fails, the impounded water is released, and catastrophic flooding may occur in downstream valley.

2.11 Dam breach analysis

The two primary tasks in the analysis of a dam breach are the prediction of the reservoir outflow hydrograph and the routing of that hydrograph through the downstream valley. Predicting the outflow hydrograph can be further subdivided into predicting the breach characteristics (e.g., shape, depth, width, rate of breach formation) , and routing the reservoir storage and inflow through the breach. The routing tasks—through the breach and through the downstream valley—are handled in most of the widely used computer models with various one-dimensional routing methods. However, the programs differ widely in their treatment of the breach simulation process. Many models do not directly simulate the breach; rather, the user determines the breach characteristics independently and provides that information as input to the routing model.

USB Reclamation (1988) grouped the analysis methods into four categories.

1. **Physically based methods**

   Predict the breach and the resulting breach outflows using an erosion model based on principles of hydraulics, sediment transport, and soil mechanics(channel modification).

2. **Parametric models**

Use case study information to estimate time to failure and ultimate breach geometry. Then simulate breach growth as a time-dependent linear process and compute breach outflows using principles of hydraulics.

3. **Predictor equations**

Estimate peak discharge from an empirical equation based on case study data and assumes a reasonable outflow hydrograph shape.

4. Comparative analysis

If the dam under consideration is very similar in size and construction to a dam that failed, and the failure is well documented. Aappropriate breach parameters or peak outflows may be determined by comparison.

2.11.1 Case study

In 1998, GEI Consultants, Inc .prepared dam breach analyses for four flood control dams in Ventura County, California. GEI developed a step-by-step parametric approach to estimate dam breach parameters. In this approach, they limited the range of judgment to a reasonable universe based on empirical equations developed from past embankment dam failures. Key steps in the analysis were five(walker,1998).

Result of one of the dam , showed that. ''A piping failure mechanism was used in the analysis because the inflow of a 10-year storm event would not overtop the dam. A 10-year storm event was routed through the basin to initiate piping, in accordance with OES guidelines. Results of the dam breach parametric analysis indicate that, with a breach side slope of 1H:1V, the final breach base width would be about 41 feet. Breach development time was estimated to be about one-half hour. The resulting peak breach discharge from the DAMBRK model was about 2,700 cfs''.

2.12 Prediction of outflow Hydrograph

The prediction of the outflow hydrograph is one of the important parameters in dam break analysis and it can further be subdivided into simulating the dam breach formation process and computing the outflow through the breach from principles of hydraulics (Kevin, 2008).

1) Simplified approaches that entirely neglect the breaching process use case study data to develop direct predictions of peak outflow and time required for failure. These predictions may be based on comparisons with one or more very similar dams that have failed (comparative analysis), or they may be based on regression relations that predict peak outflow and time of failure from relevant hydraulic parameters, such as dam height, reservoir storage, and embankment volume (predictor equations). The peak outflow hydrograph predicted using these methods serves as the input to the river routing analysis.

2) A more rigorous approach is to simulate the breach of the dam and the resulting reservoir outflow internally in computer model using a parametric approach. Final breach geometry and time of breach formation are specified, and the breach enlargement is then simulated as a simplified time-dependent process (e.g., linear increase of breach dimensions).

3) Use of a physically based dam breach simulation model that uses principles of hydraulics and sediment transport to simulate the development of the breach. This approach is more difficult, but also offers the potential for more detailed results, such as prediction of breach initiation time and prediction of intermediate breach dimensions as well as ultimate breach parameters.

All three approaches have shortcomings. Comparative analysis suffers from a lack of accurate and comprehensive case study data on a wide variety of dams, especially very large dams. Predictor equations suffer from similar problems, and regression relations based on the available data have high uncertainty. Physically based models suffer from a poor understanding of the mechanisms of breach development and an inability to model those mechanisms and the high energy erosion processes that dominate dam breach.

2.12.1 Breach parameters

Dam break received a good number of studies and researches , the result was a large number of papers , and books , this contributed for having different definitions , formulas and methods , avoiding confusion , these parameters have specific definition .

Breach depth

Is the vertical extent of the breach measured from the dam crest down to the invert of the breach.

Breach side slope factor

The breach side slope factor along with the breach width and depth fully specifies the shape of the breach opening. Accurately predicting the breach side slope angles is generally of secondary importance to predicting the breach width and depth

Breach initiation time

The breach initiation time begins with the first flow over or through a dam that will initiate warning, evacuation, or heightened awareness of the potential for dam failure. The breach initiation time ends at the start of the breach formation phase.

Breach formation

Breach formation is the duration of time between first breaching of downstream face of the dam until the breach is fully formed .For overtopping failure the beginning of the breach formation is after the downstream face of the dam has eroded away and the resulting crevasse has progressed back across the width of the dam width and the dam crest to reach the upstream face.

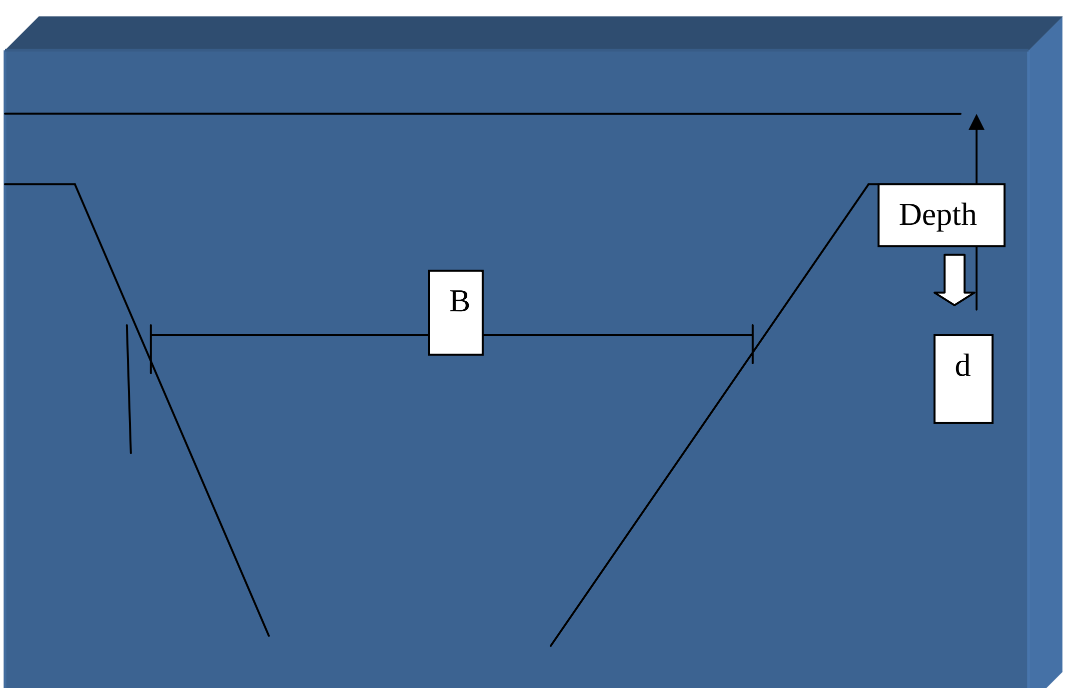


Figure 2.2 : Breach characteristic

2.12.2 Importance of breach parameters

Based on different researches done on dam break analysis follows , the peak out flow , inundation area , and flood arrival time depend on the breach parameters , nature of the reservoir , and formation time .

Singh and Snorrason (1984)

changes in breach width produced large changes (35-87%) in peak outflow for large reservoir and smaller change (6-50%) in peak out flow for small reservoir .Sensitivity to breach depth is relatively small , there is only 20 percent (20%) change in peak out flow over the range of simulation breach depth

Patrascheck and sydler (1984)

Demonstrated the sensitivity of discharge , inundation levels , and flood arrival time to the changes the breach width and breach formation time .For location near the dam , both parameters can have dramatic influence for location well Downstream from the dam . In case of locations well downstream from the dam , the timing of the flood wave peak can be altered significantly by the changes in breach parameter formation time , but the peak discharge and inundation are insensitive to the change in breach parameters .

Clearly accurate prediction of breach parameters is necessary to make reliable estimate of peak outflow and resulting in downstream inundation in close proximity to the dam.

2.12.3 Methods of prediction of breach parameters

Numerous predictor equations for peak discharge and breach parameters have been developed and are summarized in this thesis. The available equations vary widely depending on the analyst and the types of dam failures studied. In general, predictions of breach side slopes have high uncertainty, although this is of secondary importance, since breach outflows are relatively insensitive to side slopes. Predictions of breach formation time also have very high uncertainty due to a lack of reliable case study data; many dams fail without eyewitnesses, and the problem of distinguishing between breach initiation and breach formation phases has likely tainted much of the data (dam safety office, 1998).

Singh and Snorrason (1982)

Provided the first quantitative guidance on breach width. They plotted breach width versus dam height for 20 dam failures and found that breach width was generally between 2 and 5 times the dam height. The failure time, from inception to completion generally between 2 and 5 times the dam height. The failure time, from inception to completion of breach, was generally 15 minutes to 1 hour. They also found that for overtopping failures, the maximum overtopping depth prior to failure ranged from 0.15 to 0.61 meters (0.5 to 2.0 ft).

MacDonald and Langridge-Monopolis (1984)

Proposed a breach formation factor, defined as the product of the volume of breach outflow (including initial storage and concurrent inflow) and the depth of water above the breach invert at the time of failure. They related the volume of embankment material removed to this factor for both earth fill and non- earth fill dams (e.g rock fill, or earth fill with erosion-resistant core). Further, they concluded from analysis of the 42 case studies cited in their paper that the breach side slopes could be assumed to be 1h:2v in most cases; the breach shape was triangular or trapezoidal, depending on whether the breach reached the base of the dam.

Froehlich (1987)

Froehlich developed new prediction equations for average breach width and time of failure. In contrast to his 1987 relations, the new equations are not dimensionless. Both 1995 relations had better coefficients of determination than did the 1987 relations, although the difference for the time of failure relation was very slight Froehlich did not suggest a prediction equation for the average breach side slopes in his 1995 paper, but simply suggested assuming breach side slope factors of Z = 1.4 for overtopping failures or Z = 0.9 for other failure modes. He noted that the average side slope factor for the 63 case studies was nearly 1.0. The data set showed that there are some significant outliers in this regard.

Von Thun and Gillette (1990) and Dewey and Gillette (1993)

Used the data from Froehlich (1987) and MacDonald and Langridge-Monopolis (1984) to develop guidance for estimating breach side slopes, breach width at mid-height, and time to failure. They proposed that breach side slopes be assumed to be 1:1 except for dams with cohesive shells or very wide cohesive cores, where slopes of 1:2 or 1:3 (h:v) may be more appropriate.

Von Thun and Gillette proposed the following relationship for average breach width

$$B = 2.5hw+Cb$$

With *hw* being the depth of water at the dam at the time of failure, and *Cb* a function of reservoir storage as follows

Table 2.1 Von Thun and Gillette, coefficient estimation

Source: Dam Safety 1998

Reservoir size M3	*Cb meters*
$< 1.23 * 10^8$	6.1
$1.23 * 10^8 – 6.17*10^8$	18.3
$6.17 * 10^7 - 1.23 *10^7$	42.7
$> 1.23 *10^7$	54.9

2.13 Model description

2.13.1 Description of Hydrological model HEC-HMS

HEC-HMS 3.4 is hydrological model developed by US corps of Engineers , designed to simulated the precipitation –runoff process of dentritic watershed system .It is designed to be applicable in a wide range of geographic areas for solving the widest range of problems . This include large river basin water supply and flood hydrology , and small urban or natural watershed runoff , hydrograph produced by the program are used directly or in conjunction with other software for studies of water availability , urban drainage , flow forecasting

, reservoir , spillway design , flood damage reduction , flood plain regulation , and system operation .

Capability include a linear quasi –distributed runoff transform (Mod Clark) for use with gridded precipitation , continuous simulation option with either one-layer or more complex five-layer soil moisture method , and a versatile parameter estimation option .The program is a generalized modeling system capable of representing many different watershed . A model of the watershed is constructed by separating the hydrologic cycle into manageable pieces and constructing boundary around the watershed of interest.

The program feature a completely integrated work environment including a database, data entry utilities, computation engine, and results reporting tools. A graphical user interface allows the seamless movement between the different parts of the program.

2.13.2 Hydraulic Model HEC -RAS

It is model developed by Hydrologic Engineer center – River Analysis system (HEC -RAS) US corps of Engineer . HEC -RAS is an integrated system of software , designed for interactive use in a multitasking environment .It allows simulation of one dimension (1D) steady and unsteady flow , water surface profile calculation , and inundation mapping .The system comprises a graphical user interface (GUI) , separate Hydraulics analysis components , data storage and management capabilities , graphics and reporting facilities .

Existing capability include

1- Analysis features : stream flow profile , unsteady simulation , Federal Emergency Management (FEMA) encroachments , split flow optimization sediment transport capacity and bridge scour, dam and levee breaching navigation dam operation , channel modifications , mixed flow regime ,sediment transport , water temperature modeling.

2- Can be used to model both overtopping as well piping failure breaching for embankment dams , additionally the more instantaneous type of failures of concrete (generally occurring from earthquakes) can also be modeled .

3- Geometric Feature : bridge hydraulic , embankment/weir hydraulic , extensive culvert hydraulic (nine types of culvert), multiple opening analysis (bridge and culvert) , inline structure (spillways , gates , and weir) , lateral structure (gates weir , culvert , and rating curves) , storage/bonding areas , hydraulic connections between storage areas , pumping stations , floating ice , levees , extensive data import and export GIS connection .

4- Graphical output : water surface profile plots , cross section , rating curve , stage and flow hydrographs , generalized profile plot of any variable , 3D view of river system , and graphical animation

5- Tabular output : detailed output tables for all structure , summary output table.

Chapter three

3.0 Methodology and data analysis

3.1 Methodology and Materials used

3.1.1 Methodology

Methodology should be done accordingly and systematically. In order to get satisfying result in sufficient period of time.

1) Collecting data and relevant documents

- Hydrological data for the reservoir and river (discharge , water level ,) some of this data or parameters will be used as boundary condition
- Cross sections of the river
- Meteorological data.
- Topography map and digital elevation models (DEM).
- Land use and related data
- Dam Feature data.

2) Data process (filling and checking), for both Meteorological and hydrological

3) Determine the flowing parameters to be used as input parameters to (HEC-HMS)

- Design precipitation
- 1 % chance precipitation
- Reservoir Routing

4) Development of Hydrological model (HEC-HMS) to generate output hydrograph

- Probable maximum flood
- 1 percentage exceed flood

5) Determine breach parameter using Bureau of reclamation (1982), and Von Thun and Gillete (1999) methods

6) Simulate potential dam break scenarios and corresponding water surface profile by using HEC-RAS model

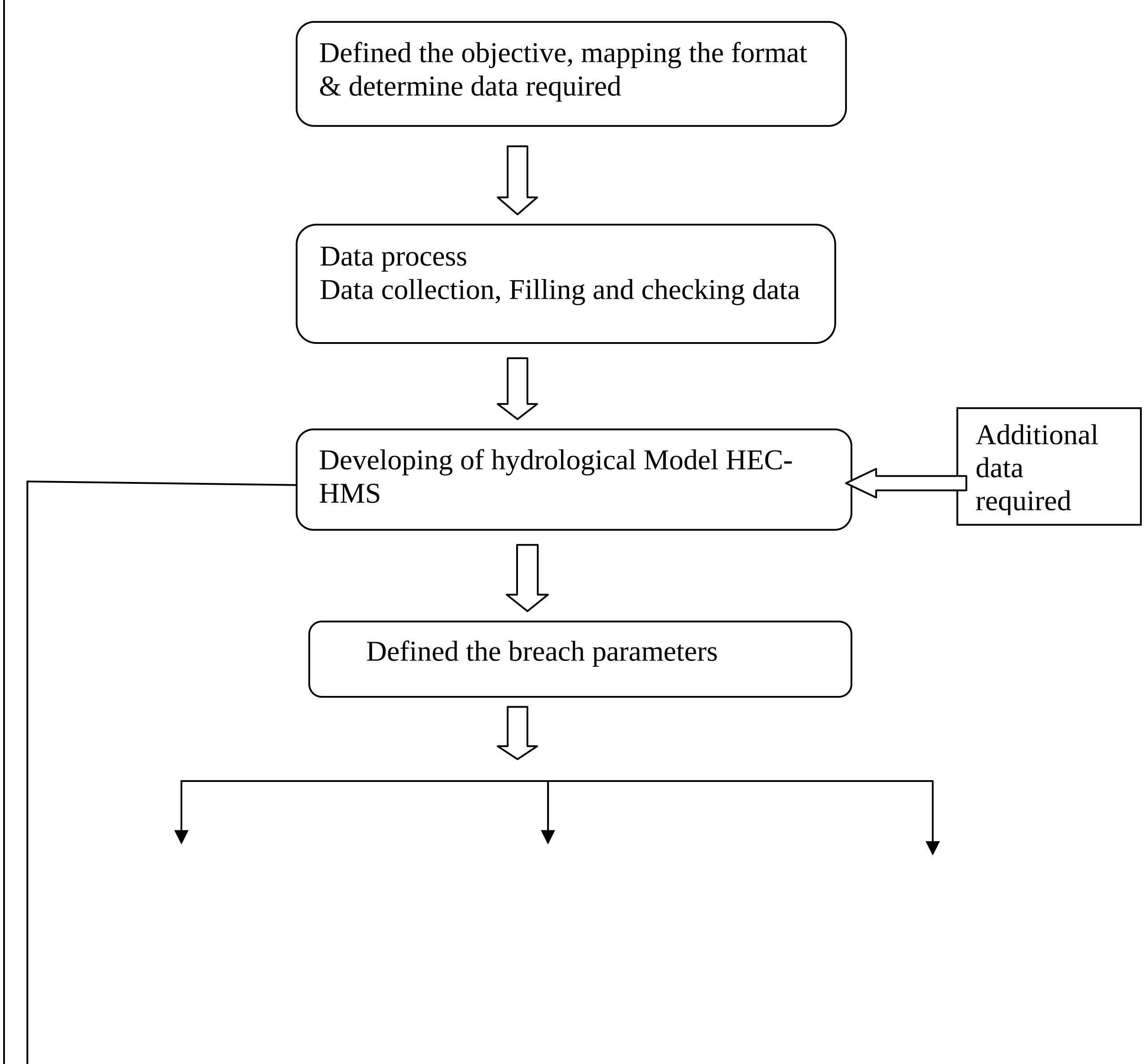
7) Analysis and discussions of the results.

Table 3.0: Model input and output

S/N0	Model Name and software	Input	Output
1	HEC-HMS	➢ Design precipitation ➢ 1 percent precipitation	➢ PMF ➢ 1 percent exceed flood
2	HEC-RAS	➢ Geometric data ➢ Boundary condition ➢ Breach parameter ➢ Dam & Spillway data ➢ Reservoir routing ➢ PMF ➢ 1 percent exceed flood	➢ Water surface profile ➢ peak discharge for any scenarios ➢ water volume

		➢ Flow data	
3	HEC-DSSVue	➢ HEC-HMS result	represent HEC-HMS result
4	Arc-GIS	➢ Topographic map	Sub basin of the study area

Methodology chart



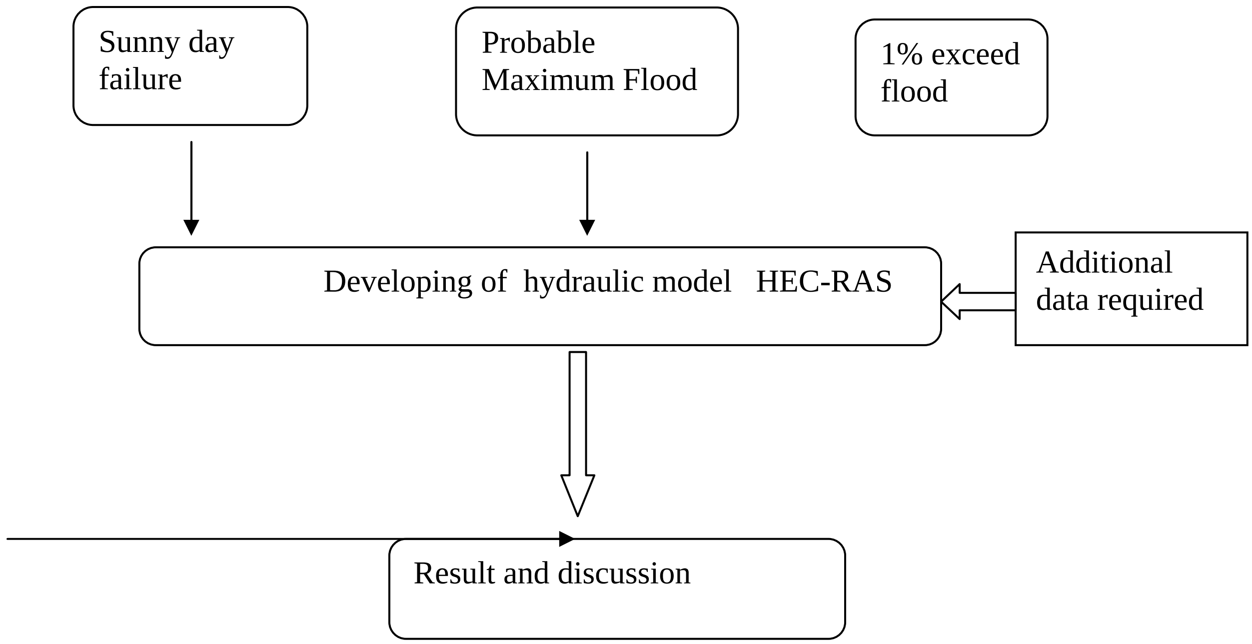


Figure 3.0: Methodology chart

3.1.2 Materials used

In the area of water resource planning , designing , and management . Completed data sets are required for many variables such as stream flows , rainfall , evaporation , and temperature . Data and tools used are :

- GIS data
- Hydrological data
- Dam , reservoir and spillway data
- Meteorological data
- Hydrological Model HEC-HMS
- Hydraulic Model HEC-RAS
- Arc-GIS software

3.1.2.1 GIS data

GIS data , and most of maps including digital elevation model (D.E.M) with resolution 90X90 M. Topography map, Contour map , land use , land cover , gauging and meteorological stations maps. Are obtained from Ministry of water resources department of GIS.

3.1.2.2 Hydrological data

In this research, Hydrological data is considered one of most important parameter. Data has been collected from two different sources

A) Department of hydrology in Ministry of water Resources.

B) Previous research and report done in the basin

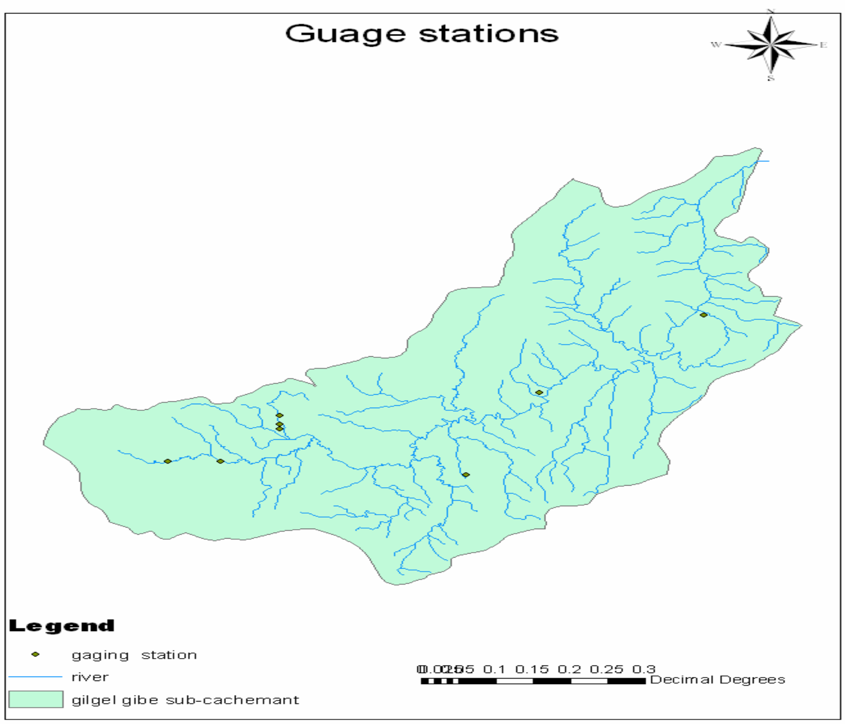


Figure 3.1: Gauging stations within the basin

Table 3.1: Summary for the of stations were used with daily data

S/No	Station name	Station Number	Starting records	Ending records	Missing data	Location	Area in square Km
1	Awaita Babu	091030	1989	2003	yes	upstream	-
2	Ghibe Seka	091017	1980	2005	yes	Upstream(river stage)	280.4
3	Bidru Awana-Sokurul	091019	1981	2005	yes	downstream	41
4	Kito-Jimma	091023	1982	2005	yes	upstream	85
5	Awaitu-juma	091024	1982	2005	yes	upstream	72
6	Bulbul-Serbo	091032	1986	2005	yes	upstream	526
7	Gilgle-Ghibe Asenabo	091008	1969	2008	yes	upstream	2966

3.1.2.3 Hydraulic data

River length collected from hydrological report (1997), Gilgle Hydrometrical project (EELPA)

3.1.2.4 Meteorological data

Meteorological data are obtained from Ethiopia Meteorological Agency. From ten stations in upstream only one stations is first class , the rest of stations are third and fourth classes. Because of some constrain getting long time series for meteorological data was not possible. Therefore, most of the stations are with five years daily data .Other stations have more than five years data are obtained from others reports and studies done in the basin .

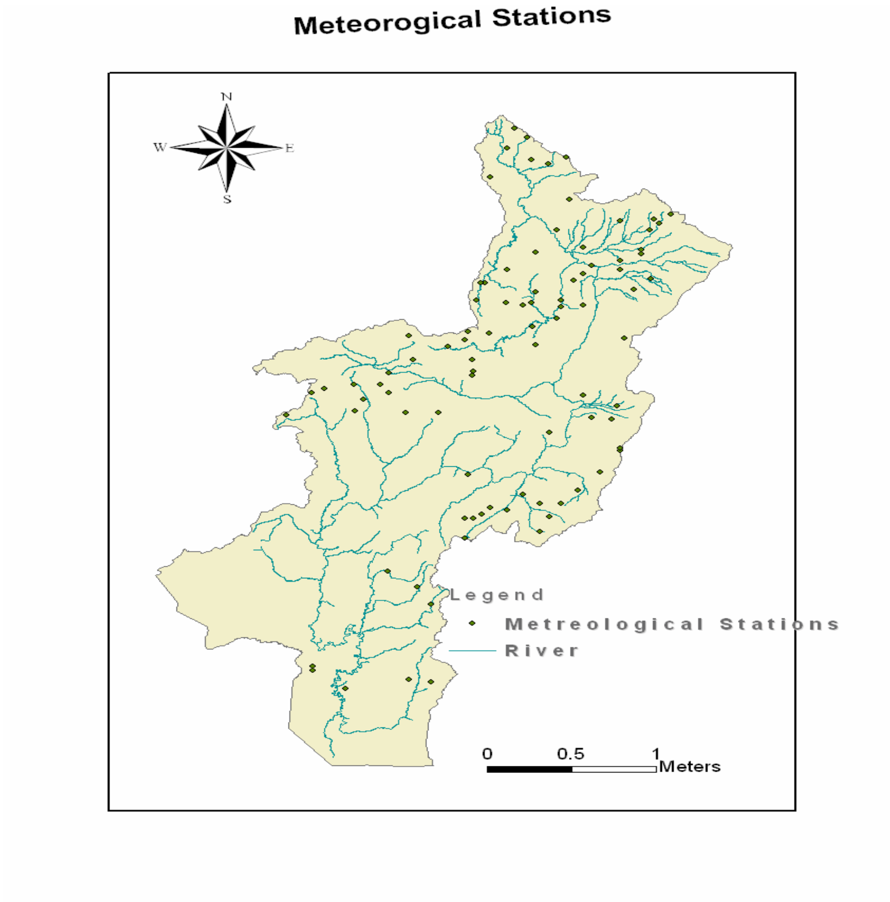


Figure 3.2: Meteorological Stations

3.1.2.5 Dam, reservoir and spillway data

Dam data collected from two sources Ethiopia Electrical power corporation (EEPCO), and research done in the basin .

Table 3.2: Dam Feature

Type of Dam	Rock fill
Dam dimension	Measurement
Dam height (D/S toe to top of Dam)	40 M
Dam crest elevation	1675 M asl
Dam bottom elevation	1635 M asl
Length of Dam	1700 M
Width of Dam crest	7 M

Upstream slope	1V: 2H
Downstream slope	1V: 2H
Drainage area for reservoir	4225 Km^2
Spillway	
Number of spillway	1
Number of gates	4
Spillway dimension	12 X 4 M
Spillway Elevation	1671 m asl
Surface area	
At HRWL	51 Km2
At top of dam	-
Storage	
At spillway crest	51 km3
At top of dam	-
Design	
PMP	-
PMF	-
Q100	14250 m3/s

Source: Ethiopia Electrical power corporation (EEPCO)

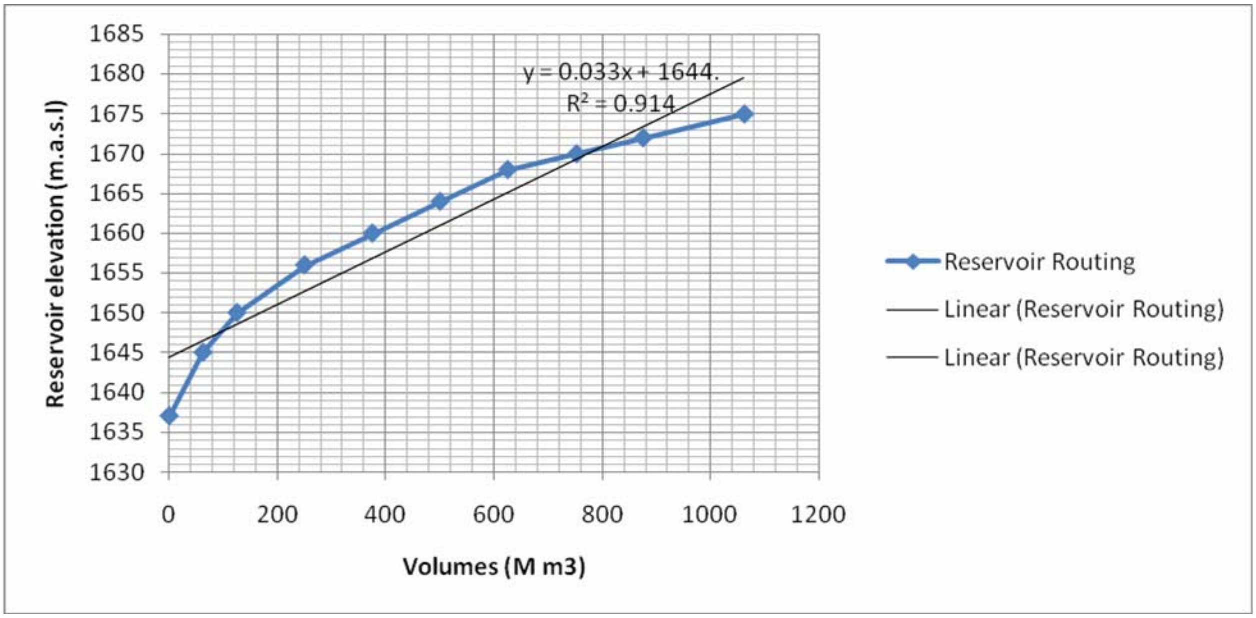


Figure 3.3 (a): Reservoir Routing

Source: Ethiopia Electrical power corporation (EEPCO)

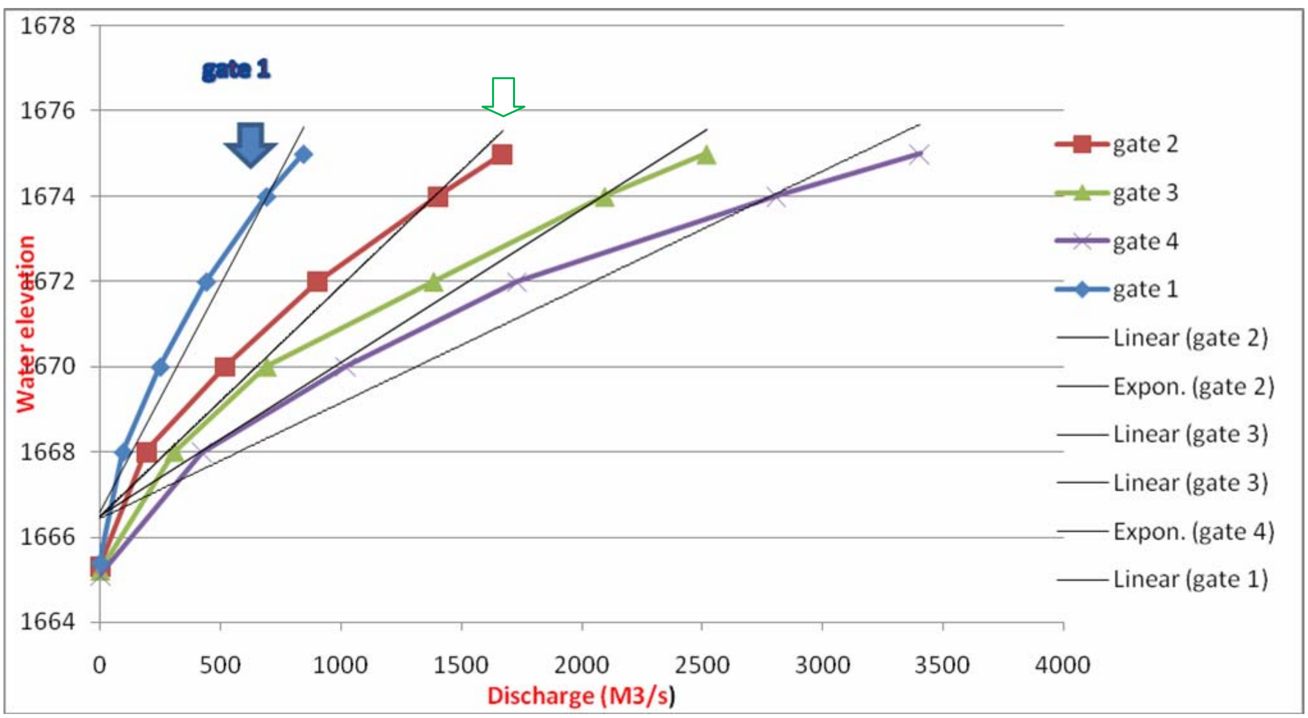


Figure 3.3 (b): Spillway Rating

Source: Ethiopia Electrical power corporation (EEPCO)

Reservoir bottom elevation is 1635 m.a.sl as shown in Figure 3.3 (a) .The relation between reservoir elevation and storage water is vary from bottom to the top .From bottom and for the first two points with the same interval 5 m.a.s.l 1640 to 1645 and 1645 to 1650 , water storage is the same 40 million cubic meter .While from 1650 to 1655 , 1655 to 1660 , and 1660 to 1665 water storage is 120 million cubic meter. On the top of the reservoir with the same interval 5 m.a.s.l , from 1670 to 1675 water storage is 160 million cubic meter.

3.2 Filling and analysis data

3.2.1 Filling Hydrological data

It is necessary, if it is not must to have long time series data. Unfortunately, records of hydrological data are short and often have missing observation. Attracted by the importance of estimating missing hydrological data or extending of the short records. Various models and techniques are available to deal with this problem such as regression analysis, time series analysis, average method, and artificial neural network. For this research regression was used for estimating missing data and extension of data . Gilgel Ghibe @Asendabo was the friend station for most of the stations because it has long time series (1969-2008) about 40 years compare to the rest of stations ,and no much missing data compare to the rest of the stations . The correlation coefficient range from good to acceptable. Table 3.3 show the result of filling the missing and generating data.

Table 3.3: Regression summary for the stations

S/No	Station Name	Friend station	Regression Equation	Correlation coefficient
1	Awaita Babu	Gilgel @Asendabo	QawaituBabu=0.04640Qasend +0.1957	$R^2 = 0.782$
2	Kito@Jimma	Gilgel @Asendabo	QaWaitu@Jim = 0.71 Qasend - 0.23	$R^2 = 0.670$
3	BulBul-Serbo	Gilgel @Asendabo	QBulbul = 0.247QAsend + 0.1089	$R^2 = 0.867$
4	Gilgel Ghibe @Seka	Gilgel @Asendabo	QG.seka=0.03225QAsend+0.572	$R^2 = 0.536$
5	Awaitu@Jimma	Gilgel @Asendabo	Qawait=0.0281QAsend - 0.0907	$R^2 = 0.73$
6	Gilgel Ghibe @Asendabo	Gilgel @Asendabo	Average method	

3.2.2 Analysis of hydrological data

3.2.2.1 Moving average

Has been applied for showing the variations and to indicate weather trend or cyclic pattern are available . The moving average is usually constructed with moving period from3 to 5 year. In this research three years was been selected through applying the equation

$$Y_{n-1} = \frac{(X_{n-2}+X_{n-1}+X_n)}{3} \longrightarrow \quad 3.4$$

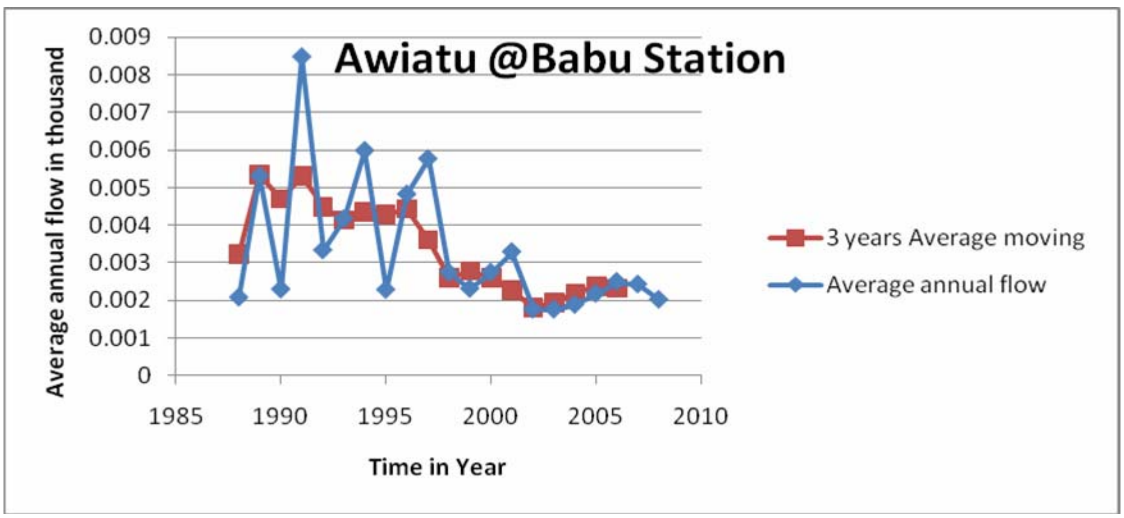


Figure 3.4: 3 years moving average for Awiatu @Babu Station

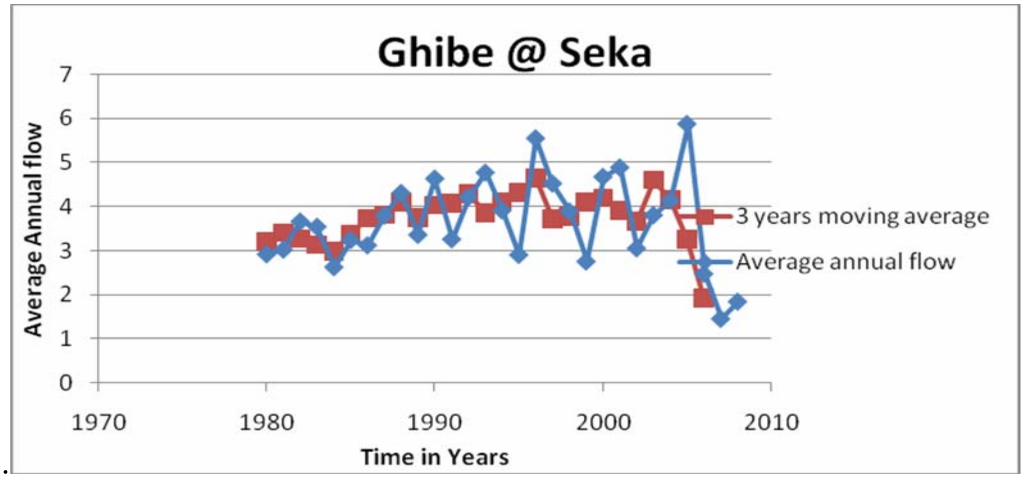


Figure 3.5: 3 years moving average for Ghibe Seka

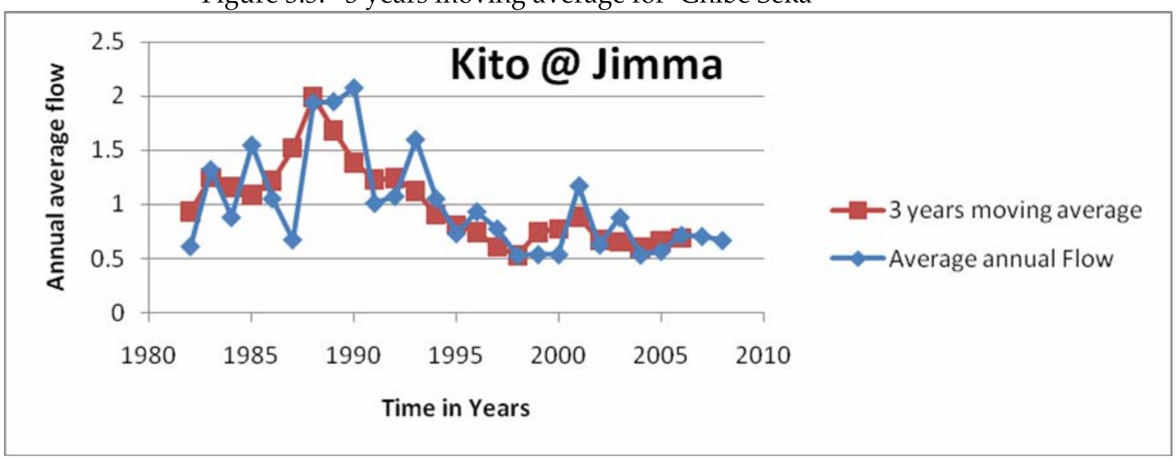


Figure 3.6: 3 years moving average for Kito @Jimma station

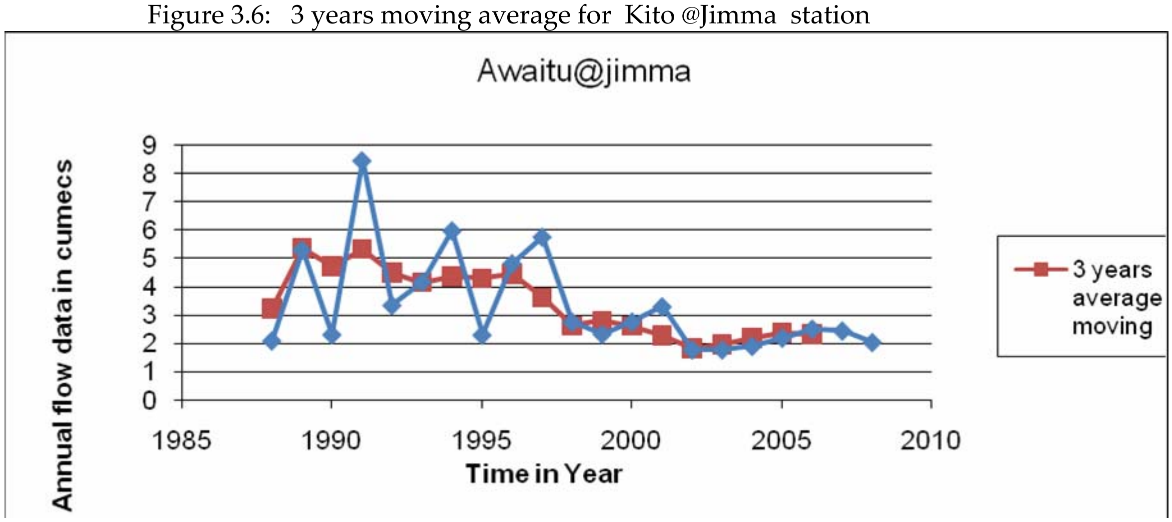


Figure 3.7: 3 years moving average for Awatu station

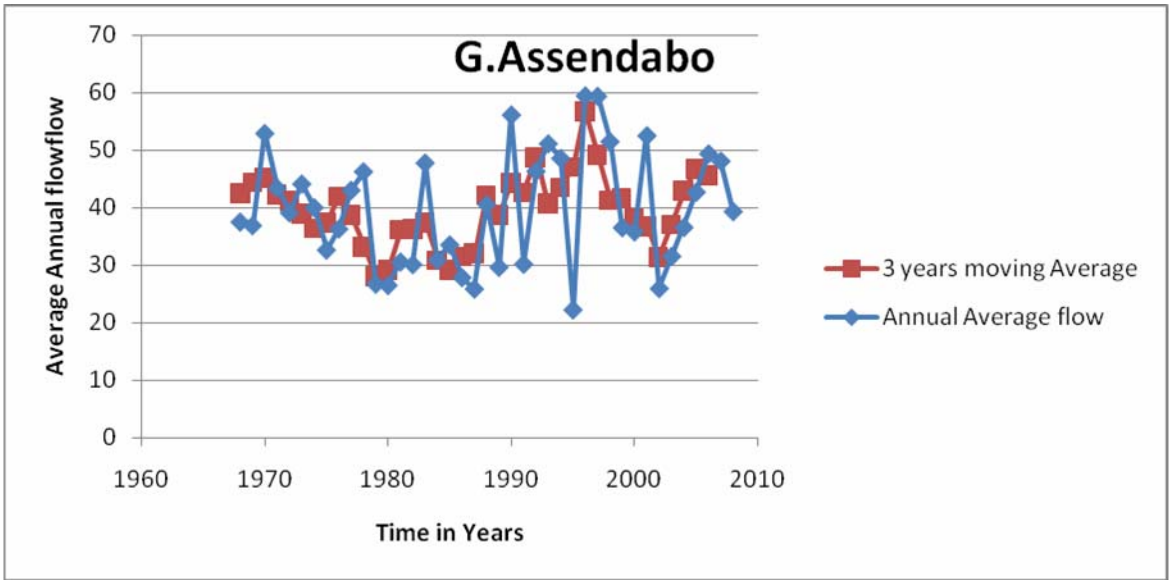


Figure 3.8: 3 years moving average for G.Assendabo

The moving average curve as showed, from figure 3.5 to figure 3.9. It is superimposed over the original flow series, through the variations in the original data are smoothened out to some extent in the moving curve. No apparent trend or cyclist is visible in the moving average.

Gibe @ Assendabo station

The station has large flow compare to the rest of the stations . The station is vary in average annual flow from year to year with different rate and direction , for example from 1970 to 1971 there was increasing .While from 2000 to 2001 , there was decreasing .

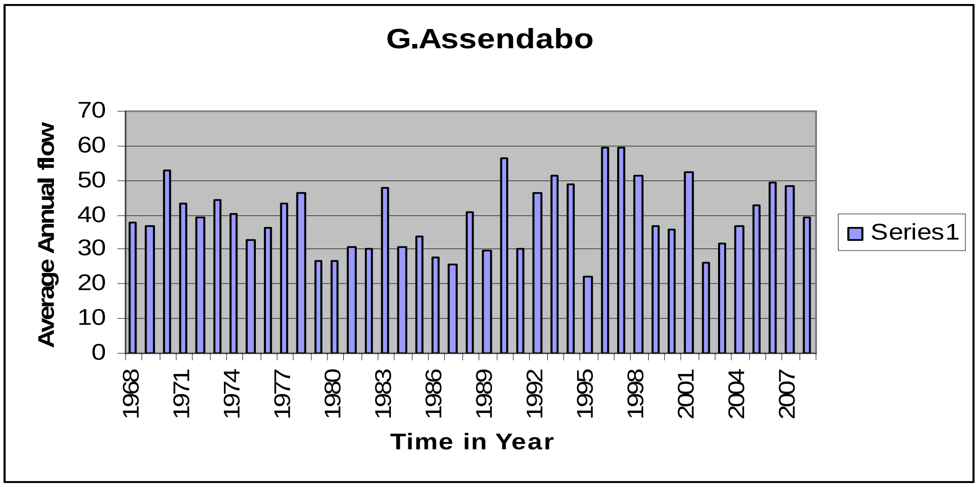


Figure 3.9 (a): Average Annual flow for Gibe @Assendabo station

Awaitu@Jimma station

The station can be groubed with the lower station , in average annual flow .There is no much change from year to year for the last 20 Years ,except some few years there was some change for example 1991 **.**

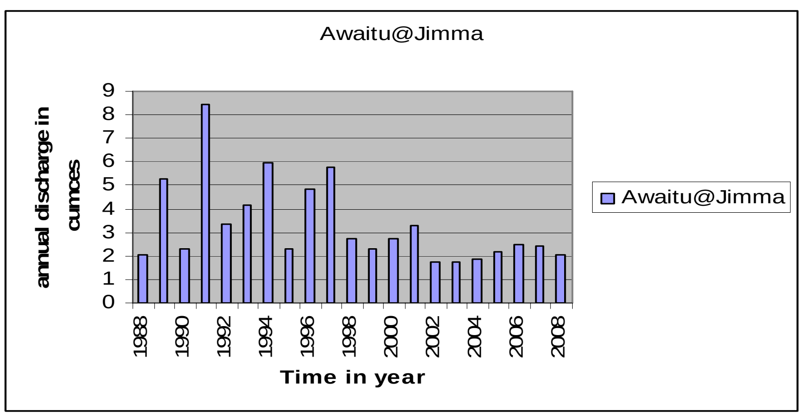


Figure 3.9 (b): Average Annual flow for Awaitu @Jimma station

Bulbul-Serbo station

Bulbul –Serbo with drainge area estimated by 520 KM^2 , can be consider as intermidle station based on average annual flow measured from (1986 to 2005) .Annual flow is vary from year to year and one important observation that there is aseries decreas in annual flow in last three years .

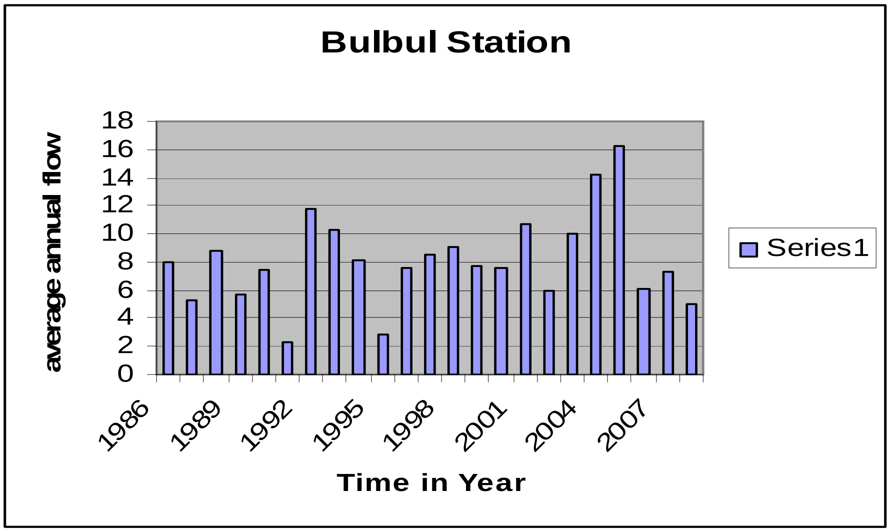


Figure 3.9 (C): Average Annual flow for Bulbul-Serbo station

Awaitu @Babu Station

The characterestic of the station is smilar to Awaitu @Jimma , that there is no much change from year to year except some years .

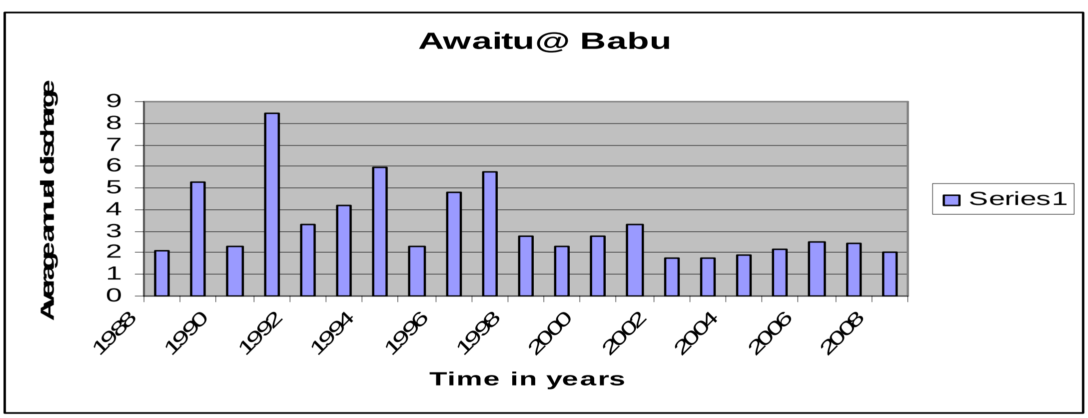


Figure 3.9 (d): Average Annual flow Awaitu @Babu station

Kito @Jimma

Kito station with flow data from 1982 to 2008 showed that , can be grouped into two higher group from 1982 to 1994 , and lower group from 1995 to 2008 .

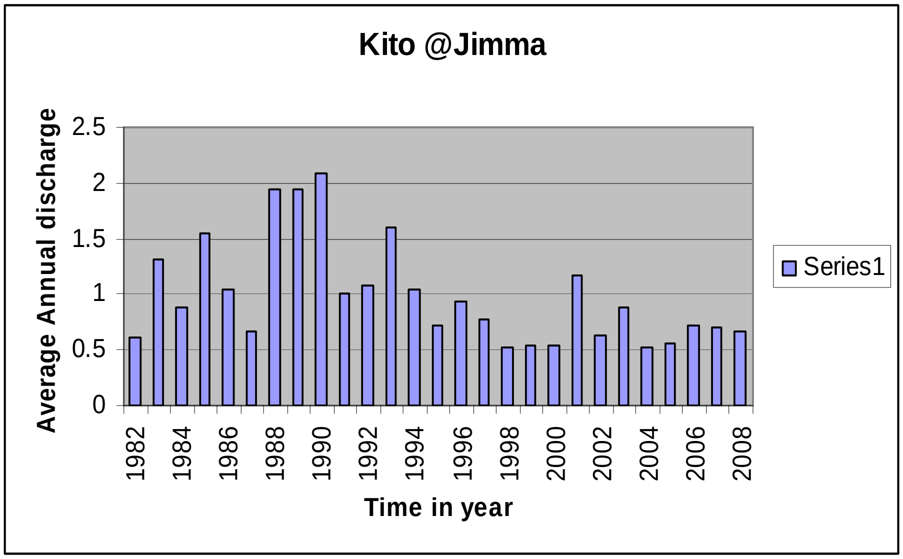


Figure 3.9 (e): Average Annual flow for Kito @Jimma station

Gibe seka Station

The variation of flow data is not that much , except for the last three years , the flow tends to reduced .

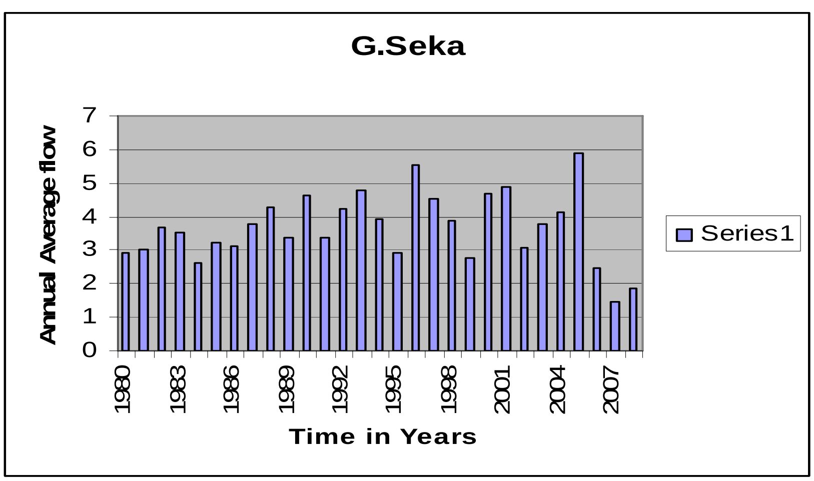


Figure 3.9 (f): Average Annual flow for G.Seka station

2) Flow duration curve

A flow duration curve developed for Assendabo station to characterize the variability of stream flow during the time . From continuous record of daily flow data. Flow duration curve draw with two main scenarios . Annually flow duration curve with records of 40 year, developed by using calendar year method. Where average monthly rates of the years arranged in the descending order . Then the discharge of the wettest month of the year entered in the first row, and that of the driest month in the last row .Similarly , the average monthly rates of all other years fig 3.10 . The figure shows clearly that the curve can divided into two parts .Steep part start from (0 to 35) and flat from (35 to 100) . The second scenarios , where the flow duration curve is computed for year 2008 (Monthly and daily) fig 3.11 showed the percentage of time that certain values of discharge daily ,or monthly are equaled or exceeded . Generally the shape of the flow duration curve depend on both the observation period and the number of data used in the analysis . Mean monthly data generally yield a much steeper curve than annual data.

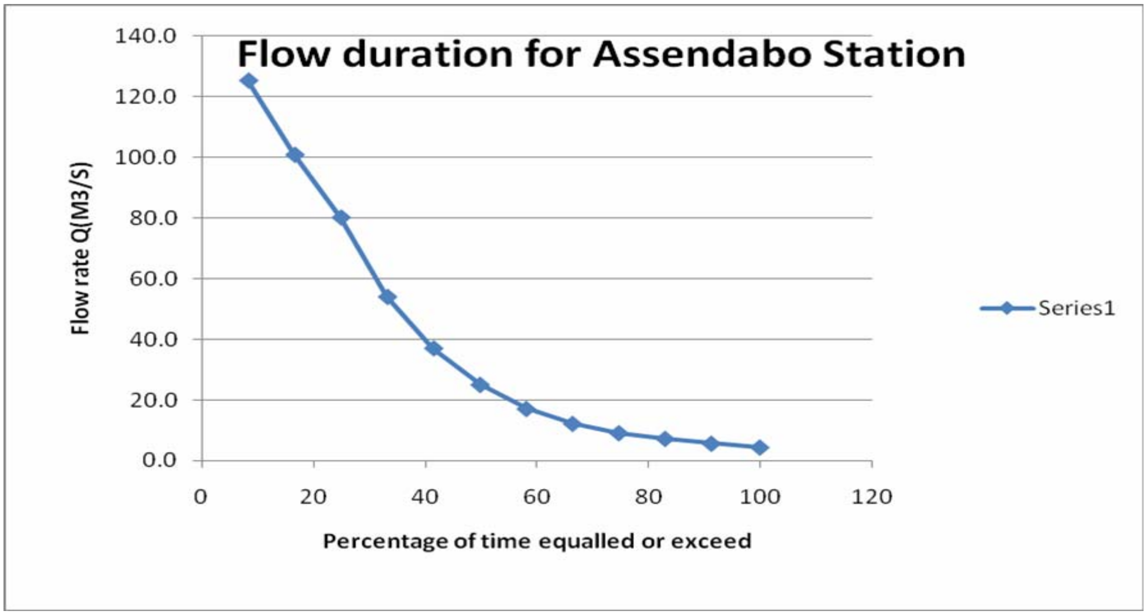


Figure 3.10: yearly flow duration curves for Assendabo station

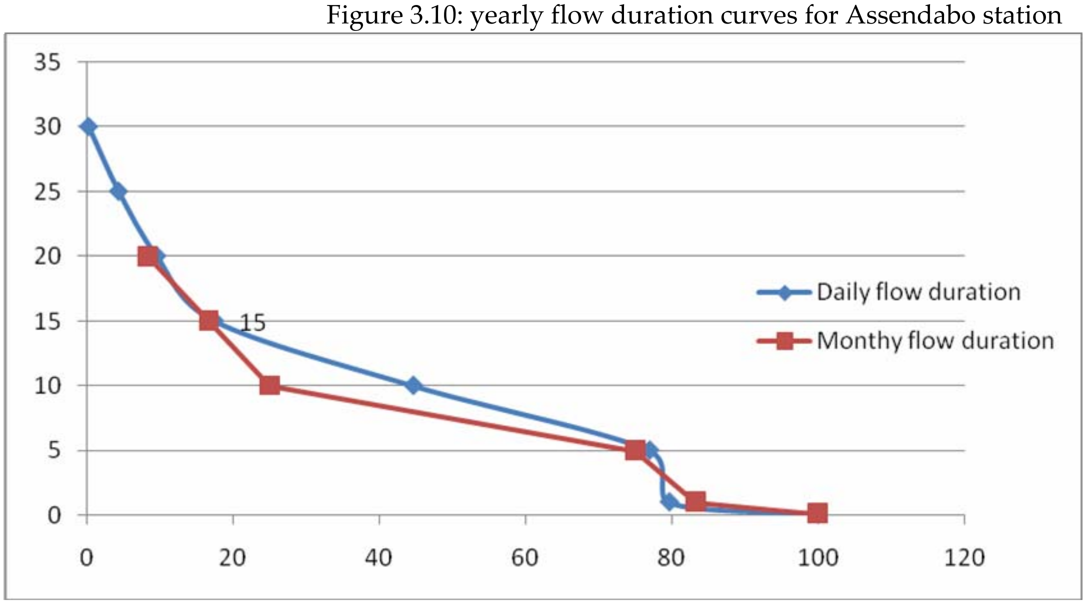


Figure 3.11: Monthly and daily flow duration curve

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3.3 Filling and analysis of Meteorological data

3.3.1 Filling meteorological data

Within the short time series data, missing data were available. This is not a problem because of possibility of calculating this missing data, by using one of the following methods

- Arithmetic mean Method
- Normal ratio method
- Graphical method
- Long term mean rainfall of new station method

From the Four methods knowing for filling the missing rainfall data . Normal ratio method used because of two reasons .Firs the annual precipitation at index stations differs by more than 10 % from the rainfall data is missed . For example annual rainfall in year 2004 was 1468.1 mm, 1442mm , 1182.8 mm, 1188.5 mm, and 1481.5 mm for stations Jimma , Serba , Omo -Nade, Sekoru , and Saja school respectively . And the second reason , more than three adjoining stations rainfall data are available.

In this method , the missing rainfall is determined by weighting the normal annual precipitation, Of each index station. If Na , Nb , and Nc are the normal annual precipitation of index stations A , B , and C respectively . And Nx is the normal annual precipitation of station X which rainfall data (Px) is missed , and Pa , Pb , and Pc are the rainfall of A , B , and C stations . Then the missing data can be calculated by using the formula

$$P_x = \frac{N_x}{3\left[\frac{P_a}{N_a}+\frac{P_b}{N_b}+\frac{P_c}{N_c}\right]} \longrightarrow \qquad 3.5$$

3.3.2 Analysis of Meteorological data

Analysis of observed rainfall data with the normal methods such as double mass curve technique was not possible .In absent of long time series which will require for accumulated mean annual rainfall.

3.3.2.1 Development of Design precipitation

PMP is defined as theoretically the greatest depth of precipitation. For a given duration that is physically possible over a given size storm area. At a particular geographical location at a particular time of year.

There are two main methods for estimating PMP. Including meteorological and statistical approaches, but due to shortage of data for applying the first method. There was a great tendency to adopt the second method known as statistics method (**Hershfield approach**) .Which is dealing with yearly maximum precipitation with general storm rather than local storm . The method required maximum daily rainfall at the observation points.

$$X_{pmp} = X_{av} + K_m\sigma \longrightarrow \qquad 3.6$$

X_{pmp} is PMP for a given set of data

Xav mean of annual maximum

Km is a factor that depends on rainfall duration and varies inverse proportionally with the size of P (WMO 1986). For 24 hour rainfall Xav = 50 mm corresponding to Km value 17.5, where Xav = 100 mm corresponding to Km =15.

Hirschfield's method generally tends to give lower PMP value than meteorological method (Abdullah2004).

Highest observation annual maximum rainfall is key parameter. Table 3.4 shows summary of the ten (10) stations .

Table 3.4 (a): Monthly and Daily maximum Rainfall for Bita Station

Year	Jan	Feb	Mar	Apr	May	June	Jul	Aug	Sep	Oct	Nov	Dec	Max rainfall	Day of Max rainfall
2004	95	51	57	281	306	178	116	174	217	143	79	69	45.8	12 Aug
2005	24	1.4	149	143	252	136	169	134	225	137	51	2.8	50.3	17 Aug
2006	34	67	202	117	286	226	170	133	210	167	100	109	38	27 Sep
2007	95	51	107	179	240	168	152	227	257	99	128	2.3	45.3	22 June
2008	64	36	48	235	312	117	233	306	194	231	56	63	44	27 Oct

Tabl 3.4 (b): Monthly and Daily maximum Rainfall for Bonga station

Year	Jan	Feb	Mar	Apr	May	June	Jul	Aug	Sep	Oct	Nov	Dec	Max rainfall	Day of Max rainfall
2004	32	55	59	179	59	179	241	203	241	279	150	33.4	48.4	27Aug
2005	34	40	158	164	319	202	173	179	185	140	90	0	49.2	3May
2006	31	68.5	156	89	216	186	288	206	184	148	129	109	59.1	10Jul
2007	117	40	97	199	295	276	117	216	189	87	56	0	43	3May
2008	32	55	59	179	214	203	241	279	150	245	78	33.4	39.4	6Jun

Table 3.4 (c): Monthly and Daily maximum Rainfall for Shedature station

Year	Jan	Feb	Mar	Apr	May	June	Jul	Aug	Sep	Oct	Nov	Dec	Max rainfall	Day of Max rainfall
2004	93	56	89	136	221	204	156	101	133	190	22	127	20.2	5 Feb
2005	137	20.4	185	135	350	214	84	98	254	138	82	0	32.2	21 Mar
2006	31	80	193	158	216	206	422	197	366	190	22	0	38.1	23Sep

2007	104	50	87	129	206	391	358	423	573	174	26	11	40.1	19 Sep
2008	32	39	180	343	457	307	563	222	669	172	60	42	53	7 Sep

Table 3.4 (d): Monthly and Daily maximum Rainfall for Babu station

Year	Jan	Feb	Mar	Apr	May	June	Jul	Aug	Sep	Oct	Nov	Dec	Max rainfall	Day of Max rainfall
2004	50	12	38	139	121	254	271	309	277	161	111	47	53.2	13 Aug
2005	14	79	132	236	312	141	228	289	105	103	31	103	50.5	13Aug
2006	34	78	198	61	241	251	359	339	292	177	49	14	63.7	22May
2007	28	63	72	81	314	233	282	270	335	80	1	0	37.2	29 June
2008	14	47	56	83	331	332	385	311	380	107	73	50	91	22June

Table 3.4 (e): Monthly and Daily maximum Rainfall for Assendabo station

Year	Jan	Feb	Mar	Apr	May	June	Jul	Au g	Sep	Oct	No v	Dec	Max rainfal l	Day of Max rainfall
2004	45.2	7.7	72	118	67	216	242	267	127	52	28	37	46	17 June
2005	29.9	1.9	98	166	185	9.9	175	135	176	14	0	0	48.5	20 May
2006	7.4	40.6	176	116	119	212	278	138	69	120	37	25	72.8	9 Oct
2007	48.6	45	99	94.3	75.5	159	212	189	185	11	1.7	0	44.3	30 July
2008	10.3	48.6	52	141.8	226	157.6	205	183	144	138	64	0	53.3	28 Oct

Table 3.4 (f): Monthly and Daily maximum Rainfall for Omo-Nade station

Year	Jan	Feb	Mar	Apr	May	June	Jul	Aug	Sep	Oct	Nov	Dec	Max rainfall	Day of Max rainfall
2004	34	0	63	72	55	202	220	253	131	80	29	44	36.2	24 Oct
2005	39	4.2	92	60	37	160	259	141	113	121	0	0	40.2	17 Aug
2006	16	69	123	151	94	170	219	112	58	60	67	80.2	60.3	20 Oct
2007	78	78	83	108	96	239	326	326	261	294	64	0	50.2	25 Sep
2008	4.2	0	32	92	152	209	150	580	466	272	0	0	67.7	2 Sep

Table 3.4 (g): Monthly and Daily maximum Rainfall for Jimma station

Year	Jan	Feb	Mar	Apr	May	June	Jul	Aug	Sep	Oct	Nov	Dec	Max	Day of

													rainfall	Max rainfall
2004	51	28	46	131	162	128	216	219	201	133	67	84	46.2	7 Aug
2005	45	0.5	194	141	173	177	273	228	229	68	30	0	60.9	14 mar
2006	16	77	182	110	212	207	327	240	170	91	128	101	35.6	25 Nov
2007	37.5	51	104	122	196	143	247	177	256	51	5.9	0	44.6	24 Jul
2008	34	12.3	39.4	113	249	238	210	237	133	186	93	0	48.1	10 Jun

Table 3.4 (h): Monthly and Daily maximum Rainfall for Saja School station

Year	Jan	Feb	Mar	Apr	May	June	Jul	Aug	Sep	Oct	Nov	Dec	Max rainfall	Day of Max rainfall
2004	71.2	17.1	39	8	146	56	217	243	396	161	99	13	60	26-Sep
2005	70	1.7	141	104	193	161	202	203	199	122	22	0	66.5	8-Oct
2006	37	71	106	143	159	225	291	265	74	58	38	40	42.8	25-Apr
2007	41	11.5	68	87	196	261	192	149	368	15	0	0	46.3	6 Jun
2008	0.2	6.3	30	72	228	159	631	117	204	193	78	0	104.2	19 Jul

Table 3.4 (k): Monthly and Daily maximum Rainfall for Sekoru station

Year	Jan	Feb	Mar	Apr	May	June	Jul	Aug	Sep	Oct	Nov	Dec	Max rainfall	Day of Max rainfall
2004	49	26	37	150	63	130	182	279	152	82	17	23	31.6	2-Oct
2005	103	31	103	105	289	228	141	312	236	132	79	1.1	50.5	13-Aug
2006	34.4	78	198	61	241	251	359	339	292	177	49	14	63.2	22May
2007	28	63	72	81	314	233	233	282	270	336	80	0	37.2	29Jun
2008	14	47	56	83	331	332	385	311	380	107	73	50	90.9	22Jun

Table 3.4 (l): Monthly and Daily maximum Rainfall for Serbo station

Year	Jan	Feb	Mar	Apr	May	June	Jul	Aug	Sep	Oct	Nov	Dec	Max rainfall	Day of Max rainfall
2004	52	30	115	102	185	133	232	286	174	47	24	60	35.4	16 March
2005	27	0.8	134	78	145	231	183	162	191	191	32	0	72.2	21May
2006	10.4	92	179	145	157	215	335	168	198	124	102	42	42.2	2June
2007	52	94	81	91	204	239	277	160	123	24	1.5	0	106.9	14June
2008	75	7.2	30	111	191	235	206	177	116	92	78	0	61	30June

One day annual maximum rainfall values for ten stations were analyzed. To extract the main parameters that can be used to calculate later on probable maximum precipitation (PMP) .For the three sub basins upstream of the study dam , Table 3.5 shows these parameters.

Table 3.5: Statistical parameters

Average (X_{av})	Mean Deviation (M.D)	Standard Deviation (S.D)	Coefficient of variation (σ^2)	Variance (C_v)
75.5	16.6	19.4	375.6	26%

parameters estimated in table 3.5 will apply into equation 3.6 , for getting PMP for three subbasins .Table 3.6 shows the result

Table 3.6: Values of PMP and Km for subbasins

Sub basin	Maximum rainfall	K_m	PMP
Upper subbasin	91.1	15.45	392
Upper left subbasin	67.7	16.62	390
Upper Right subbasin	108.1	14.58	391

As mentioned in section 3.1.2.6 that from ten stations within the catchment only one station first class .Even the data of this station was not in form intensity , duration , and the frequency or clear return period intensity such as 10, or 20 minute .Estimating intensity duration curve was not possible. Therefore these hyetograph is developed by interpolation.

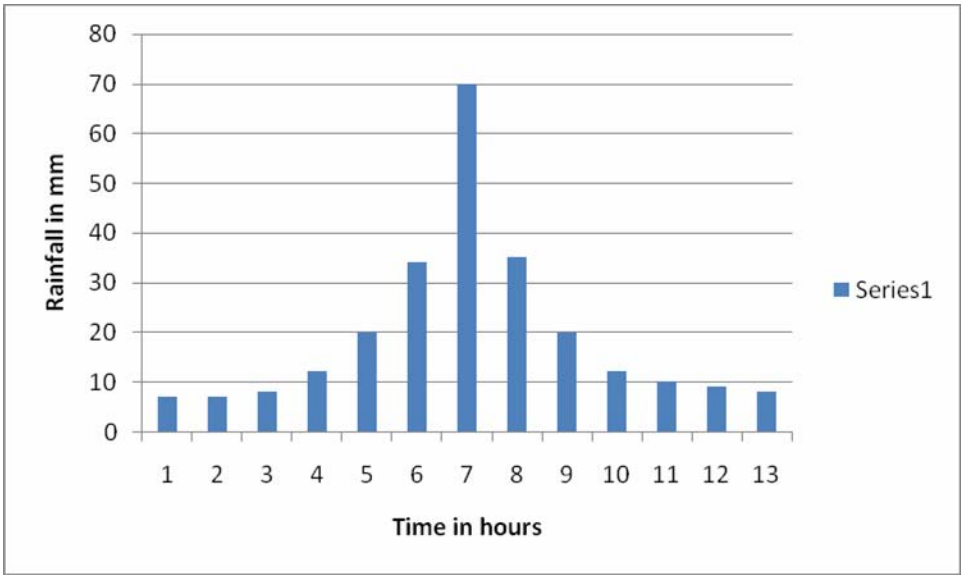

Fig 3.12: Hyetograph for Upper subbasin

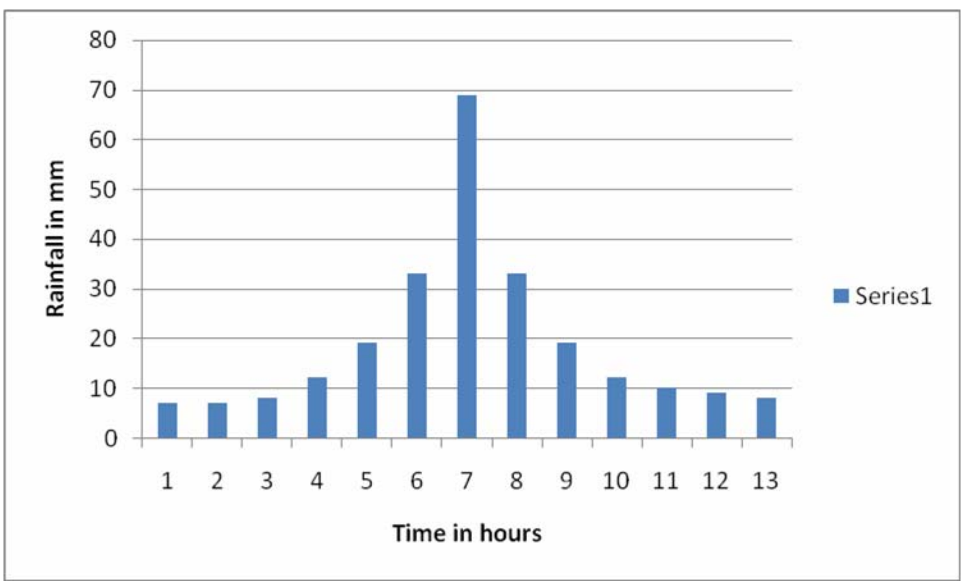

Fig 3.13: Hyetograph for Upper Right sub basin

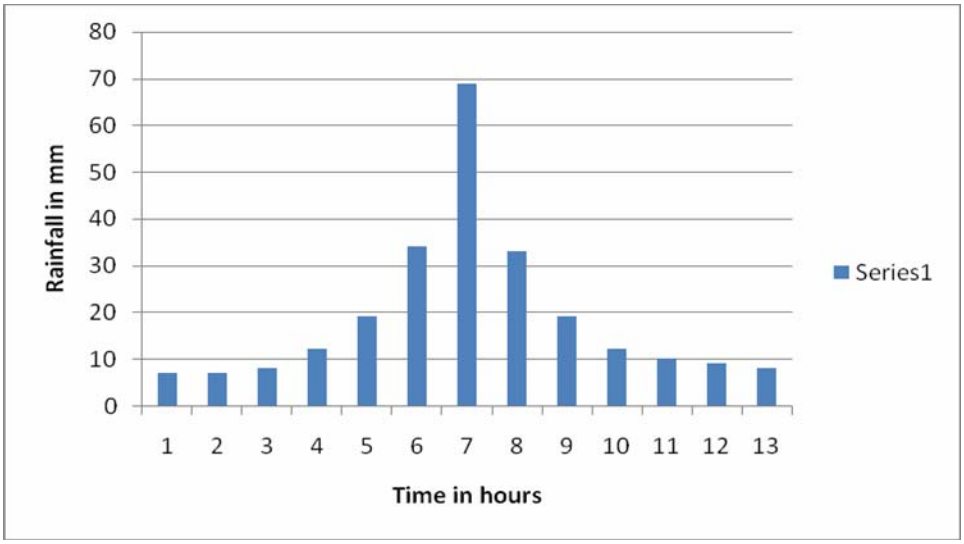

Fig 3.14: Hyetograph for Upper Left sub basin

3.3.2.2 1% percent chance (100 year)

1% percent chance is developed by using the data from Sekoru station. Because it has long time series data (23 year) compare to the rest of the station. **Weibull's** formula applied to estimated 100 year rainfall. This value will be used in HEC-HMS model to developed 1 %chance flood after transfer to hyetograph , and will be distributed uniformly over subbasins.

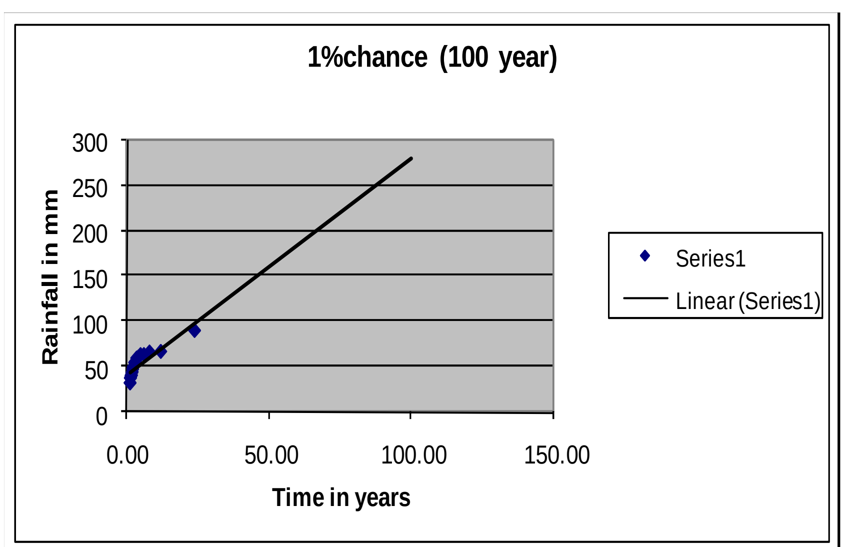

Fig 3.15:   Estimating of 1% chance

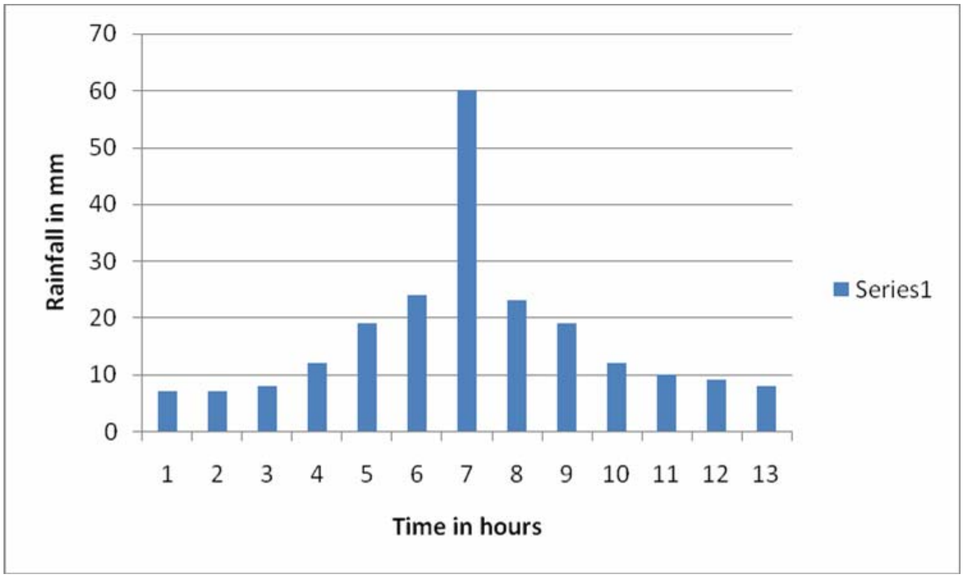

Fig 3.16:   1% chance hyetograph for the three subbasins

3.4 Estimation of breach parameters

Estimation of the initial parameter is a common task for most of HEC family models. In case of HEC- RAS breach parameters, are initial parameters need to be determine such as breach width , slope angle , time of failure , breach height depend to location of breach and the scenario type .

Several equations and formulas are available for estimating breach parameter . Bureau of reclamation (1982) and Von Thun & Gillette (1999).

Table 3.7: Equations for estimating breach parameters (Dam Safety office,1998).

S/N0	Reference	Breach width (M)	Time of failure (H)
1	Bureau of reclamation (1982)	$B = 3\mathbf{h}_w$	$T = 0.011\ \mathbf{B}$
2	Von Thun & Gillette	$B = 2.5h_w + C_b$	$T = 0.015\mathbf{h}_w$ (easily erodible) $T = \frac{\beta}{(4h_w + \sigma_1)}$ (highly erodible)

Where

B = Average breach width (M)

T = Failure time (hr)

h_w =height of water above the breach invert at time of failure

C_b offset factor a function of reservoir function (for reservoir $< 1.23 * 10^6$ $C_b = 6.0$

Chapter Four

4.0 Development of Hydrological and Hydraulic models

4.1 Hydrological model HEC-HMS and initial parameters

4.1.1 Determination of initial parameter

4.1.1.1 Delineation of the study area

Using ArcGis software upstream of the study has been divided into three sub basins.

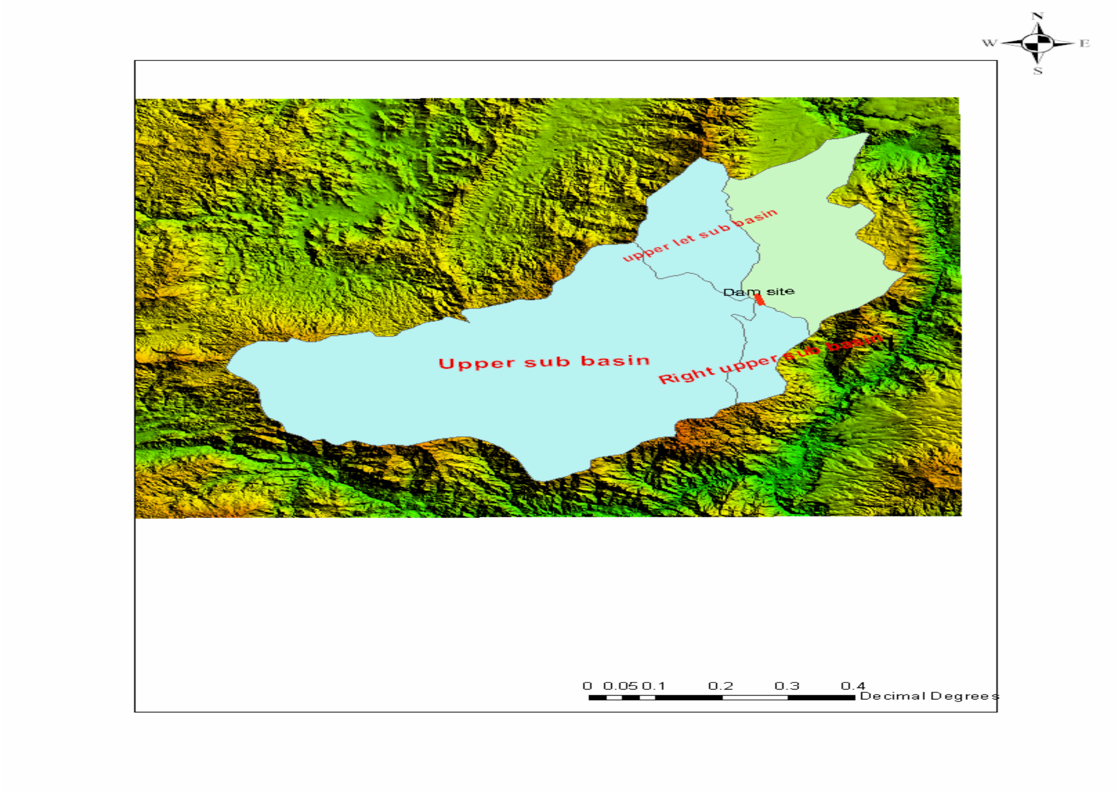

Fig 4.0: Delineation of subbasins

Table 4.0: subbasins

Subbasin Name	**Area in Km²**
Upper subbasin	3461.48
Upper left subbasin	232.37
Upper Right subbasin	52.82

4.1.1.2 Land use

Forest is common type for the three subbasins. Most of the forest found along the rivers and stream, other types of land is differ from subbasin to anther.

1- Upper subbasin: most of the area is dominate by agriculture, flowed by coffee under trees, silviculuture, silvo-pastoral, and urban.

1- Upper Right subbasin: range between two croups agriculture, and postural.

2- Upper Left subbasin: it consist of Agro-silviculture, Agriculture, Silvicu , and postural .

4.1.1.3 Reservoir Routing

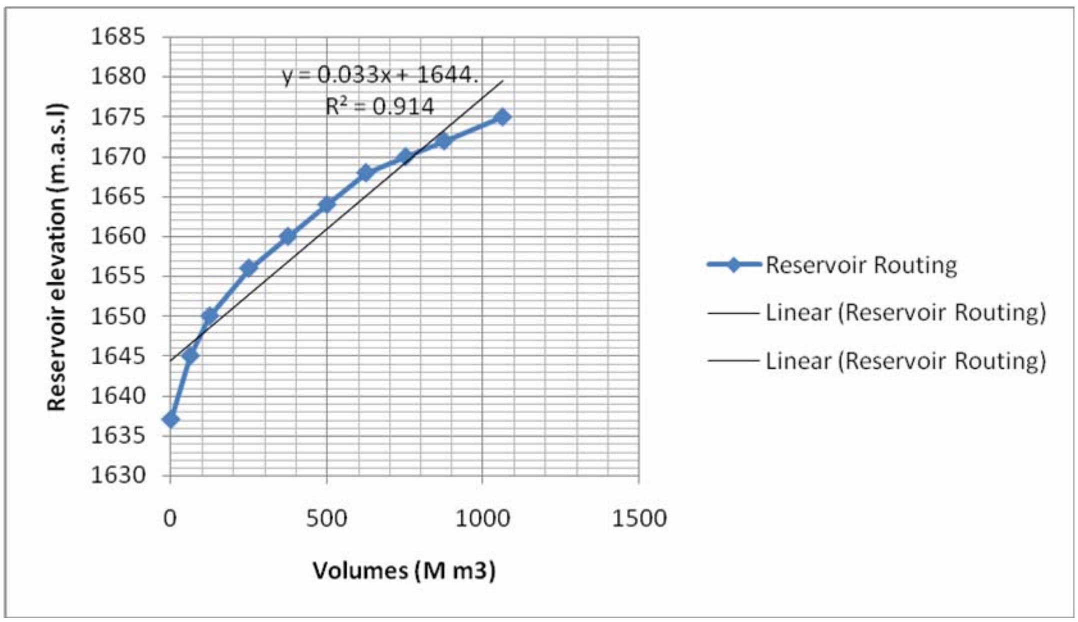

Figure 4.1: Reservoir Routing

Source: Ethiopia Electrical power corporation (EEPCO)

Table 4.1: Reservoir Routing

Elevation (m.a.s.l)	Storage (M m^3)
1635	0
1645	62.5
1650	125
1656	250
1660	375
1664	500
1668	625
1671	785
1675	1062

Elevation , storage table is developed from reservoir Routing curve Figure 4.1

4.1. 2 Development of Hydrological model HEC-HMS

HEC-HMS model developed for the study area. With assumption to some parameters, based on the characteristic of subbasins. Initial storage assumed in low water level elevation of reservoir 1635 m.a.s.l. Dam is modeled with no seepage, and method of storage is elevation storage. Number of spillway is one in main direction with four gates and the total length is 48 m, with crest elevation 1671 m.a.s.l. HEC-HMS Model developed to achieved some objectives , firstly is to estimated 1 % chance flood resulting from 1% chance precipitation. .Secondly is determing whether two scenarios can overtopped in case of probable maximum flood and 1 %chance .lastly hydrograph of the two scenarios probable maximum flood and one chance flood will be used as boundary condition for HEC-RAS model.

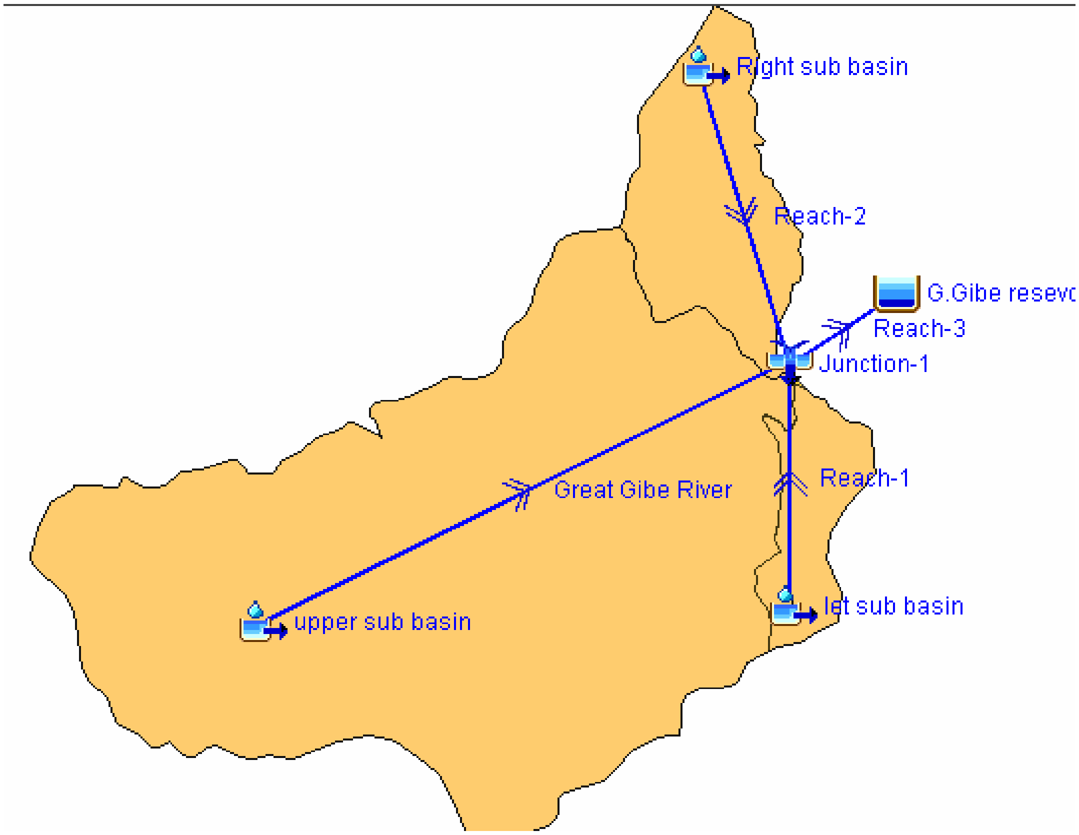

Figure 4.2: Represent the study area into HEC-HMS model

4.1.2.1 Probable maximum Flood (PMF)

Probable maximum flood (PMF) estimated for each subbasin using hyetograph of Maximum probable precipitation (PMP). (PMP) estimated from statistical method, three subbasins hydrograph used to determine the effect of PMF to the dam. Hydrological model HEC-HMS, estimated peak hydrograph for the three subbasins are 1225 m3/s , 380 m3/s , and 24000 m3/s for upper subbasin , upper left subbasin, and upper right subbasin respectively figures 4.3 , 4.4 & 4.5

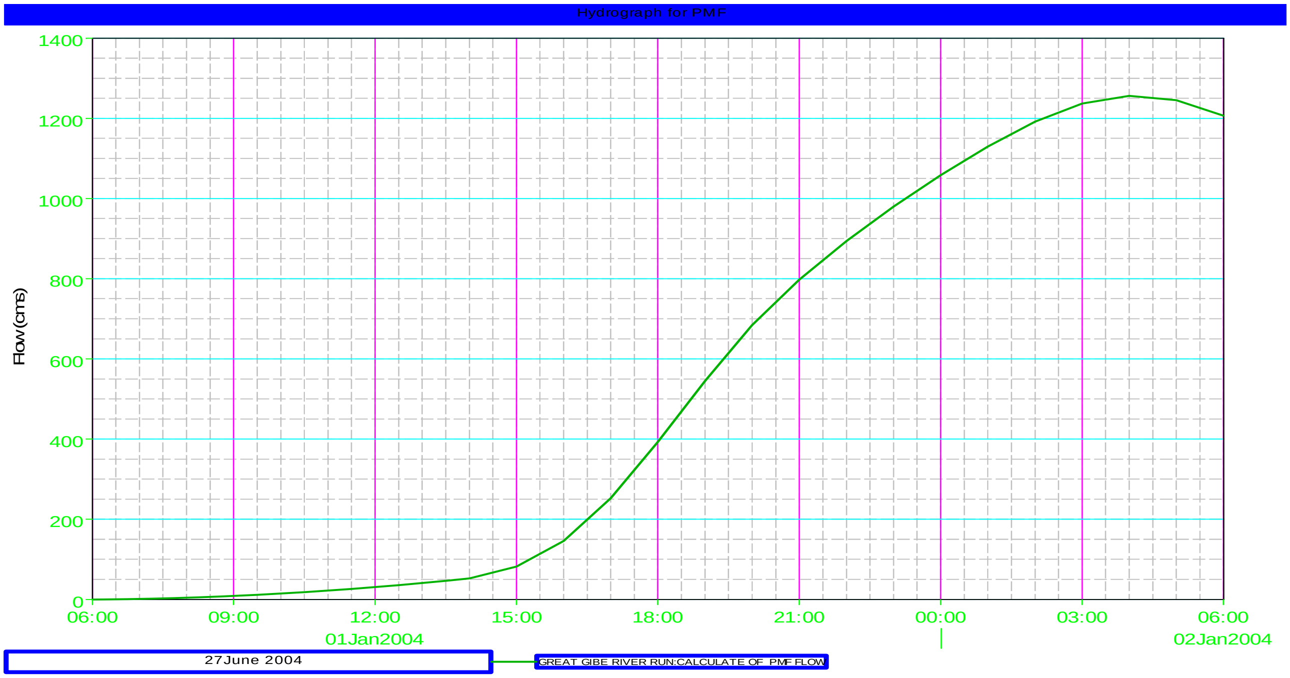

Figure 4.3: Upper subbasin hydrograph for PMF

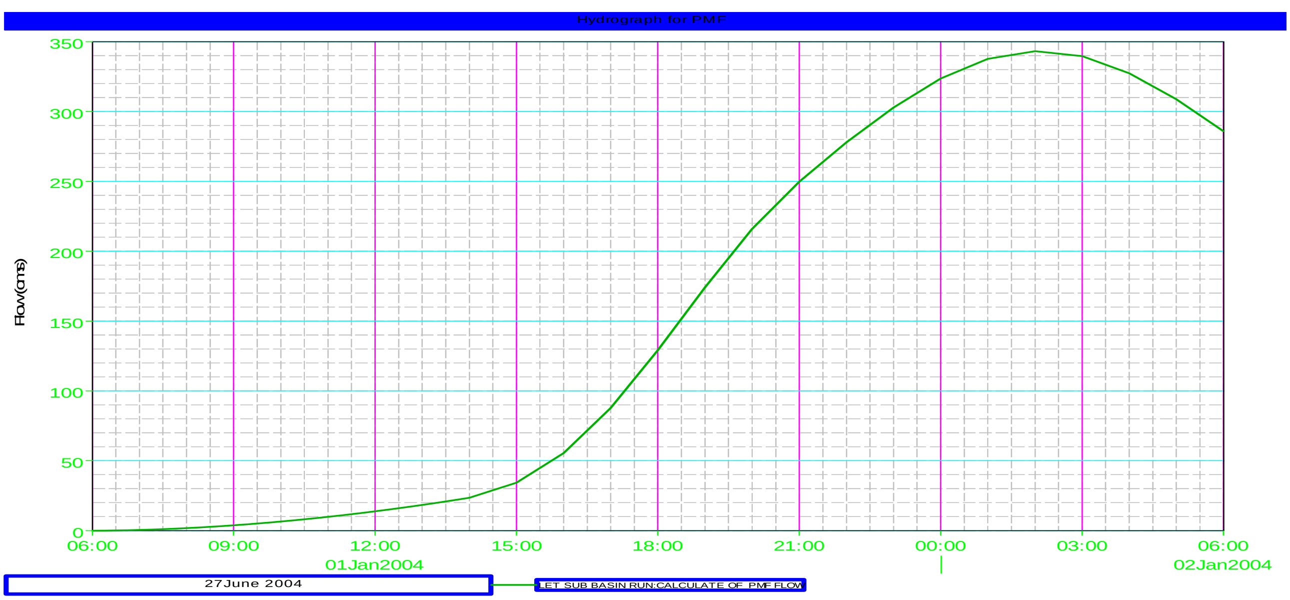

Figure 4.4: Upper left subbasin hydrograph for PMF

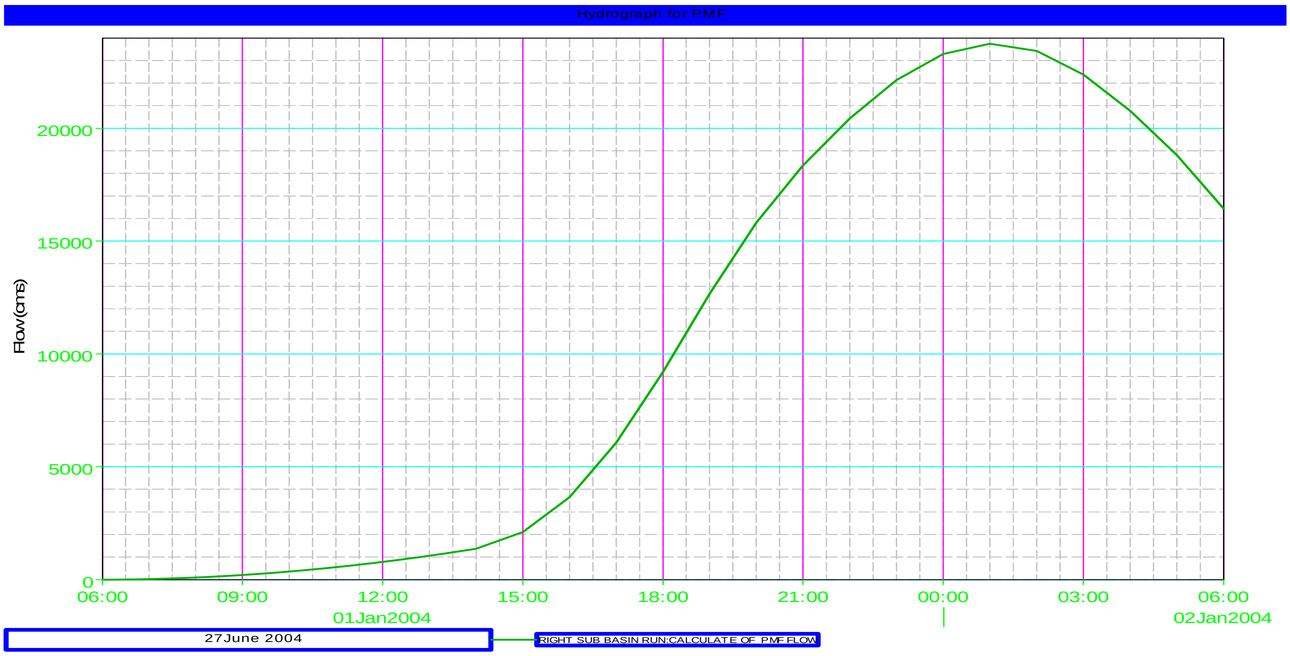

Figure 4.5: Upper Right subbasin hydrograph for PMF

4.1.2.2 1% chance flood

Hydrological model HEC-HMS, estimated 1 % chance flood .Input parameter to the model for 1% chance flood event was 1 % chance precipitation hyetograph. 1% chance precipitation estimated from one station and distributed uniformly over the catchment area(three subbasins).

Peak hydrograph for the three subbasins are 385 m^3/s, 175 m^3/s, and 7600 m^3/s for upper subbasin , upper left subbasin , and upper right subbasin respectively .

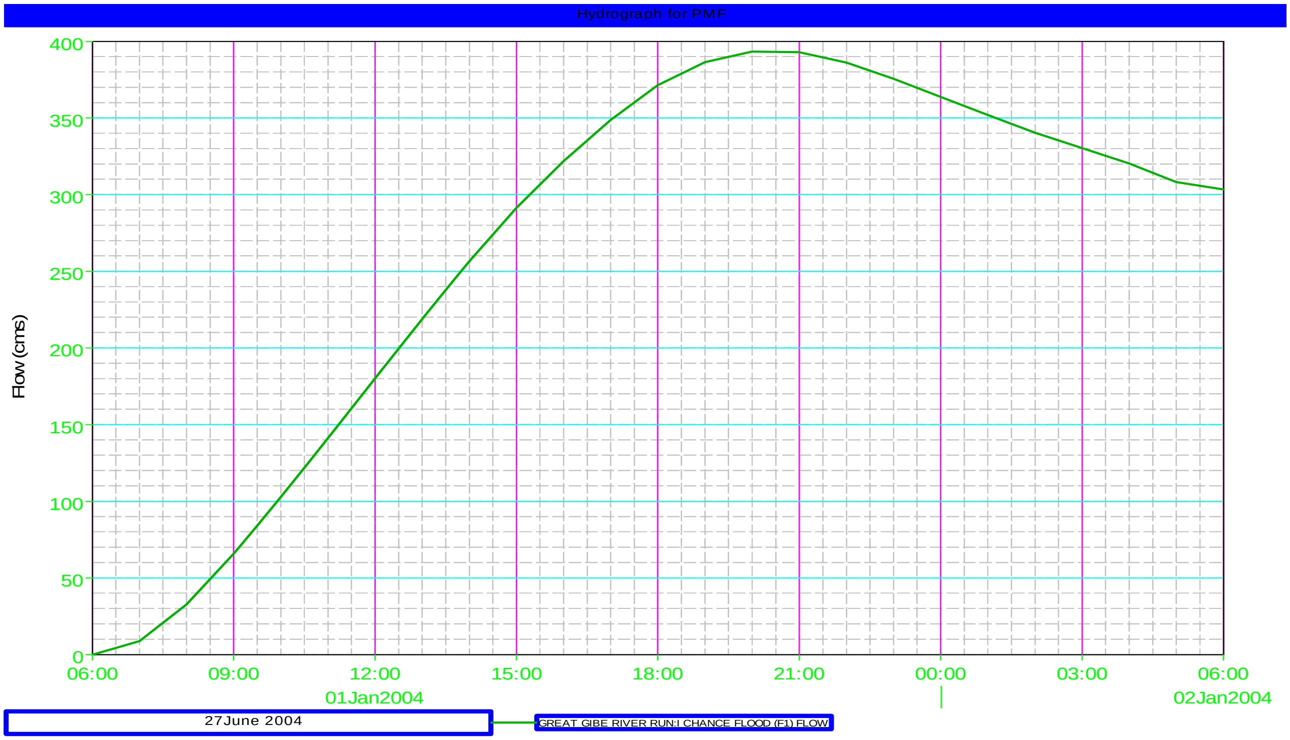

Figure 4.6: Upper subbasin hydrograph for 1% chance flood

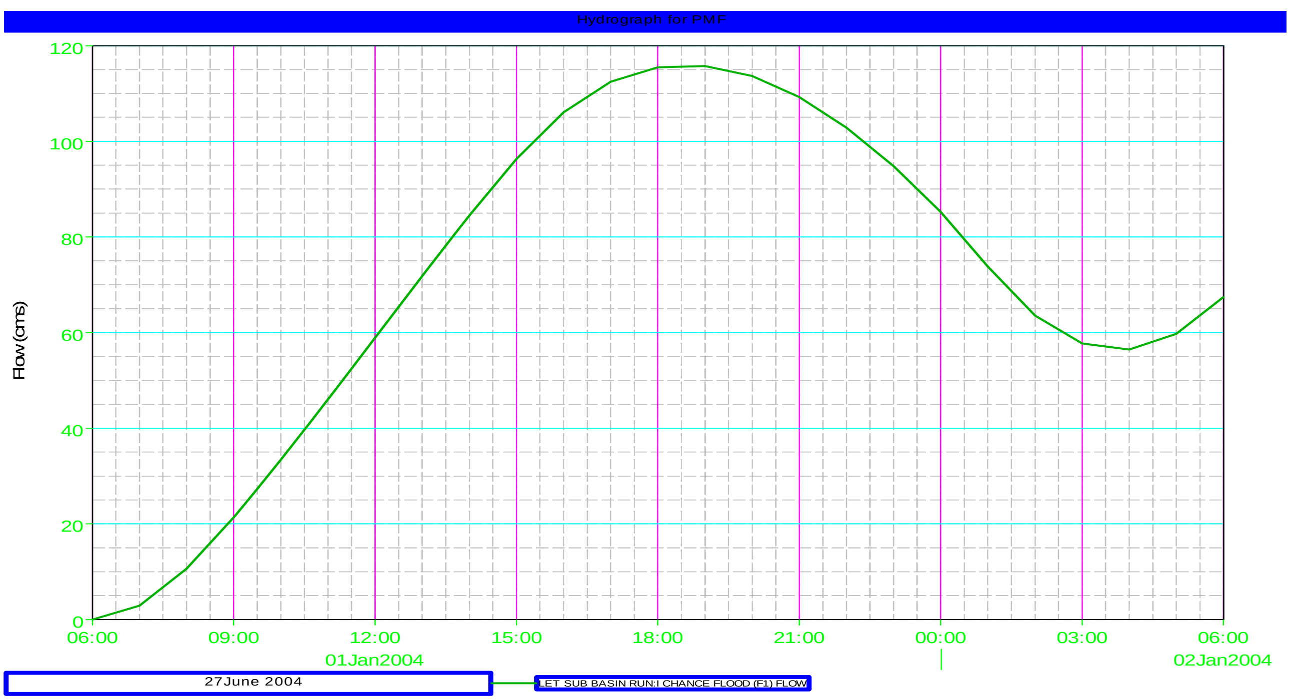

Figure 4.7: Upper left subbasin hydrograph for 1% chance flood

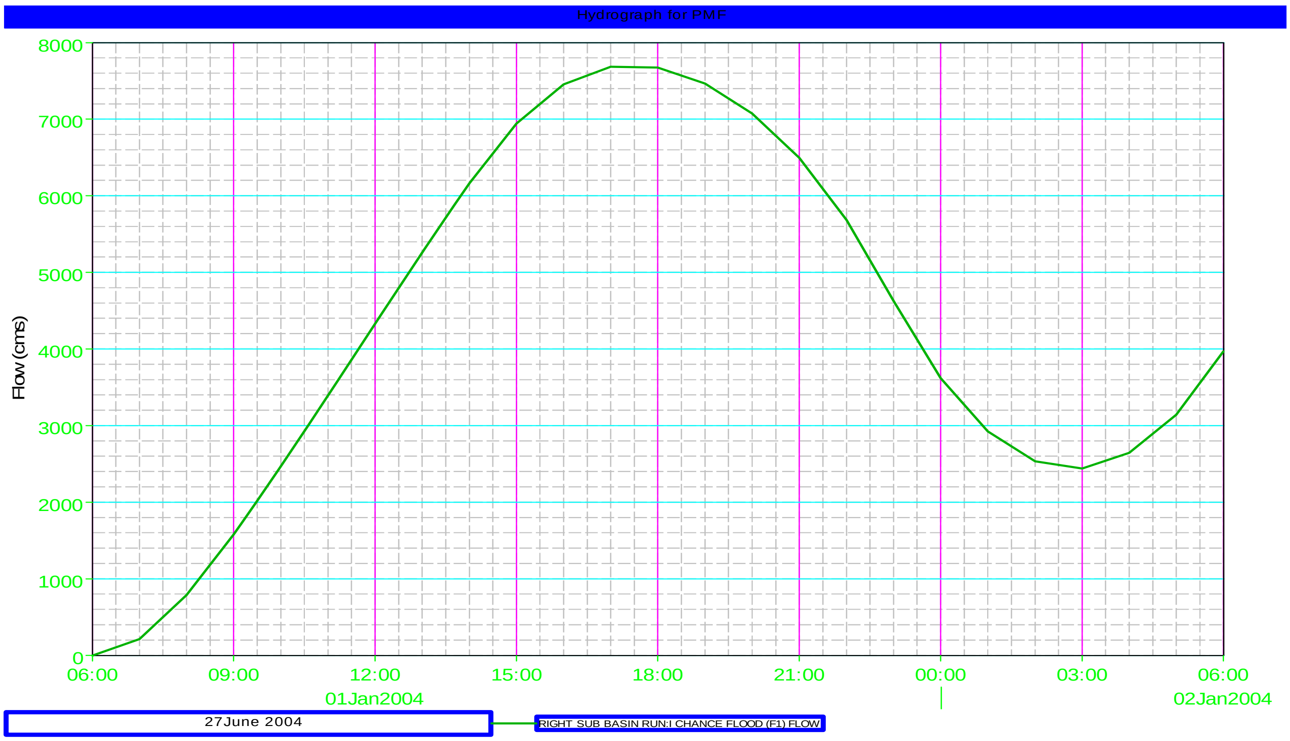

Figure 4.8: Upper Right subbasin hydrograph for 1% chance flood

4.2 Development of Hydraulic models HEC-RAS

Answering the question whether HEC-RAS is the only model available for dam break analysis. In fact they are numerous models and tools for analysis dam break and resulting outflow hydrograph. Some of known and widely used models in the field are National Weather service (NWS), Dam break flood forecasting model (DAMBRK), HEC-1 package, and NWS simplified dam break flood forecasting model (SMPDBRK). Possibility of getting the model (free), the nature of data availability and model ability were the strong reason for selecting HEC-RAS .HEC-RAS is computer model developed to achieved some objectives, such as determine the water surface profile , peak flow and outflow hydrograph for the three scenarios.

4.2.1 Water surface profile

Geometric data entered to the model by represent the subbasin by reach . Cross sections allocated to the river (schematic). Three cross sections entered from upstream to downstream, the different in cross sections were adjusted through the model by using adjust feature and multiple by factor . Junction data entered to show the distance of river to junction from upstream toward junction.
Within flow data part, three profiles were estimated 1 year, 50 year, 1 % chance for having a good result and comparison. Boundary condition was in downstream based on assumption that the flow is subcritical, normal depth (S) was used as downstream boundary condition.
When all the geometric data, flow data, and boundary condition entered performing the hydraulic calculation.

4.2.2 HEC-RAS for Dam break analysis

Study dam modeled throught HEC-RAS computer program . Dam and spillway are entered throught lateral structure. Which has ability to model lateral weir , gated spillway , culvert , and diversion rating curve .A lateral structure added to the model after selecting the river and reach . Dam and spillway represent in model by distance and elevation method .Reservoir upstream of the dam entered as sroage area, connected to river by using lateral structure , reservoir routing enered to stroage by elevation method .

Breach parameters entered in same way of dam , by distance and elevation method . The shape of breach is assume trapezodial growing with time , all breachs were creat for piping failure. Piping coefficent which equal to 0.8 has chosen by the model , but the other data such as center of breach , final elevation , initial piping elevation , and side slope are entered by resercher .

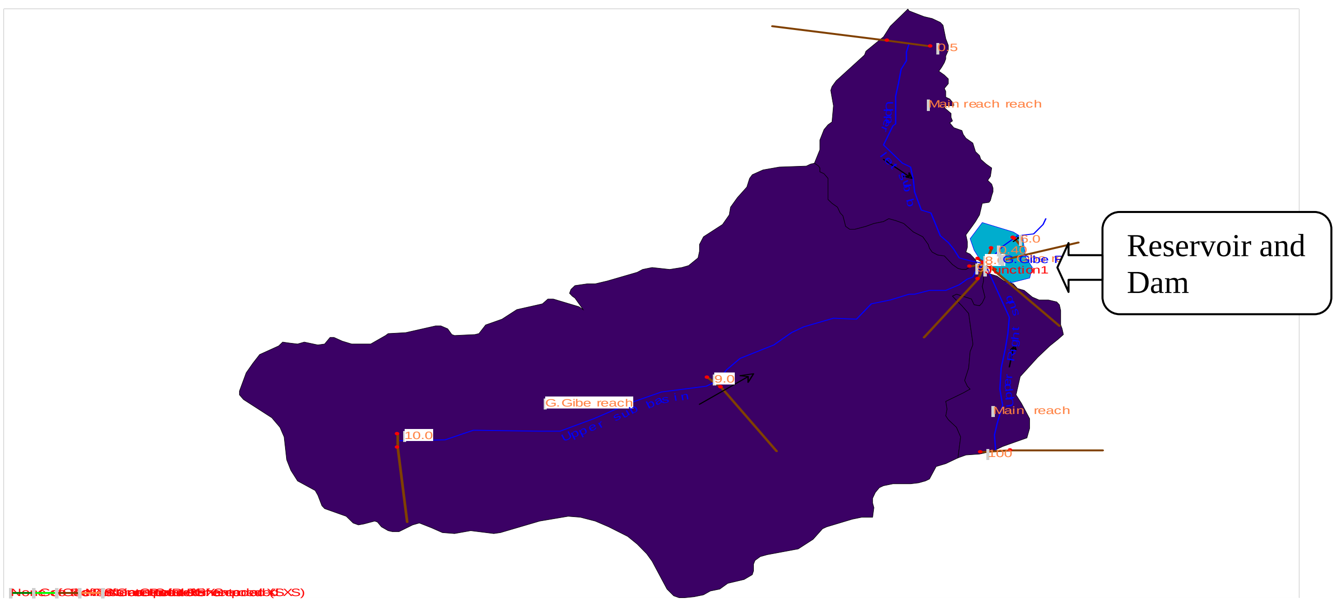

Figure 4.9: Study area reprsent in HEC-RAS model

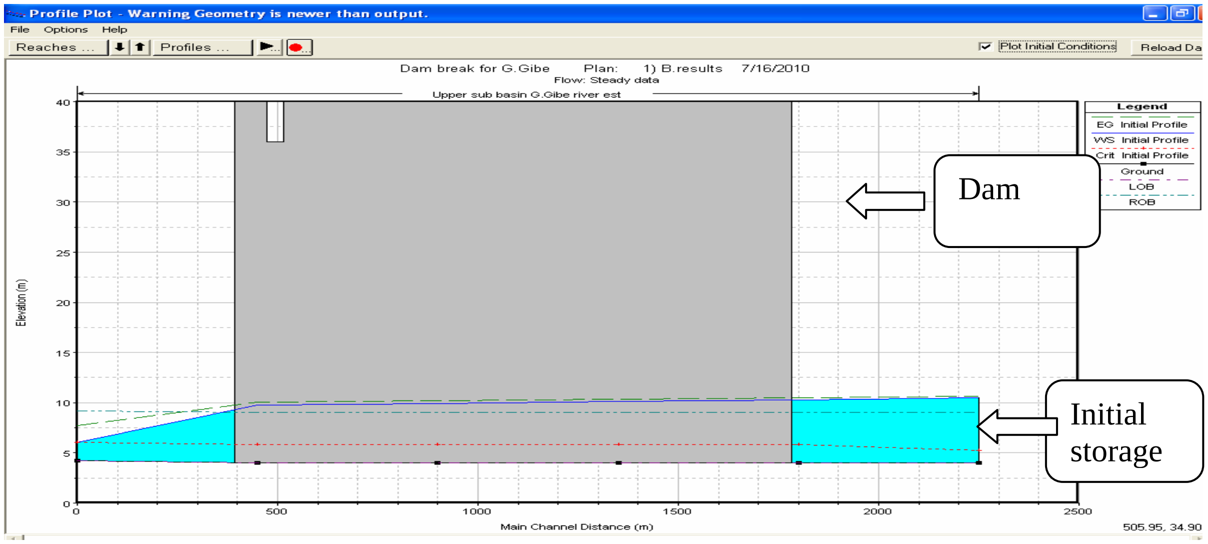

Figure 4.10: Represent of dam and spillway in the model

4.2.2.1 HEC-RAS for probable maximum Flood (PMF) event

Breach parameters are estimated using Bureau of reclamation (1982) and Von Thun & Gillette. The water height assumed to be 36m.

Table 4.2: Estimated breach parameters for PMF event

Method	**Breach width (M)**	**Time of failure (hr)**	**Angle of slope**
Bureau of reclamation (1982)	**108**	**1.2**	**0.6**
Von Thun & Gillette	**96**	**0.54 (easily erodible)**	**0.6**

4.2.2.2 HEC-RAS for 1% chance Flood (100 Year) event

Breach parameters are estimated in same way of 1% chance day using Bureau of reclamation (1982) and Von Thun & Gillette. The water height assumed to be 36m.

Table 4.3: Estimated breach parameters for 1% chance event

Method	Breach width (M)	Time of failure (hr)	Angle of slope
Bureau of reclamation (1982)	**60**	**0.40**	**0.6**
Von thun & Gillette (1999)	**56.0**	**0.30 (easily erodible)**	**0.6**

4.2.2.3 Sunny day failure

Failure with sunny day, it will be greats under normal operation condition (Reservoir pool), without any additional inflow to the reservoir .Failure mechanism is piping. Applying Bureau of reclamation (1982) and Von Thun & Gillette equations. The water height assumed to be 30 m.

Table 4.4: Estimated breach parameters for sunny day

Method	Breach width (M)	Time of failure (hr)	Angle of slope
Bureau of reclamation (1982)	**90**	**0.594**	**0.6**
Von thun & Gillette (1999)	**81.1**	**0.45 (easily erodible)**	**0.6**

Chapter Five

5.0 Results and Discussion.

The objectives of the research was achieved by using two computer models ,hydrological model HEC-HMS and hydraulic model HEC-RA , which will be discussed in this chapter .

5.1 HEC-HMS Result for PMF and 1% chance flood

The results of the HEC-HMS model showed that there is no overtopping over the dam for the two scenarios 1 % chance flood and PMF . Only overtopping over spillway in PMF event . Overtopping will happen with elevation 1672.9 m.a.s.l that means the height of water over the spillway crest is 1.9 m , as shown in Figure 5.0 .

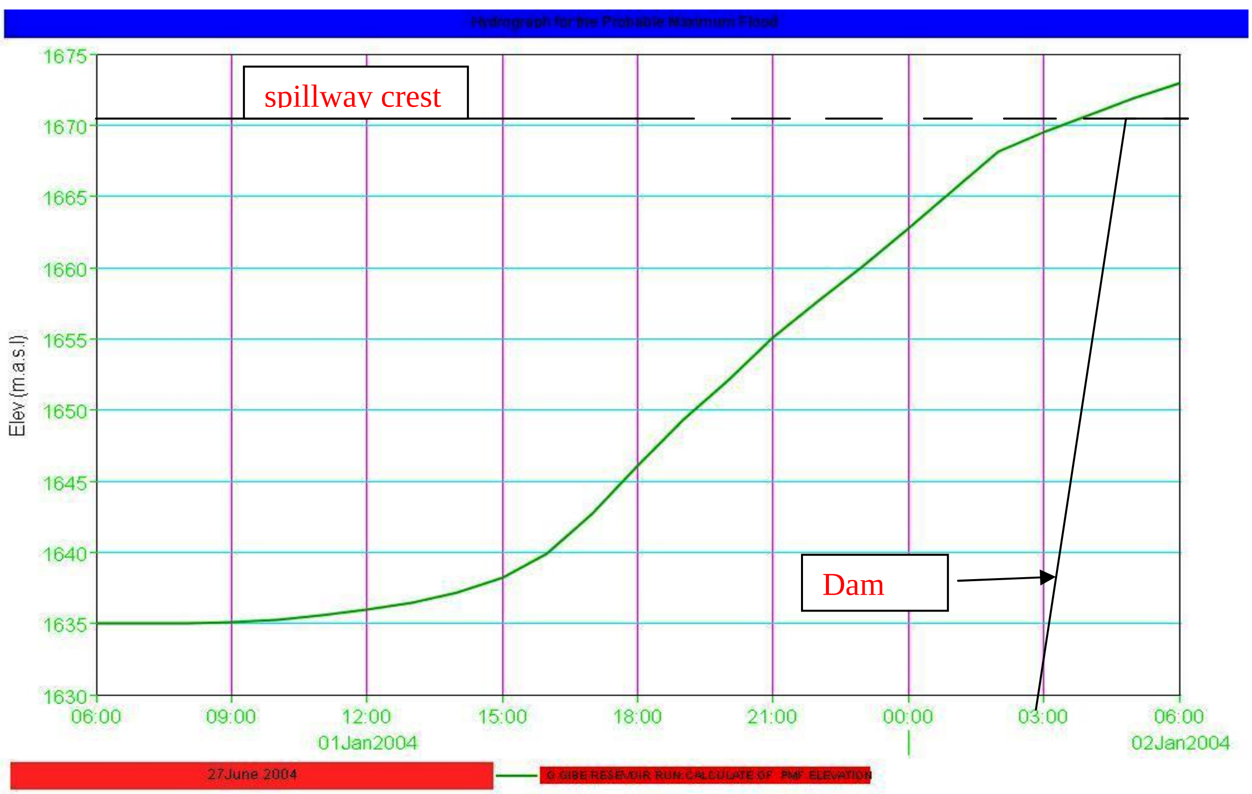

Fig 5.0: HEC-HMS Result for probable maximum flood (PMF)

While for 1 % chance flood. Maximum water level is 1660 m.a.s.l , less than spillway crest elevation 1671 m.a.s.l ,as shown in Figure 5.1 .

Study dam design for 100 years return period. The maximum flood estimated is 14250 m^3/s for this return period .Based on the return period, study dam constructed with 40 M height. Applying some principle of hydraulic to case study dam. It will indicated that the maximum water level for the dam was 36 M,(1671 m.a.s.l), and free board is 4 m. comparison between design flood and estimated flood by using model .The design flood is greater than model Flood. Because for 1 chance (flood estimated by model) the maximum elevation is 1660 m.a.s.l as shown in figure 5.1.

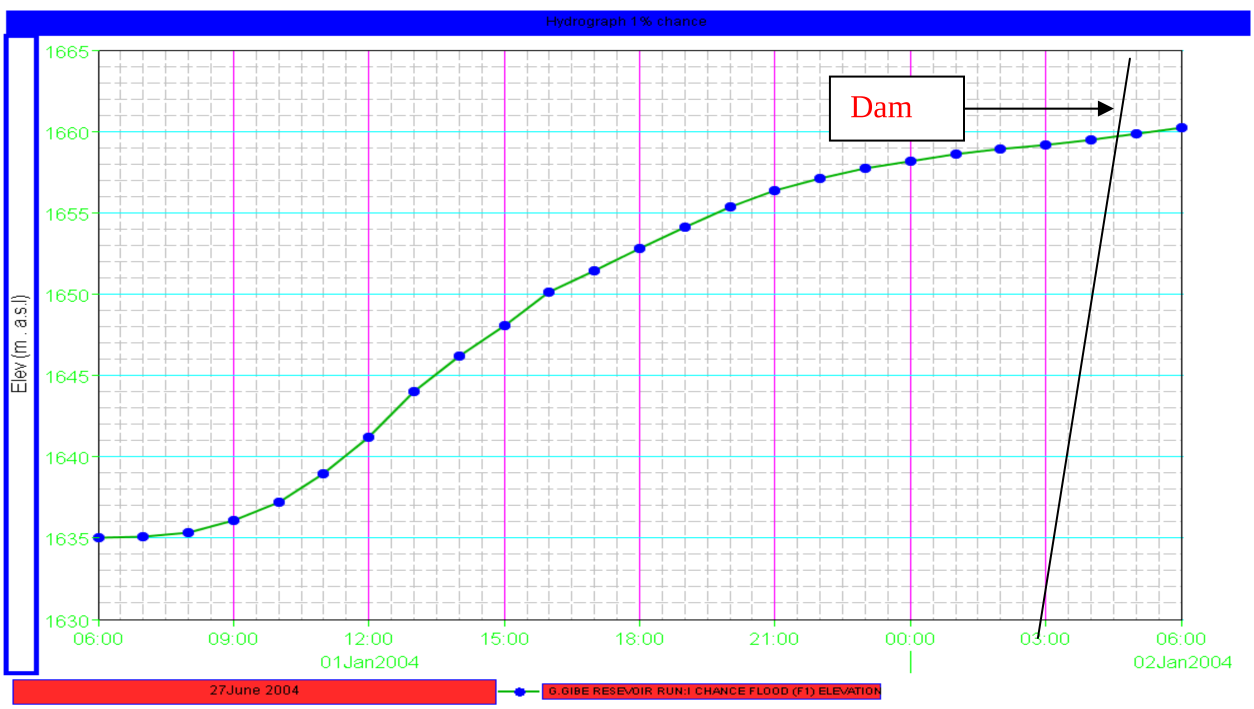

Fig 5.1: HEC-HMS Result for 1% chance

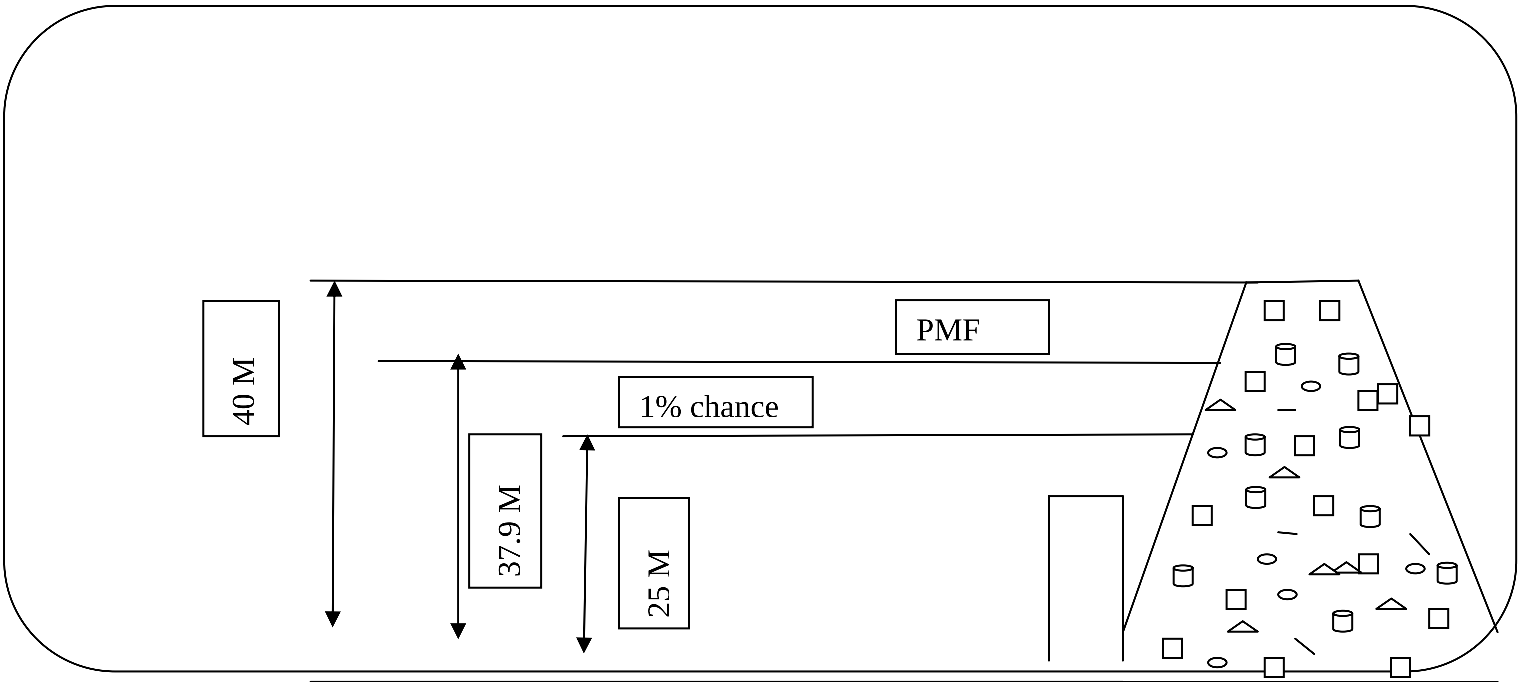

Figure 5.2: Result of the two scenarios to the Dam

Table 5.0: Result of the two scenarios to Dam

S/No	Scenario	Effect to the dam
1	1% chance (100 year)	No overtopping
2	PMF	No overtopping

Input parameters to the model for probable maximum flood were probable maximum precipitation. The model estimated the peak hydrograph for the three sub basins 1225 m^3/s, 380 m^3/s, and 24000 m^3/s, for upper subbasin, upper left sub basin, and upper Right subbasin respectively. While there was no overtopping over the dam crest, only overtopping will occur over the spillway (1.9 1bove the crest).

Most of the studies when probable maximum flood applied to dam or any hydraulic structure design with return period flood there will overtopping over the structure. In the case study there are two reasons my effect or reduced the value of probable Maximum flood (PMF).

- Statistical method (Hershfield approach) generally tends to give lower value than meteorology approach.(Abdullah,2004).
- Time series data used for Statistical method (Hershfield approach) 5 years only is very short, so if the time series is long there is probability of getting higher PMF.

Input parameters to the model for one chance flood, was one chance precipitation hyetograph. The hyetograph is developed from one station only. This result 1% chance cannot consider as accurate and properly. Because one chance precipitation was Estimated from one station with long time series data , then distributed uniformly over the three subbasins .In case of developing 1 chance for each sub basin the result will be more accurate.

5.2 HEC-RAS for water surface profile

Water surface profile developed for the main river, which called great Gibe for three return periods one year, 50 years, and 100 years, (to show the possibility of flood from the river). Legend in figure 5.3 gives indicate for each period, with truth that water level in one year is constant for all other return periods. That means the water level in the river should not be less than this level**.**

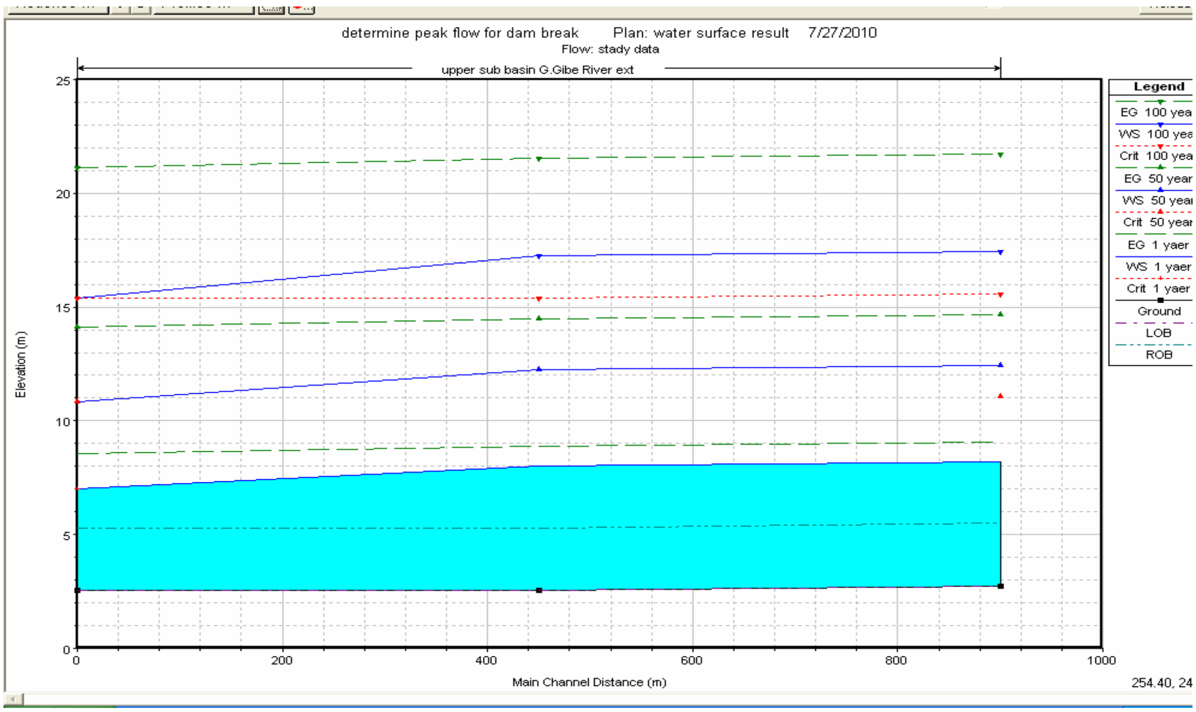

Figure 5.3: HEC-RAS for Water surface profile for the main river

5.3 HEC-RAS PMF, 1 % chance and sunny day

The simulated outflow hydrograph for probable maximum flood . Showing reservoir depletion discharge or outflow hydrograph of the proposed dam in the event of dam break (dam failure) . The hydrograph indicates a peak discharge value of 25184 m^3/s, and a total duration of significant outflow of about 1.2 hours. Model estimated this result based on breach parameters estimated by using Bureau of reclamation method (1982). The outcome of the simulated maximum outflow hydrograph is 97454.4 (1000M^3). Meanwhile in case of using Von Thun & Gillette (1999) method a peak discharge value of 24959.73 m^3/s and outflow hydrograph 97255.16 (1000 M^3), with time duration 0.54 hour. This analysis, the dam break was set for one of the worse-case scenarios (first scenario), whereby complete dam breach developed for probable maximum flood.

For the second scenario, 1% chance flood. Model simulated a peak discharge value of 8151.91 m^3/s, and outflow hydrograph of 40622.88 (1000 M^3), with a total duration of o.40 hour .The result achieved by using Bureau of reclamation method (1982). Thun & Gillette (1999) method indicates a peak discharge of 3833.2 m^3/s , and out flow hydrograph of 23385 (1000 M^3).The total duration time was 0.30 hour.

Sunny day failure (the third scenario) occurred, when there is no additional water added to the reservoir. Outcomes of the model simulated maximum discharge using Bureau of reclamation method (1982) of 571.4 m3/s , and out flow hydrograph of 1868.48 . With time estimated by 0.594 hour. The outcomes of the simulated maximum discharge using Thun & Gillette (1999) was 494.52 m^3/s, and out flow hydrograph was 1265.53(1000 M^3).Thun estimated 0.45 hour as a time of duration.

Analysis the results of hydraulics model HEC-RAS. Indicate that the peak flow , and out flow hydrograph, are depending to breach parameters, nature of the reservoir ,additional inflow , and formation time .

5.3.1 Additional inflow

Form the three scenarios applied, two scenarios having additional inflow .Probable maxim flood , and 1 % chance precipitation. While one scenario without additional inflow (sunny day). Peak flow ,and outflow from the breach , depends whether there is additional inflow to the reservoir or not. A clear example is 1% chance event and sunny day. The breach width for 1% chance is 60 M. While breach width for sunny day was 90 M. That means breach width for 1% chance is less than sunny day . But the peak flow for 1% chance by using

Bureau of reclamation method was 8151.91 M^3/S as shown in figure 5.4 . And for sunny was 571 M^3/S as shown in figure 5.5 .The reason is for 1% chance there is additional inflow adding to the reservoir , Meanwhile for sunny day no additional inflow added.

The same result can be achieved by using Von Thun & Gillette (1999) method. Peak flow hydrograph for 1 % chance is 3933.2 M^3/s as shown in figure 5.6 . And for sunny day is 494 M^3/s as shown in figure 5.7.

Additional inflow has great contribution to outflow hydrograph. With the same data of 1 % chance , and sunny day . Using Von Thun & Gillette (1999) method for 1% chance event outflow hydrograph is 23385 ($1000M^3$) as shown in figure 5.6. And for sunny day is 1265.53 ($1000M^3$) as shown in figure 5.7 .In case of Bureau of reclamation method Outflow hydrograph for 1% chance is 40622.88 ($1000M^3$) as shown in figure 5.4 , and for sunny day is 1868.45 ($1000M^3$) as shown in figure 5.5 .

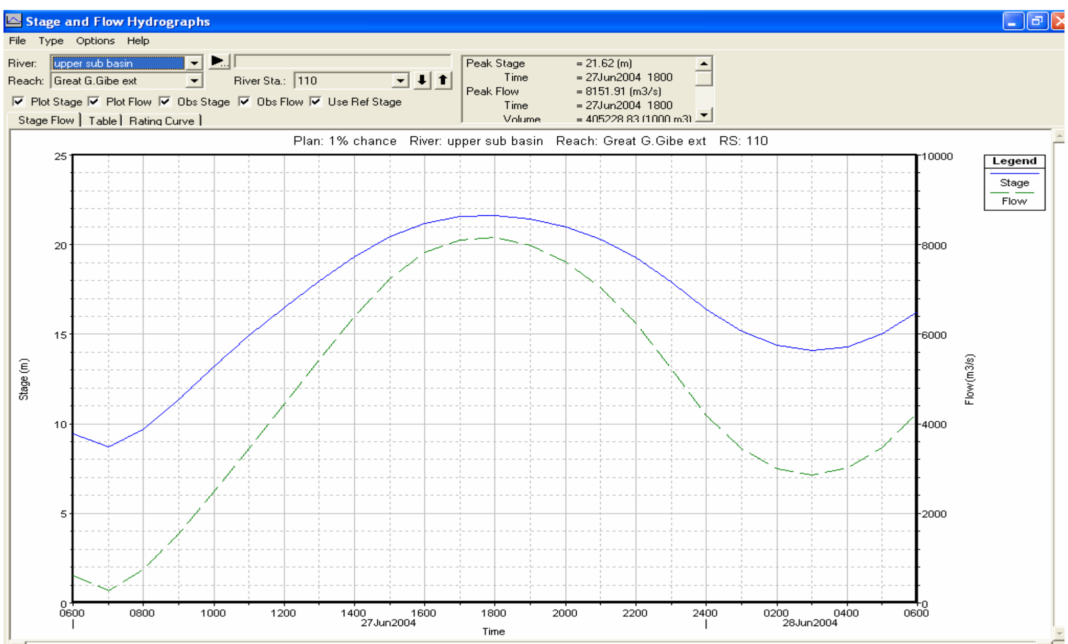

Figure 5.4: Breach hydrograph using Bureau reclamation (1982) method for 1% chance

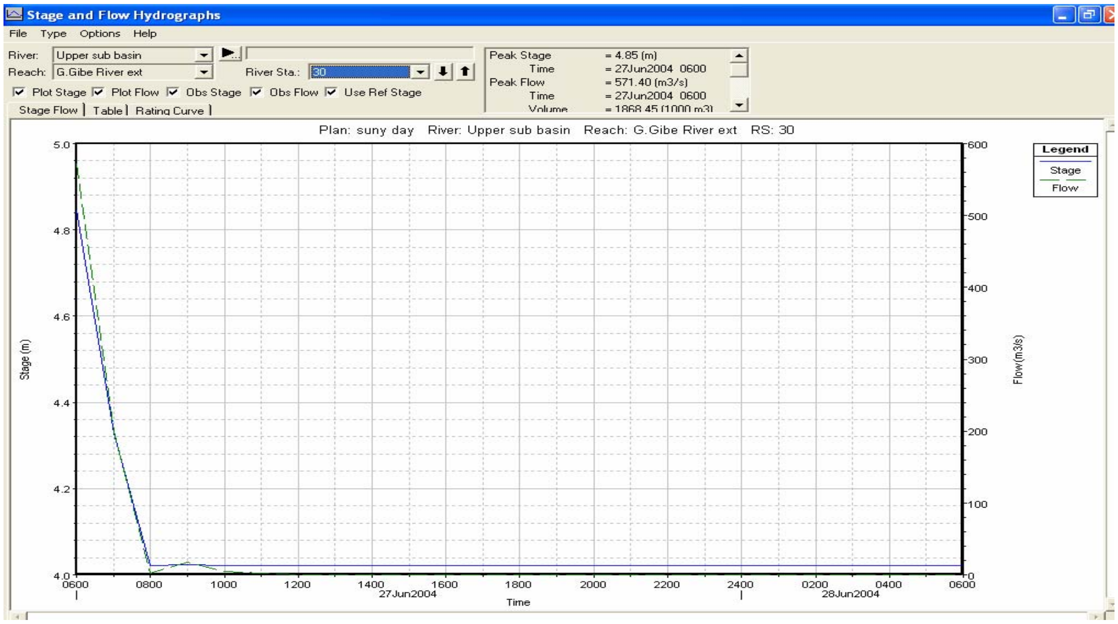

Figure 5.5: Breach hydrograph using Bureau of reclamation(1982) method for sunny day

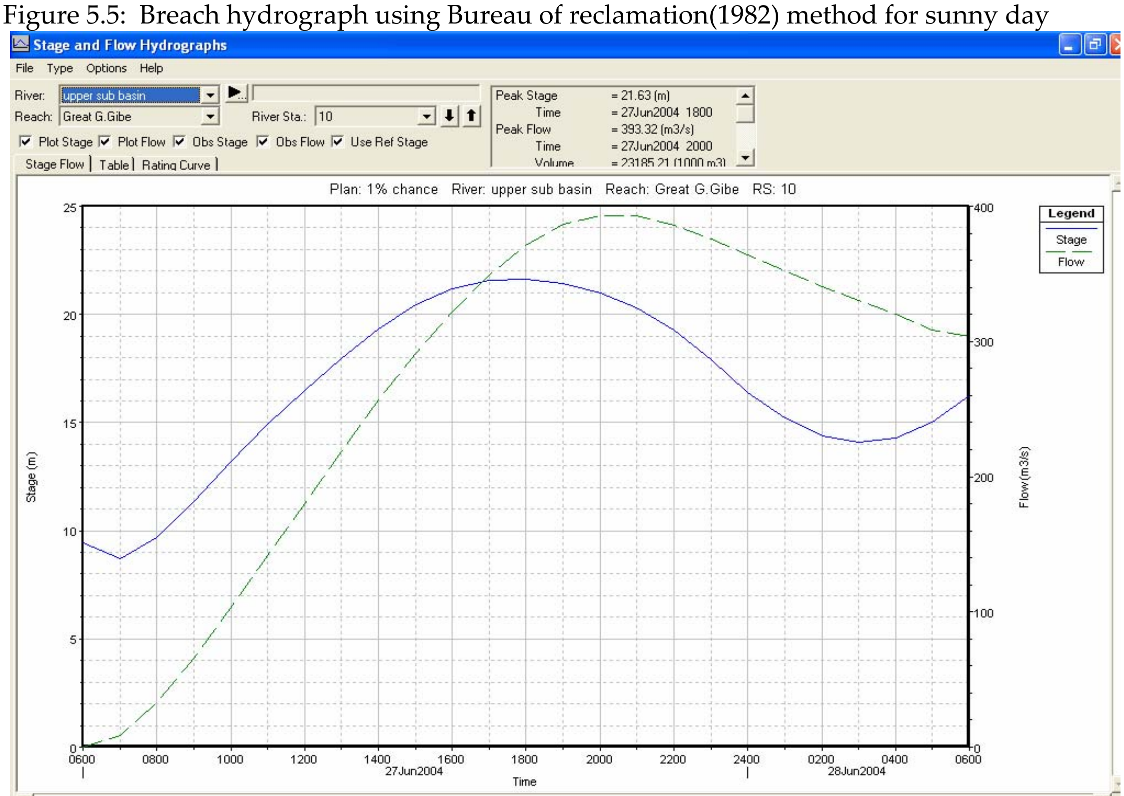

Figure 5.6: Breach hydrograph using Von Thun & Gillette (1999) method for 1 % chance

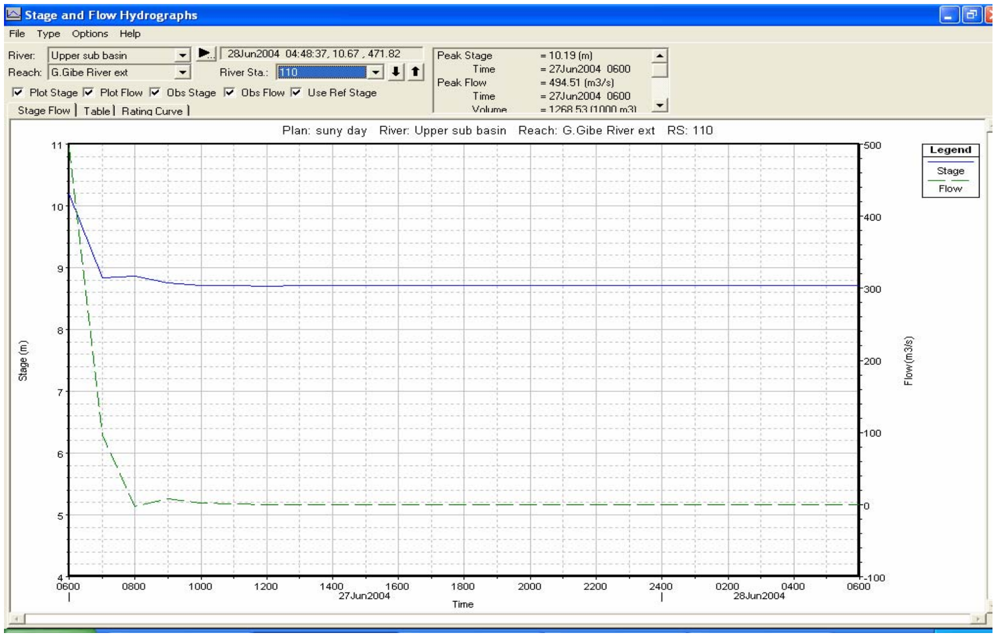

Figure 5.7: Breach hydrograph using Von Thun & Gillette (1999) method for sunny day

5.3.2 Breach parameters and additional inflow

The results of the model showed the effect of Breach parameters to the peak flow. Comparison can be made for PMF event and 1% chance . Two are having additional inflow to the reservoir, but they vary in breach width , time of failure, and additional inflow for each event .For PMF the breach width 108 M and time of failure 1.2 hour, while for 1% chance was 60 M , 0.4 hour for width and time respectively . Peak flow for PMF was 25184 M^3/S as shown in figure 5.8. And for 1% chance was 8151.9 as shown in figure 5.9 using Bureau of reclamation method. Von Thun & Gillette method so far in same way for PMF 24959 M^3/s as shown in figure 5.10 . And 393.32 M^3/s for sunny day as shown in figure 5.10.

Most of the time there is a relation between peak flow and outflow hydrograph. which indicated by the results of outflow hydrograph for PMF and 1 % chance .

Using Bureau of reclamation method, the outflow hydrograph for PMF is 97454.4 (1000M3) as shown in figure 5.8, and for 1% chance 40622.88 (1000M3) as shown in figure 5.9 . In case of Von Thun & Gillette method .Out flow hydrograph for PMF is 24959.73 (1000M3) as shown in figure 5.10, meanwhile for 1% chance out flow hydrograph is 23385 (1000M3), as shown in figure 5.11.

The result of the model shows that the outflow hydrograph depending on the reservoir status , concerning availability of addition inflow to the reservoir. That might leads to have higher outflow hydrograph for PMF more than 1 %chance ,greater than sunny day .

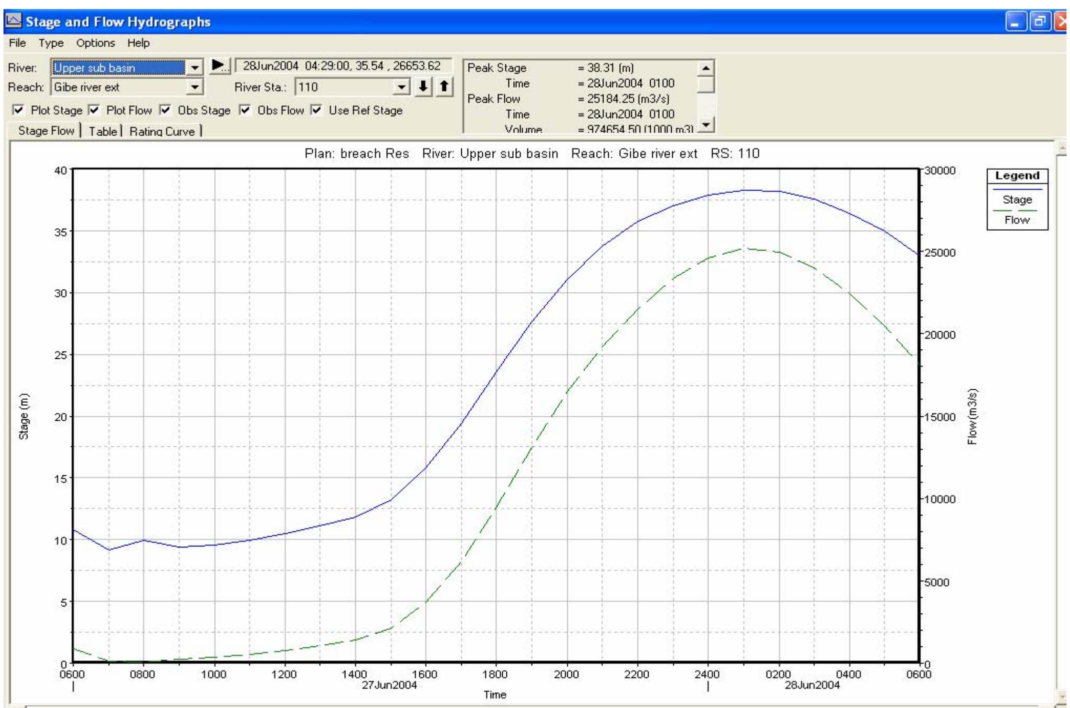

Figure 5.8: Breach hydrograph using Bureau of reclamation(1982) method for PMF

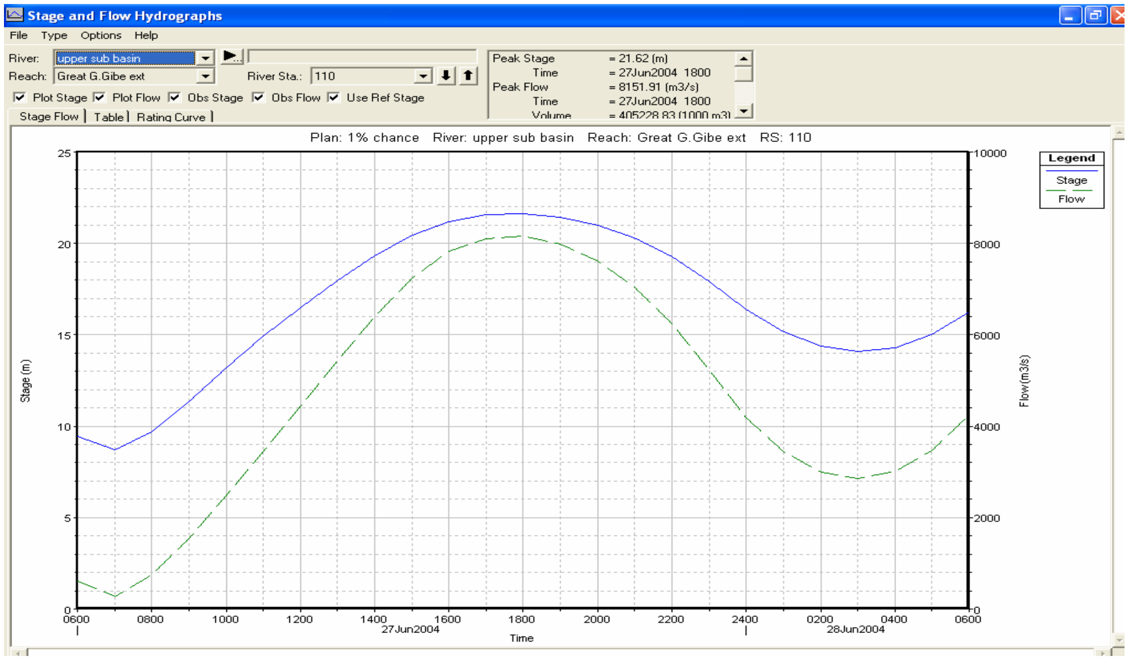

Figure 5.9: Breach hydrograph using Bureau of reclamation (1982) method for 1 % chance

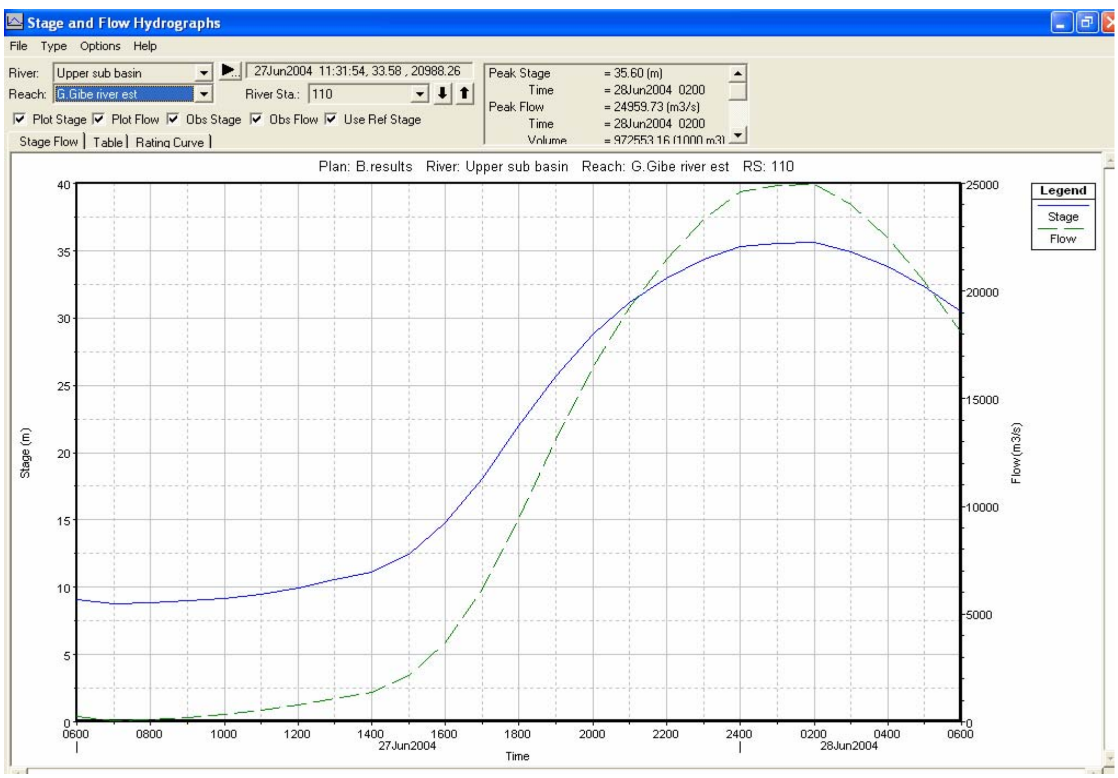

Figure 5.10: Breach hydrograph using Von Thun & Gillette (1999) method for PMF

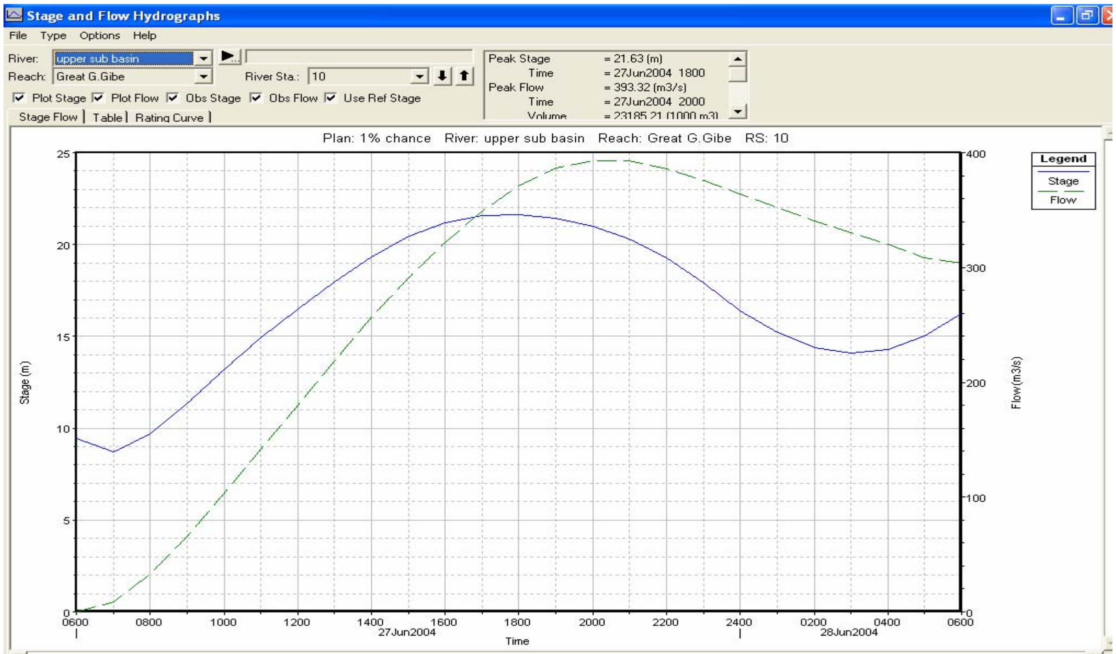

Figure 5.11: Breach hydrograph using Von Thun & Gillette (1999) method for 1 % chance

Table 5.1: Summary of the three scenarios for Bureau and Thun equations .

Method	PMF		1 % chance		Sunny day	
	Peak flow (M^3/s)	Water volume (1000 M^3)	Peak flow (M^3/s)	Water volume (1000 M^3)	Peak flow (M^3/s)	Water volume (1000 M^3)
Bureau of reclamation(1982)	25184	97454.4	8151.91	40622.88	571.4	1868.45
Von Thun & Gillette(1999)	24959.73	97255.16	3833.2	23385	494.51	1265.53

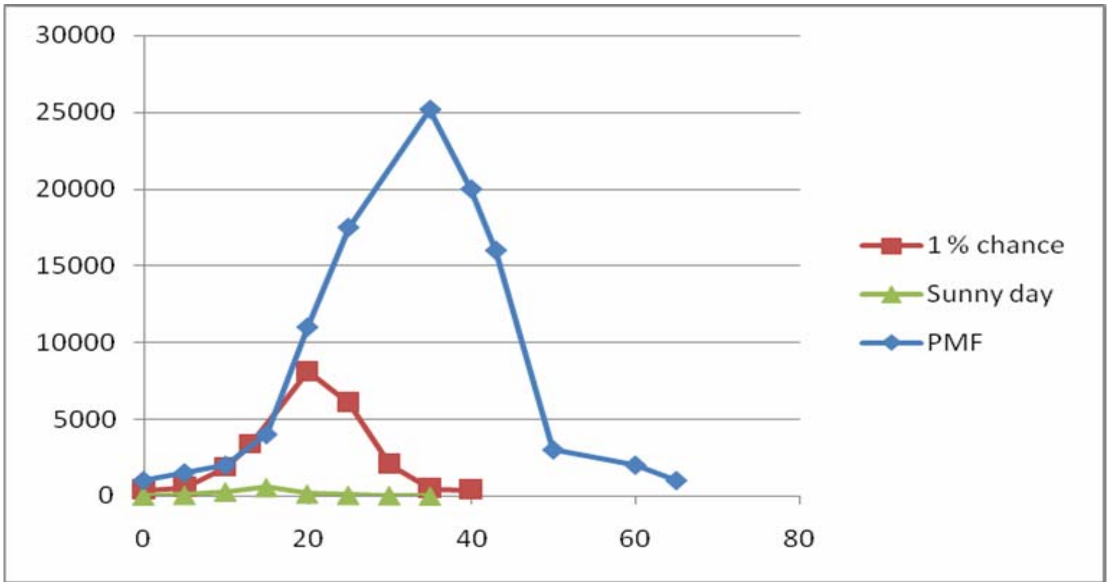

Figure 5.12: Peak flows for three scenarios using Bureau of reclamation(1982) method

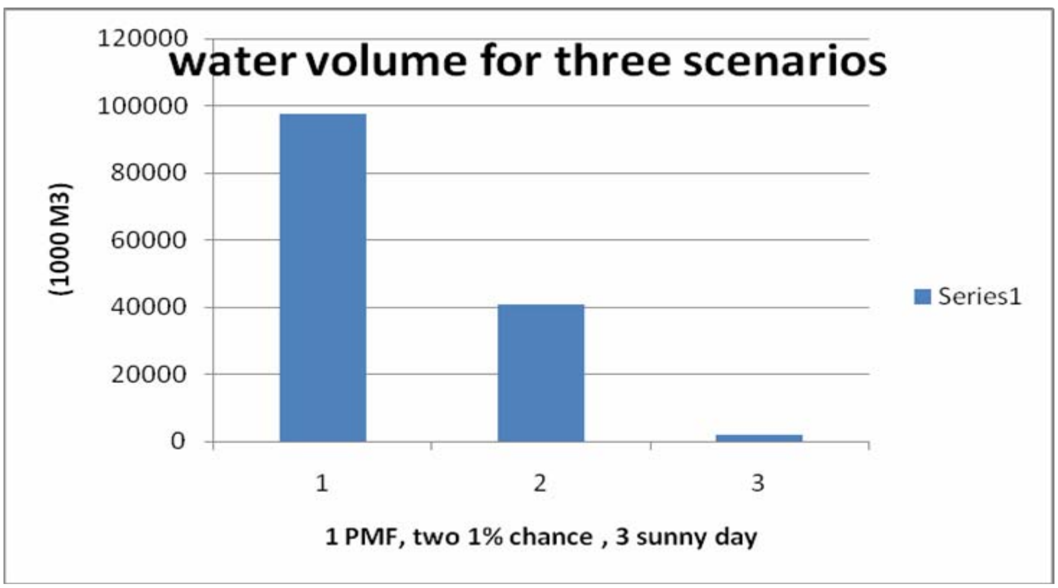

Figure 5.13: Outflow hydrograph using Bureau of reclamation(1982) method

Chapter six

6.0 Summary, Conclusion and recommendation

6.1 Summary

Omo –Gibe River Basin which is considered as one of the most important basin in Ethiopia in water resources potential, this time three hydropower schemes exist, Gibe I, II, and III. The basin is located in South-west part of Ethiopia. It lies between 4 30 and 9 30 North latitude, and 35 00 and 38 00 East Longitudes**.** A case study dam is one of three hydropower dam –Gibe I- rock fill dam with asphalt concrete face, constructed for hydropower purpose with install capacity 183.9 MW, the dam height is 40 M, and crest length 1700 M, surface area at maximum water level 51 Km^2 , and dead & live storage are 171 and 668 Mm^3 respectively **.**

The main objective of the study is to analyze, and evaluate the impact of 100 year (1% chance) flood, and probable maximum flood (PMF) as inflow scenarios as well as the fully reservoir pool sunny day dam failure, through overtopping and piping failure, additional to other specific objectives .This objectives achieved by using two computer models all are from HEC family which are freely download (HEC-HMS and HEC-RAS).

Data collected from different sources, depend to the nature of the data .Hydrological data collected from ministry of water resources, meteorological data from Ethiopia Meteorological Agency and studies done in the basin. Dam and hydraulic data are collected from Ethiopia Electric Power and hydrological report done by Ministry of water resources. Collecting of the data was the first task in methodology and data analysis chapter followed by filling missing data, and generated data .Hydrological data filled and generated by using regression method . Assendabo was the friend station because of two reasons ,firstly No much missing data ,secondly it has long time series data .while for meteorological data normal ratio method has been used because of the annual index more than 10% .

Before entering the data into the model , analysis of hydrological model has been done with three years moving average , result of the three years showed that "the variation in original data are smooth is no curve or apparent trend cyclist" .The second important observation from the analysis of hydrological data is that only two stations effect of that flood that occurred in 2006 made damage to property and loss of life .Flow duration developed for Assendabo station for knowing the characteristic of the river for daily , monthly , and yearly time .

Input parameters for hydrological model estimated, this include probable maximum precipitation, which is calculated by using statistical method rather than meteorological method for the three subbasin . 1% chance precipitation estimated from one station and distributed uniformly for the three subbasins.

Hydrological model HEC-HMS developed for the basin after determined initial parameters or data required such as create subbasins, which done by using GIS software. Where upstream of the study dam has been divided into three sub basin namely upper sub basin , upper left sub basin , upper right sub basin with their corresponding areas .HEC-HMS is deterministic model , it requires to determine all the parameters to the model for getting results , therefore other parameters had been determined such as probable maximum precipitation , 1% chance precipitation .

HEC-RAS is one dimensional model for unsteady flow routing , cable of integrating complex channels and structure under very dynamic hydrologic condition .The model developed for the study area with the same sub basins created for HEC-HMS , each sub basin represent by reach for estimating water surface profile and peak flow for the three scenarios . Water surface is calculated for three return periods one year, 50 years , 100 years .Dam and spillway data model by lateral structural using stations and elevation method , flow will comes from upstream to downstream through the breach which will create in dam body , breach shape assume to be trapezoidal in shape growing with time .Two methods used for determined the breach parameters Bureau of reclamation (1982)and Von Thun & Gillette for showing the effect of breach to peak flow , all breaches made for piping failure with piping coefficient equal to 0.80 estimated by the model .PMF hydrograph from HEC-HMS model used as boundary condition in HEC-RAS PMF event , the same with 1% chance flood (result of HEC-HMS used as boundary condition) . Normal depth of Bidru station downstream of dam used as downstream boundary for HEC-RAS .Sunny day failure , the breach will occur under normal condition without any additional flow .

Results of the hydrological model HEC-HMS show that there is no overtopping over dam crest for the two scenarios , only overtopping over spillway from probable maximum flood (PMF) , in addition hydrograph for the three sub basins has been determined for two events PMF , and 1% chance .This hydrographs are used as boundary condition for HEC-RAS model .

The results of the HEC-RAS model estimated peak flow for PMF using Bureau and Thun are 25184, 24959 M^3/s respectively, while for 1% chance are 8151 and 3933.2 $M^{3/}$ s and for sunny day 571.4 and 494.5 M^3/s.

During discussing the results , some points come out such as: the result estimated by hydrological model HEC-HMS for PMF is not given accurate value , because PMP which estimated by statistical method (Hershield approach) tend to give lower value , the same with the result estimated for 1 % chance flood , because 1% chance estimated from one station only .

HEC-RAS results indicate that the peak flow from the breach is depend to the nature of the reservoir , when there is additional flow added to the reservoir peak flow will be more , and also the result of the model indicate the relation between breach parameters (Average width , time of failure) and the peak flow.

6.2 Conclusion and Recommendation

One aspect of dam is the reduction of loss of life and damage to downstream properties. Safety management studies often require the prediction of breaching parameters such as breach shape, outflow hydrograph and breach deformation time. Therefore, to be able to decrease and manage the dam related hazard especially in close to population centers, agricultural lands and industrial areas, some breach parameters must be predicted, modeled and mapped more accurately before the construction of dam. Otherwise, loss of life and property damages will increase. 1D hydraulic modeling gives satisfactory results for prediction of breaching and probable flood extent. However, more accurate and resolution digital elevation model is necessary to be able to model of dam breach floods.

Most of the empirical methods were based on case studies, therefore the predicted results depend on the different types and numbers of cases studies .For example, if 10 cases studies are used in a model, they will give a certain result, whereas if there are 20 cases, it will be more accurate perhaps or even it can also be missing the point because of the difference of shapes and sizes of breaches and type of materials forming the dam. So what can be suggested, is that each method should be very specific to a certain size and shape of certain type of embankment dams.

The computer model has successfully determined the peak flow and volume of water for different scenarios, for dam break analysis .Due to lack of data it was not possible to determine flooded area .Therefore it is recommended that this aspect should be considered in further study.

Hydrologic Engineering Center –River Analysis system (HEC-RAS) has ability to model piping and overtopping failure only for embankment dam, therefore there is a need for model has ability to model others embankment dam failures.
A hazard evaluation is the basis for selecting the performance standards to be used in dam design or in evaluating existing dams. When flooding could cause Significant hazards to life or damage to property, therefore the flood should design for probable maximum flood rather than any return period.

Omo Gibe River basin has a good contribution to the development of Ethiopia through providing electrical power, but only one meteorological first class is available, there for updating and increasing the number of station within the basin is recommended.

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Glossary of terms

Breach	Opening formed in the dam
Breach parameter	Parameter physically need to describe include depth , width , and side slope angle
General storm	Strom duration of 24 hours
Geometric data	How rivers and stream are connected
Local storm	Strom of duration less than 6 hours
Meteorology station class one	It is the station consist all meteorological parameters
Meteorology station class Three	It is the station consist of only two parameters (rainfall , and temperature)
Meteorology station class Four	It is the station which consist of only one parameter (rainfall)
Overtopping	Resulting of removal of soil particles in the inner body of the dam by the erosion action of seepage flow
Piping	Involving a reservoir level higher than the dam crest, and therefore, erosion of the structure by flow scouring effect
PMF	The flood that may be expected from the most severe combination of critical meteorological and hydrological condition that is reasonably possible in the drainage basin under study.

Appendices

Appendices A monthly flow data

Gilgle Assendabo monthly flow in M^3/s (1968-2008)

year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1968	186.4	177.3	115	186	311.5	1257.5	2075	4444.5	303.2	1321	403.2	231.7
1969	196	205.5	467	286.7	518	1594.3	2392	3600	2706	748	631	193
1970	128	154	280	301	358	1148	3309	6958	4087	1885	650	223
1971	153	73	19	172	376	1212	2578	3916	4066	2118	874	380
1972	209	177	184	308	629	746	3068	3997	2640	889	817	740
1973	335	114	32	160	520	1101	3070	4462	3955	1807	454	217
1974	140	80	125	93	536	1523	2623	2623	3875	1238	334	162
1975	92	102	91	240	187	817	2332	3144	2835	1680	321	161
1976	93	68	89	74	307	984	3303	3641	2987	717	839	246
1977	223	190	194	109	136	848	2893	3627	3453	1767	1989	360
1978	152	127	146	108	847	0	3019	4966	3276	2188	463	310
1979	227	353	228	298	536	858	2030	2554	1432	911	212	180
1980	112	67	96	187	294	1349	1981	2950	1707	651	228	119
1981	63	67	332	207	567	233	2879	1010	2879	1412	378	158
1982	138	64	72	83	240	504	1000	2103	3243	`1689	1152	754
1983	292	162	106	221	299	1070	1134	2482	4857	445	1844	617
1984	294	110	75	114	292	1419	2723	2894	2351	609	267	199
1985	80	52	39	169	461	1100	2304	3656	2752	1142	384	188
1986	66	73	125	108	137	1031	2323	2192	2612	1029	310	219

1987	102	62	233	166	497	1250	1784	1904	2317	1106	460	204
1988	124	121	52	44	108	324	1747	4895	3707	2962	620	231
1989	149	141	110	634	357	823	1631	2396	2356	1297	452	535
1990	255	182	301	269	507	1467	2856	2380	3654	5613	2820	297
1991	195	148	232	139	329	1044	2099	3539	2559	632	93	81
1992	20	41	4	39	371	1274	2674	5876	3594	2337	601	242
1993	227	299	139	611	1665	2354	4006	3727	2550	1888	1002	318
1994	183	89	135	150	669	2127	4088	5271	3778	851	341	182
1995	101	86	78	243	368	457	1409	2028	2373	584	240	193
1996	209				1291	2736	2650	3879	2725	1449	521	277
1997	204	353	76	516	816	2125	2187	3281	4171	4169	420	1673
1998	857	454	508	353	648	908	2660	5396	3010	2653	1030	460
1999	312	164	226	182	514	1066	2467	3369	1786	2250	780	341
2000	197	107	74	218	617	894	1951	2566	2634	2340	1095	476
2001	711	660	719	1133	2030	3644	3303	2325	1546	1546	893	1555
2002	303	165	224	292	221	890	1790	2368	1837	679	367	391
2003	380	148	279	300	178	729	2510	2639	2838	954	382	306
2004	200	138	127	201	416	978	2021	3071	2869	2410	614	409
2005	284	135	319	264	1261	1041	2107	4013	3772	1607	591	306
2006	238	258	303	363	446	1078	3754	5240	3266	1612	8856	728
2007	472	429	289	458	648	1733	3043	3576	4149	2037	505	289
2008	226	167	118	204	489	1360	2473	3521	2782	984	1667	440

Ghibe Nr Seka Monthly flow in M^3/s

year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1980	16	13	14	45	65	219	169	222	127	114	40	24
1981	14	8	21	12	33	100	213	254	250	141	43	20
1982					19	40	113	233	321	241	88	50
1983	30	22	31	12	56	62	121	292	337	233	68	34
1984	21	14	13	13	20	93	227	243	187	65	37	31

1985	16	11	10	25	49	166	256	380	223	83	43	25
1986	13	12	24	23	27	152	187	215	289	130	46	23
1987	18	15	39	27	68	176	261	252	282	150	64	37
1988	29	27	19	14	35	94	270	343	331	327	63	29
1989	21	19	19	33	30	85	260	265	236	126	53	89
1990	39	33	58	57	81	193	255	370	358	158	60	35
1991	29	21	20	31	54	132	234	299	265	85	38	32
1992	24	25	20	25	68	178	298	278	284	232	68	44
1993	38	28	20	54	211	242	372	261	245	164	76	36
1994	25	15	18	17	52	236	320	320	298	71	39	22
1995	13	10	10	48	65	96	180	211	226	103	48	54
1996					197	264	227	267	291	128	57	36
1997	27	14	21	48	76	209	195	230	146	289	276	120
1998	79	38	63	34	58	189	282	195	188	258	79	37
1999	24	12	16	19	55	91	188	211	148	156	54	38
2000	16	10	9	30	94	143	309	296	300	379	90	43
2001	71	65	71	64	216	388	259	169	388	226	100	75
2002	33	14	18	24	15	130	185	284	255	83	36	43
2003	22	15	21	37	19	228	339	264	330	62	34	22
2004	15	11	9	147	28	92	230	423	366	278	30	23
2005	10	2	16	80	210	184	330	458	484	198	108	74

Monthly flow of Bidru Awana Nr Sokuru station in M^3/s

year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1980	0.522	0.302	0.96	0.66	0.484	0.802	13	22				
1981												
1982	1.68	1.4	1.12	2	9	19	27	69	33	29	9	7
1983	6	4	3	2	13	10	19	33	20	40	1	1
1984	0.556	0.35				14	22	51	33	7	1	0.42
1985	2.5	2.3	3.1	9.4	6	41	48	40	40	17	4.6	3

1986	0.4	0.5	1.4	1.14	4.54	14	22	44	27	17	6.5	4.1
1987	2.8	2.2	1.9	1.3	1.3	2.1	18	50	54	33		
1988	8.7	6.2	6.3	12.4	11.2	33	47	60	44.5	15	6	9
1989	6.3	5	5.7	6	7.7	12	32	43	16	11	3.4	1.2
1990	0.42	0.36	7.3	2	8	15	26	64	37	38	27	22
1991	7	6	5	4.3	6.1	24	63	35	28	20	12	4.6
1992	5	5	4.4	8	11	20	34	48	46	12	7.6	4.6
1993	3.2	2.6	3.4	3.3	5.7	10	16	77	16	5.7	4.4	3.7
1994	7	6.2	7	9.3	10.6	8.7	13.5	24	30	5	3	2.3
1995	0.995				14	28	33	57	60	12	7.5	6.45
1996	5.6	4.3	4.9	8.3	7.2	23	15	18	29	33	16	11
1997	9	8	8	7.4	16	12.6	111.3	98	29	41	11	7.6
1998	4.5	3	3	2.6	3.6	7.4	30	22	6.5	9	4	2.7
1999	2.1	1.3	1.12	2.3	2.2	3.2	6.8	21	10.3			
2000	miss	1.9	1.97	1.95	5.6	13.5	8.7	6.64	28	7.8	3.2	2.4
2001	1.5	0.915	1.05	0.971	0.89	1.46	2.87	16.7	13	1.91	1.14	0.858
2002	0.85	0.54	0.52	1.34	0.45	4.85	61	16	43	1.12	0.667	1.05
2003	1.12	0.804	0.76	1.5	0.89	1.103	10	88	35	33	7.4	5.7
2004	1.1	0.804	0.74	1.5	0.87	1.1	10	87	35	32	7.4	5.5
2005	6.5	3.4	1.02	12	17	4.2	7	36	14	4	3	9

Monthly flow of Bidru Awana Nr Sokuru station in M^3/s

year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1981	0.522	0.302	0.957	0.658	0.484	0.802	12.6	22				
1982												
1983	1.7	1.4	1.12	2	9.6	19.2	27	69	33	29	8.6	8
1984	5.9	4.5	3.3	2.3	12.6	10.3	19	33	20	4.1	1.5	1.2
1985	0.56					14	22	50	33	7	1.4	0.42
1986	2.5	2.3	3.1	9.4	6	41	48	40	40	20	4.6	3

1987	0.38	0.48	1.41	1.14	4.5	14	22	44	27	17	6.5	4.1
1988	2.81	2.2	1.9	1.34	1.25	2.1	18	50	54	33		
1989	8.7	6.2	6.3	12.4	11.2	33	47	60	44.5	15.4	6.4	9.5
1990	6.3	8	5.7	6	8	12	32	43	16	11	3.4	1.17
1991	0.43	0.36	7	2	8	15	26	64	37	39	28	22
1992	7	6	5	4.3	6.1	24	63	35	28	20.2	11.8	6.3
1993	13	2.6	9.9	4.5	8	11	20	34	48	46	12	4.6
1994	3.2	2.6	3.4	3.3	5.7	10	16.4	77.5	16.5	5.8	4.4	3.7
1995	7	6.2	7.1	9.3	10.6	8.7	13.6	24	2.9	4.9	2.9	2.3
1996						14.4	28.3	33	57	60	12.5	6.4
1997	5.6	4.3	4.9	8.3	7.2	23	15	18	28	33	16	11
1998	9.1	7.8	8	7.4	16	12.6	111.6	98	29	41.3	16.8	7.6
1999	4.5	3	3.1	2.6	3.6	7.4	30	22	6.5	9.2	4	2.8
2000	2.1	1.3	1.1	1.3	2.3	2.2	3.2	6.8	21			
2001		1.9	2	2	5.6	13.5	8.7	6.6	2.8	7.9	3.1	2.4
2002	1.5	0.90	1.05	0.97	0.89	1.46	2.96	17	12.5	1.9	1.14	0.858
2003	0.85	0.54	0.52	1.3	0.45	4.85	61	16	43	1.12	0.667	1.05
2004	1.1	0.80	0.76	1.5	0.89	1.1	10.3	88	35	33	7.4	5.7
2005	6.5	3.4	1	11.8	17	4.2	6.9	36	14	3.7	3.1	8.7

Monthly flow for Kito Nr Jimma station in M^3/s

year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1982	5.7	3.2	3.7	4.8	7.8	13	17	34	34	37	32	31
1983	23.5	20	21	21	25	24	42	72	49	36	8.6	9.7
1984	15.7	14	14	16	18.4	31	33	39	42	32	32	33
1985	28	25	27	35	41	49	64	73	72	61	49	43
1986	38	31	28	26	27	44	56	36	40	30	17	12.6
1987	9.2	6.2	7.6	5.4	6.8	16.5	42	34.6	34.6	30	27	26
1988	28	28	34	36	45	54	79	95	97	97	63	54
1989	50	45	51	54	32	33	49	73	70.4	86	86	83.2

1990	71	64	72	61	66	76	103	100	82	32	19	13
1991	5.1	2.7	3	13	35.9			53	57	53	39	
1992				13	36			53	56	53	39	33
1993	29	27	30	35	42	57	81	73	67	65	46	33
1994	23	18	19					66	74	43	25	18
1995	11	9	13	23	27.2	27.3	36	33	40	29	11	8.5
1996									47	34	22	17
1997	15	8.9	6.3	6.6	7.8	16	14	29	30	30	53	60
1998	19	5	5.1	4.4	5.2	4.4	15.1	53	23.11	32.3	26	24
1999	5.9	3.6	4.1	4.2	9.6	10.4	19.1	33	23.11	32.3	26	24
2000	17	9.3	9.8	17.3	26	22	23	15	13	14		
2001	13	12	11	11	20	26	34	48	52	32		
2002	11	5.6	4.4	4.9	5	21	25	39	48	31	14	18
2003	14	10.4	13	18	13	25	42	43	50	36	30	25
2004	13	11	17	15	25	37	23	17	8.6	4.1	3.4	Miss
2005	3.1	2.3	3.1	2.2	2.2	8.3	25	34	53.4	33.8	23	18

Monthly flow for Awaitu at Jimma station in M^3/S

year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1982				0.51	6.1	41	59	114	75	33	19	
1983	4.6	2.5	2.5	7.5	4.4	3	39	204	148	99.4		
1984				0.95	0.78	1.8	14	59	47	58	10	6.8
1985	1.2	1.3	0.77	0.68	6.7	68.5	190.3	189	100	32	6.6	2
1986	0.098	0.651	0.61	13	1.9	48	118	55	124	23	6.9	3.8
1987	0.77	0.59	3.3	1.8	4.7	31	66	74	134	28	20	11
1988	4.67	4.1	3.1	2	3.8	31.6	111	146	138			
1989	0.718	0.26	3	25	11	62	149	131	151	91	12	18
1990	4.9	6.1	34	44	85	171	145	200	231	79	20	3.9
1991	2.1	1.9	8.2	27	81	155	227	224	218	44	9	7.3
1992	2.8	3.6	1.9	5.6	42	48	244	246	158	443	41	14

1993	7.4	11	9	37	131	176	125	75	108	80	15	5
1994	3.5	2.6	3.1	4	66.6	221	320	304	217	23	24	7
1995	1.3	1.3	3	6	21	34	141	111	205	66	47	6
1996	6.1	2.6	8	111.5	222	217	184	209	237	774	9.6	5
1997	6.7	1.7	7	21	79	329	160	108	89	285	202	125
1998	44	12.4	11.7	28	132	250	249	531	405	482	33	6.9
1999	2.1	1.5	2.4	8.8	30	56	35	144	45	128	24	3.9
2000	8.9	6.5	6.87	8.4	14.5	20	27	48	54	30	20	11
2001	6.4	4.2	4.6	5.1	6.7	15.6	57	35	27	26	7.5	5
2002	4.6	2.73	3.3	3.12	2.5	8.1	33	100	32	7.8	4	3.8
2003	3.7	2.5	3.5	5.6	2.7	8.9	25	24	21	11.4	3	3.1
2004	3	2.54	2.4	10.3	9.7	6.2	18.8	30.6	19.5	7.7	4.6	39
2005	4.3	2.4	2.7	5.9	39	15	43	64	111	37	5.8	3.3

Monthly flow for Awiatu Nr Babu

year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1989	33.4	28	37	55	48		315	533	540	86	77	101
1990	26	19	39	31	33	104	197			70	20	13
1991	9	5.6	8	12	14	490		1308	206	69	12	4
1992	6.8	4.5	5.2	10	65	82	184	431	253	86	71	27
1993	12	13	10	12.3	90	256	262	351	255	98	91	50
1994		7.4	9.2	9.4	36	138	578	817	403	165	22	4.7
1995	5.8	6.3	10.3	83.4	88.3	59	75	174	253			
1996								303	282	94	41	24
1997	18.5	10	9.3	28.6	93	249.8	209	290	187	486	391	139
1998			20	9.7	18.6	43	115	220	156	184	81	34.3
1999	20	10	10.8	10	22	60.6	151	163.7	107	177	67	
2000	13	6.9	7.2	12.2	32.2	39	120	210	202	222	94	52
2001	34	24	24	15.2	32	106	271	222	199	199	56	26
2002	21	8.6	13	10	13	90	140	155	113	41	19.2	24

2003	14.8	5.6	12	26	4.7	38	130	171	160	51		

Monthly flow for Bulbul Nr station in M^3/S

year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1986	87	83	84	89	92	240	697	585	652	205	68	42
1987	30	17	36	28	31	118	307	515	567	181	81	38
1988	19.6	16.7	3.1	0.12	11.5	39	32	1001	949	707	102	84
1989	15.3	11	6.3	45	32	126	405	577	583	158	49	84
1990	34	22	30	20	16.2	96	572	832	708	230	98	62
1991	29	0	0	0	0	0	0	447	549	97	47.4	24.4
1992	20	17.1	11.4	32	85	694	852	1021	723	433	320	107
1993	45	40	29	37	141	409	720	771	636	442	420	miss
1994	36	3.3	5.1	3	64	289	627	907	793	224	40	12
1995	14.2	15.5	16	61	106	64	108		58	103	114	111
1996								588	382	151	52	32
1997	27	15.2	8.4	20.5	58	341	408	622	287	523	660	178
1998	62	35.5	36	22	38	113	480	1100	735	553	113	50
1999	74	16	17	11.3	47	192	544	763	356	581	193	50
2000	24.5	15	12	15.2	94	275	413	655	610	382	212	57
2001	37	29	35	48	118	428	849	968	731	430	174	75
2002	51	33		14	7	74	408	1083	384	45	21	25
2003	14.2	5.6	11.2	11.8	4.23	35	837	1155	1363	165	48	23
2004	10.9	7.3	5.6	10.2	19.2	156	734	1370	1593	1232	66	38
2005	26	10.5	14.7	10.5	37	66	267	1723	2332	1363	72	79

Appendices B Water level for Bidru Nr Sekuru station in Meter

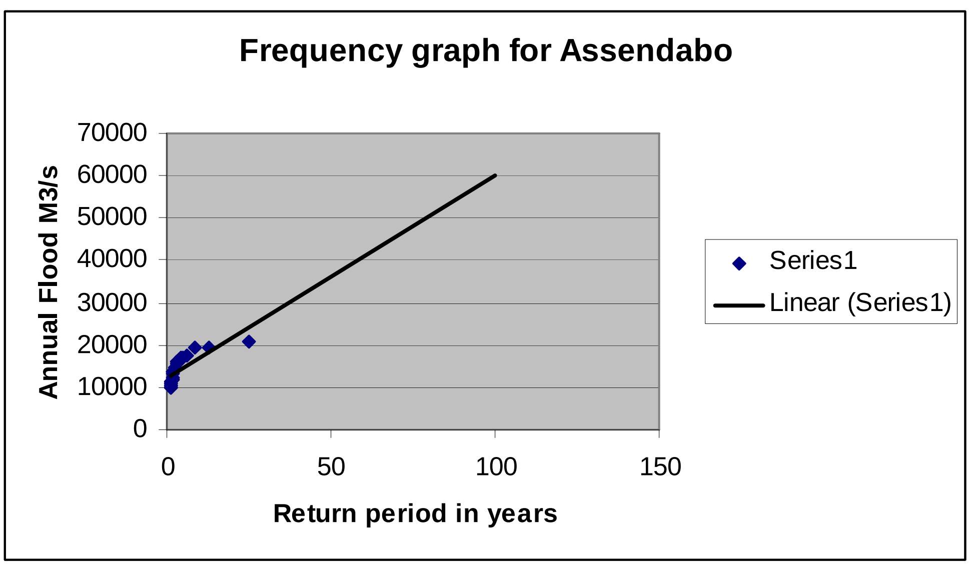
year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1981	81	6.34	5.42	6.85	6.3	6.3	6.3	6.5	13.3	18		

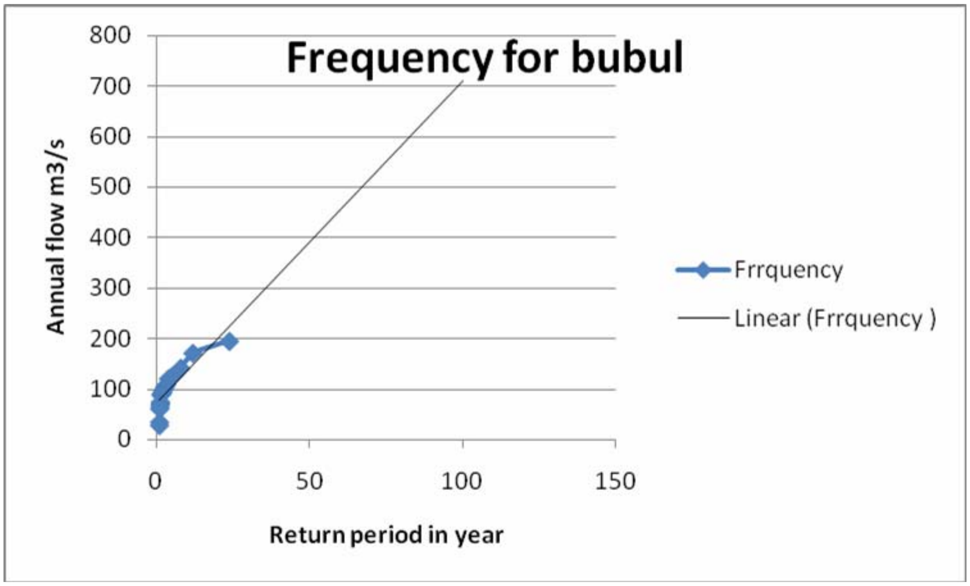
1982												
1983	3.3	2.8	2.8	3.3	7.2	10.2	13.5	25.5	15.6	15.1	7	7
1984	5.3	4.3	4.3	3.6	7.6	10	14	19.4	17	9.6	7.4	7.2
1985	6.4					11	15	24	18	8.2	5	4
1986	3.8	3.4	4.2	4.7	5.1	17	20	17.2	20	13	7.2	5.1
1987	5.9	5.7	7.2	6.9	9.05	7.05	14.7	23.2	19	16.4	11.2	8.8
1988	8.82	7.83	7.8	7.1	7.2	8	16	26	27	22		
1989	7	5.6	5.9	8.3	8	15	19.7	23	23	15.5	11.1	13
1990	11.2	9.7	10.8	10.8	11	11	13.6	21.2	24	16	14	7.22
1991	6.2	5.5	8.8	5.1	8.5	11.6	16.6	27	20	19.3	16.1	16
1992	14.3	12.4	12.5	11.4	13.3	17.5	30	27	21	22	17.2	14
1993	12.6	11.6	11.9	14	16	21	26	28	25	17.4	14.4	12
1994	10.4	8.9	10.6	10	13	15	20	30	19	13	12	11
1995	14.3	12.9	14.4	15.6	16.4	15.3	18.4	22	22.2	12.1	9.7	9
1996					16.6	21	22	28	28	18	14.5	14
1997	13.1	11	12.3	14.4	14.5	20	19.2	20.5	23.2	25	19.3	17.2
1998	16	14	15	14	18	18	18	38	33	21	26	15
1999	11.9	8.5	10.14	9.2	10.8	14.1	24	20	14	15	11	10
2000	8.6	6.7	6.4	6.7	8.4	8	10.2	14	19			
2001	6.6	7.1	7	11.8	15	14.6	13.4	16.1	12.7	9.52	8.51	
2002	7.3	5.5	6.2	5.9	5.7	6.5	9.12	17	15	8.2	6.4	5.6
2003	5.6	4.2	4.3	5.4	3.9	9.6	23	16.6	19.2	6.3	4.9	6.13
2004	6.4	5.3	5.2	5.7	5.2	6.1	10.9	31	23	24	14	13.1
2005	13.9	9.2	4.5	8.8	15	10	14	19	16	10	8.5	16
2006	14.6	16	26	14.2	11.8	18	32	18	10	6.5	57	5.1
2007	4.2	5.2	5.6	22.7	20.4	12.11	8.6	24	20	12.6	8.6	7.9
2008	7.4	4.1	2.2	2.5	5.7	9		21	19.2	10.7	10.41	7.3
2009	5	4.4	3.9	4.4	3.1	5.2	9.5	17	16	8.5		

Appendices C Mean monthly Evaporation for Jimma station mm

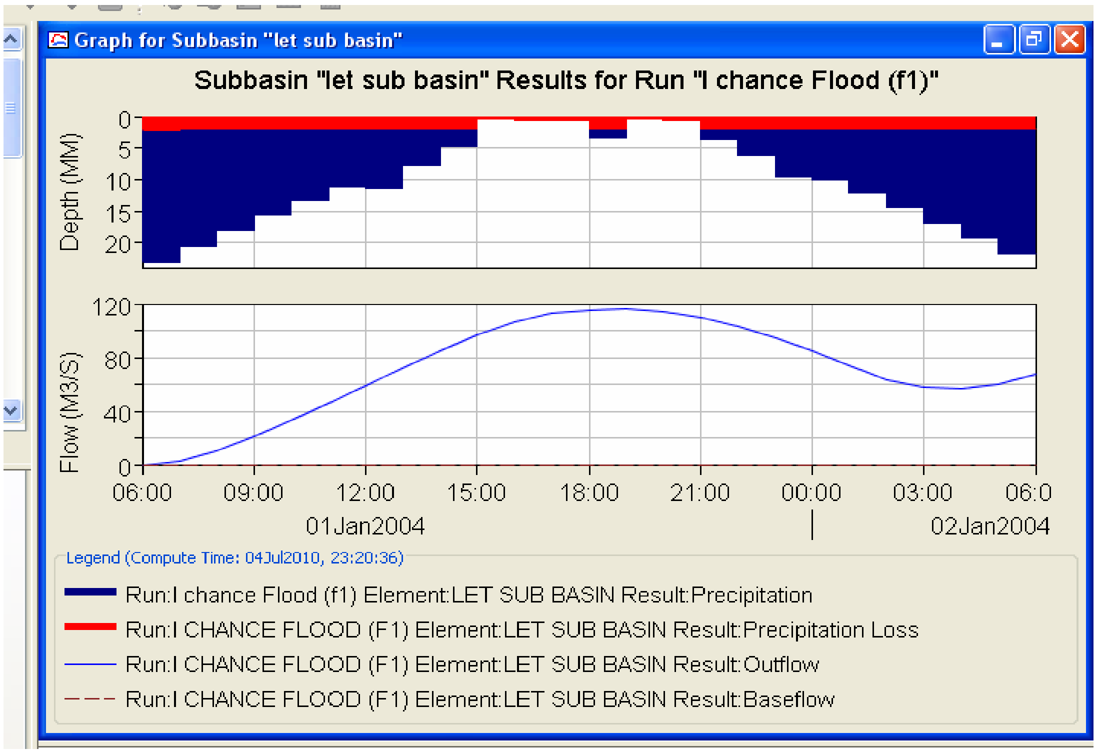
year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
2001	2.9	4.7	3.2	3.2	2.1	1.5	1.6	1.4	2.1	2.5	2.6	3.2
2002	2.8	4.1	3.1	3.5	3	2	1.8	6.3	2.1	3.7	3.4	3.2
2003	3.6	4.4	4.1	3.9	2.7	1.7	1.6	1.5	1.7	2.7	2.7	2.6
2004	3.6	4.4	4.1	3.9	2.7	1.7	1.6	1.5	1.7	2.7	2.7	2.6
2005	4.2	5.7	3.6	3.2	2.8	1.8	1.6	1.8	2.2	2.7	3.2	4.3

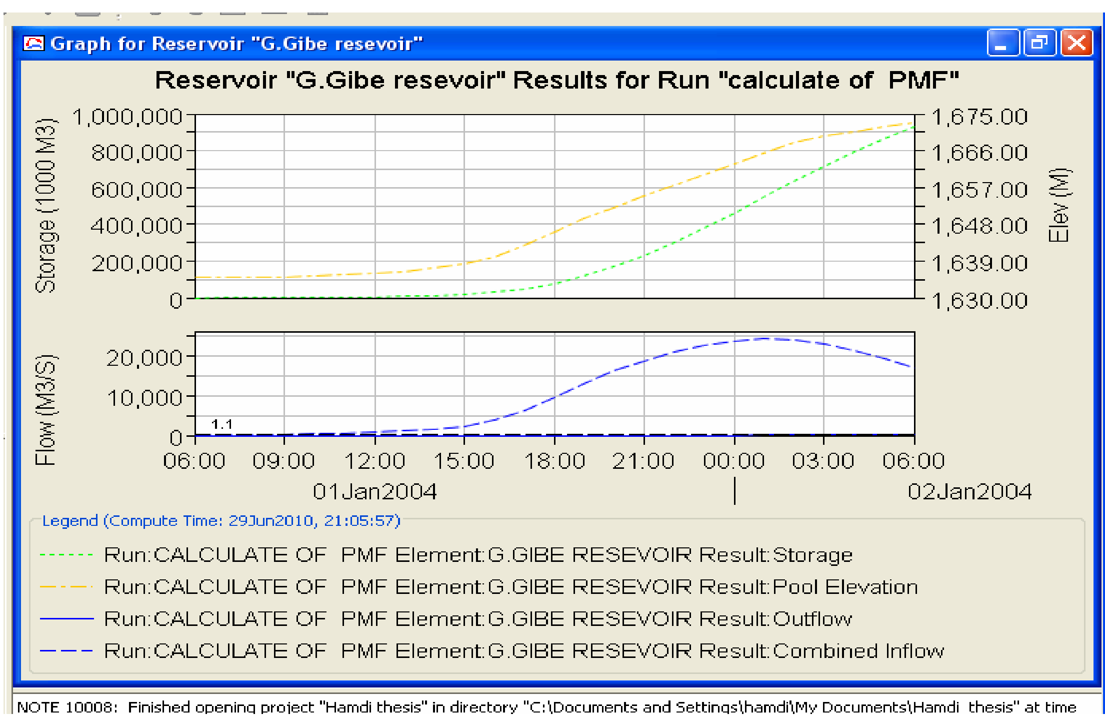
Appendices D Estimation of 1 % chance

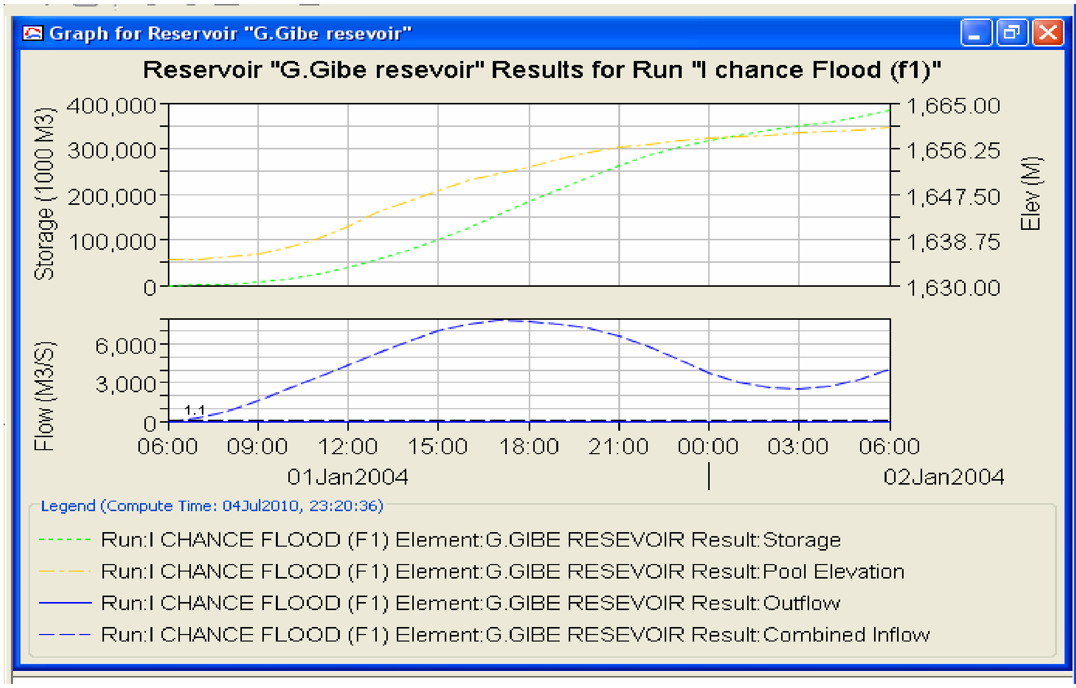


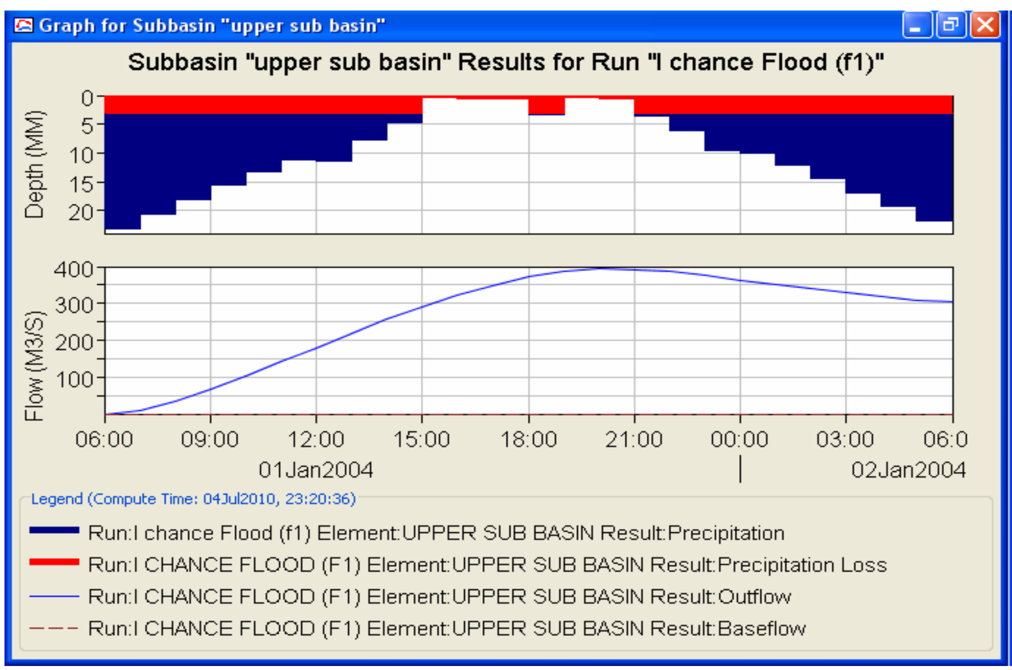


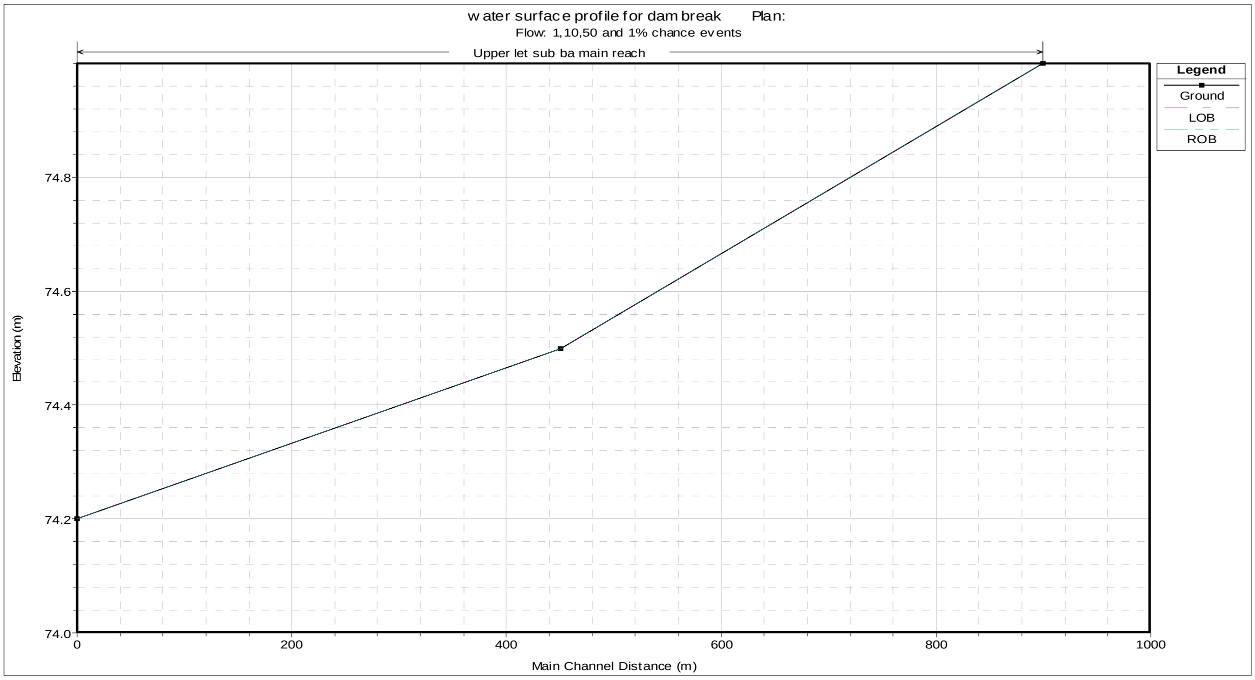
Appendices E HEC-HMS and HEC-RAS result

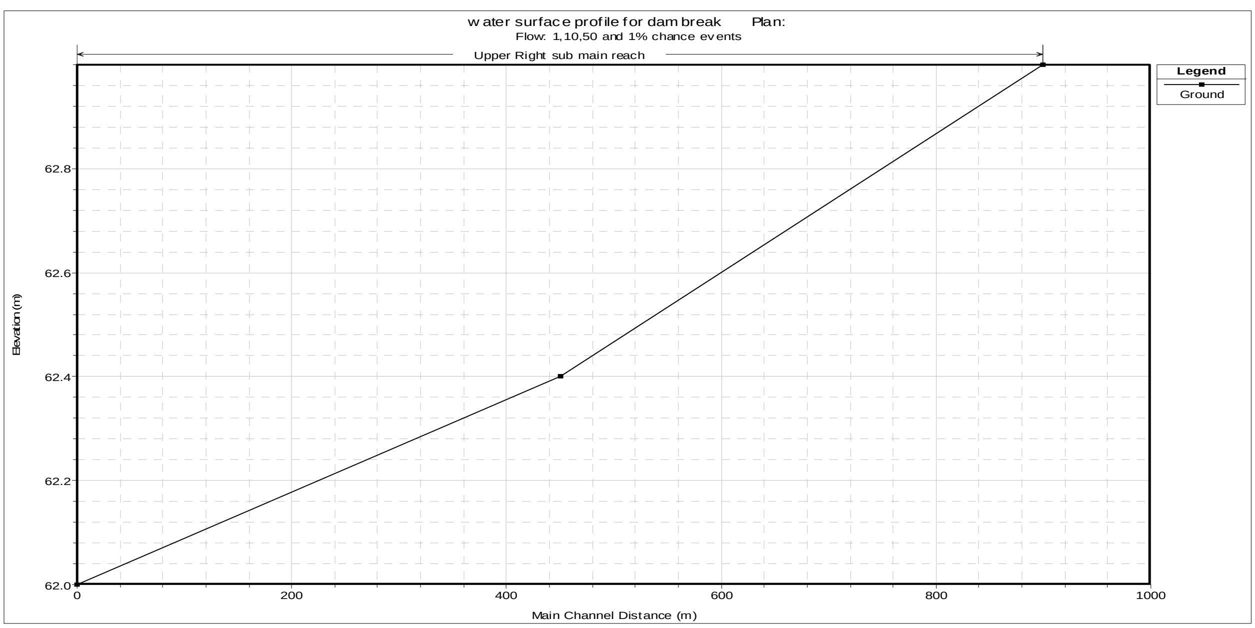

Graph for Subbasin "let sub basin"
Subbasin "let sub basin" Results for Run "I chance Flood (f1)"
Depth (MM)
Flow (M3/S)
01Jan2004
02Jan2004
Legend (Compute Time: 04Jul2010, 23:20:36)
Run:I chance Flood (f1) Element:LET SUB BASIN Result:Precipitation
Run:I CHANCE FLOOD (F1) Element:LET SUB BASIN Result:Precipitation Loss
Run:I CHANCE FLOOD (F1) Element:LET SUB BASIN Result:Outflow
Run:I CHANCE FLOOD (F1) Element:LET SUB BASIN Result:Baseflow


Graph for Reservoir "G.Gibe resevoir"
Reservoir "G.Gibe resevoir" Results for Run "calculate of PMF"
Storage (1000 M3)
Elev (M)
Flow (M3/S)
01Jan2004
02Jan2004
Legend (Compute Time: 29Jun2010, 21:05:57)
Run:CALCULATE OF PMF Element:G.GIBE RESEVOIR Result:Storage
Run:CALCULATE OF PMF Element:G.GIBE RESEVOIR Result:Pool Elevation
Run:CALCULATE OF PMF Element:G.GIBE RESEVOIR Result:Outflow
Run:CALCULATE OF PMF Element:G.GIBE RESEVOIR Result:Combined Inflow
NOTE 10008: Finished opening project "Hamdi thesis" in directory "C:\Documents and Settings\hamdi\My Documents\Hamdi thesis" at time


Graph for Reservoir "G.Gibe resevoir"
Reservoir "G.Gibe resevoir" Results for Run "I chance Flood (f1)"
Storage (1000 M3)
400,000
300,000
200,000
100,000
0
1,665.00
1,656.25
1,647.50
1,638.75
1,630.00
Elev (M)
Flow (M3/S)
6,000
3,000
0
06:00
09:00
12:00
15:00
18:00
21:00
00:00
03:00
06:00
01Jan2004
02Jan2004
Legend (Compute Time: 04Jul2010, 23:20:36)
Run:I CHANCE FLOOD (F1) Element:G.GIBE RESEVOIR Result:Storage
Run:I CHANCE FLOOD (F1) Element:G.GIBE RESEVOIR Result:Pool Elevation
Run:I CHANCE FLOOD (F1) Element:G.GIBE RESEVOIR Result:Outflow
Run:I CHANCE FLOOD (F1) Element:G.GIBE RESEVOIR Result:Combined Inflow


Graph for Subbasin "upper sub basin"
Subbasin "upper sub basin" Results for Run "I chance Flood (f1)"
Depth (MM)
0
5
10
15
20
Flow (M3/S)
400
300
200
100
06:00
09:00
12:00
15:00
18:00
21:00
00:00
03:00
06:0
01Jan2004
02Jan2004
Legend (Compute Time: 04Jul2010, 23:20:36)
Run:I chance Flood (f1) Element:UPPER SUB BASIN Result:Precipitation
Run:I CHANCE FLOOD (F1) Element:UPPER SUB BASIN Result:Precipitation Loss
Run:I CHANCE FLOOD (F1) Element:UPPER SUB BASIN Result:Outflow
Run:I CHANCE FLOOD (F1) Element:UPPER SUB BASIN Result:Baseflow


w ater surface profile for dam break Plan:
Flow: 1,10,50 and 1% chance ev ents
Upper let sub ba main reach
Legend
Ground
LOB
ROB
Elevation (m)
74.8
74.6
74.4
74.2
74.0
0
200
400
600
800
1000
Main Channel Distance (m)


w ater surface profile for dam break Plan:
Flow: 1,10,50 and 1% chance ev ents
Upper Right sub main reach
Legend
Ground
Elevation (m)
62.8
62.6
62.4
62.2
62.0
0
200
400
600
800
1000
Main Channel Distance (m)

Profile Output Table - Standard Table 1

File Options Std. Tables Locations Help

HEC-RAS Plan: suny day Profile: Initial Profile

River	Reach	River Sta	Profile	Q Total (m3/s)	Min Ch El (m)	W.S. Elev (m)	Crit W.S. (m)	E.G. Elev (m)	E.G. Slope (m/m)	Vel Chnl (m/s)	Flow Area (m2)	Top Width (m)	Froude # Chl
Upper sub basin	G.Gibe River	10	Initial Profile	1300.00	4.20	19.22	6.54	19.28	0.000024	1.47	2080.48	140.00	0.12
Upper sub basin	G.Gibe River	9	Initial Profile	1300.00	4.20	19.21	6.54	19.27	0.000024	1.48	2079.06	140.00	0.12
Upper sub basin	G.Gibe River	7.0	Initial Profile	1300.00	4.20	19.20	6.54	19.26	0.000024	1.48	2077.63	140.00	0.12
Upper sub basin	G.Gibe River ext	110	Initial Profile	1300.00	4.00	11.98	7.79	12.54	0.000125	4.45	1126.30	140.00	0.50
Upper sub basin	G.Gibe River ext	100	Initial Profile	1600.00	4.00	11.46	8.47	12.42	0.000236	5.85	1053.21	140.00	0.69
Upper sub basin	G.Gibe River ext	90		Lat Struct									
Upper sub basin	G.Gibe River ext	80	Initial Profile	1600.00	4.00	11.31	8.47	12.30	0.000253	5.96	1031.93	140.00	0.71
Upper sub basin	G.Gibe River ext	60	Initial Profile	1600.00	3.90	10.39	8.95	12.08	0.000455	7.39	760.31	116.00	0.93
Upper sub basin	G.Gibe River ext	40	Initial Profile	1600.00	3.90	8.95	8.95	11.64	0.001031	9.40	593.11	116.00	1.34
Upper sub basin	G.Gibe River ext	30	Initial Profile	1600.00	4.00	8.48	8.48	10.96	0.001244	9.52	635.79	138.91	1.44
Upper Right sub	Main reach	20	Initial Profile	1400.00	4.00	15.08	6.22	15.17	0.000066	2.02	1557.36	140.00	0.19
Upper Right sub	Main reach	10	Initial Profile	1400.00	4.00	15.05	6.22	15.14	0.000067	2.02	1553.05	140.00	0.19
Upper Let sub b	Main reach	0.5	Initial Profile	630.00	4.00	15.14	5.21	15.16	0.000012	0.87	1539.24	140.00	0.08
Upper Let sub b	Main reach	0.4	Initial Profile	630.00	4.00	15.14	5.21	15.16	0.000012	0.87	1538.48	140.00	0.08