ADDIS ABABA UNIVERSITY

Addis Ababa Institute of Technology

School of Civil and Environmental Engineering

Potential Micro-Hydropower Assessment in Upper Wabi Shebele River basin (The case of Adaba sub catchment)

A thesis Submitted to Addis Ababa Institute of Technology, School of Graduate Studies, Addis Ababa University In partial Fulfillment of the Requirement for the Degree of Master of Science inCivil and Environmental engineering

Major in Hydraulics Engineering

By

Mubarek Jemal

Advisor: Dr Ing. Geremew Sahilu

Addis Ababa

Ethiopia

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Chairman (Department of graduate committee)Signature Date

Declaration

I, the undersigned, declare that the thesis is my original work, & has not been presented for a degree at any university and that all sources of materials used for the thesis have been duly acknowledged.

Name MubarekJemal
Signature ___________

Addis Ababa Institute of Technology
Addis Ababa University

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List of Abbreviations

EEPCo	Ethiopian Electric Power Corporation
EEA	Ethiopian Energy Agency
GIS	Geographical Information System
GPS	Geographical Positioning System
GWh	Giga Watt-hour
HV	High Voltage
ICS	Interconnected System
kW	kilo Watt
kWh	kilowatt hour
MHP	Micro Hydropower
NGO	Non-Governmental Organization
NA	National Meteorological Agency
SHP	Small Hydropower
ENMSA	Ethiopian National Metrological service Agency
PVE	Present volume error
PPE	Present peak error
NSE	Nash sutcliffe Efficiency
ITCZ	Inter tropical convergence zone
DZ	Development zone
EMA	Ethiopian mapping agency
CSA	Central Statistics Agency
MoWR	Ministry of water resource
REF	Rural Electrification fund
TWh	Tera watt hour
PJ	picojule
DEUMB	Department of energy, energy utilization management & bureau
GIZ ECO	German International Cooperation Energy Coordination Office

Abstract

This study assesses the Micro hydropower potential of theAdaba sub catchment in UpperWabishebele basin. HEC-GeoHMS and Arc Hydro tools coupled with Arc GIS 10.2 were used for watershed analysis and relate with the HEC-HMS model to simulate the rainfall-runoff processes after parameters for losses, runoff transform, & open channel routing were estimated.

The model was calibrated & validated for the period 1984-1986, & 1987-1989 respectively, the performance of the model was evaluated using the statistical tools such as Nash-Sutcliffe Efficiency (NSE), Coefficient of determination (R^2), Percent Volume Error (PVE), and percent error in simulated peak (PEP).

After the potential site was selected & the gross head was determined the discharges of these sites were transferred from the model by Area-ratio method. Then Flow duration curves were prepared for each site.

A total of 17 points were determined& the total potential capacity was found to be 8.04 MW. In addition to this appropriate type of scheme was proposed & all of them turned to be Diversion & canal hydropower plant. In order to facilitate favorable condition for local decision makers as to which sites should be given the top priority the potential sites have been ranked considering different criteria.

KEY words:- Micro hydropower, HEC-HMS, HEC-GeoHMS, Calibration

Chapter 1

1. Introduction

1.1General

While the global energy demand is skyrocketing, the hunt for cleaner, cheaper and more sustainableenergy resources has been a hot issue in the past few decades. Following that, many nations havemanaged to reduce their dependence on traditional energy systems while others find it unaffordable due totheir economic situations and technological incapability.

According to United Nations 1.6 billion people live in extreme poverty. Around 2.4 billion people rely on traditional biomass for cooking and heating purposes and roughly a quarter of the world's population with 1.6 billion people does not have access to electricity. Access to modern, safe and affordable energy and energy services is considered as one of the attributes having a great potential to reduce poverty (John, 2009).

1.2The Energy sector

The Ethiopian energy sector, regardless of thenation's enormous energy potentials, is highly dependent on traditional fossil fuel energy sources andbiomass such as wood, animal dung etc. and these traditional energy sources are one cause for the massive deforestation. For instance change in forest cover between 1990 and 2010 Ethiopia lost an average of 140,900 ha or 0.93% per year. In total between 1990 & 2010, Ethiopia lost 18.6% of its forest cover or around 2,818,000ha(UN, 2010).

As more than 80% of the country's population is engaged in the small-scale agricultural sector and live in rural areas, traditional energy sources represent the principal sources of Energy in Ethiopia. A survey by the Central Statistics Agency (CSA) in 2004 showed that about 71.1% of the total households use kerosene for lighting followed by firewood (15.7%) and electricity (12.9%). A higher proportion of urban residents use electricity (75.3%) for lighting, while the use of kerosene (80.1%) and firewood (18.5%) are predominant in rural areas. Major types of cooking fuel used by all households are firewood, leaves, dung cakes and kerosene. The study by CSA at the country level, suggests that about 81.4 % of the households use firewood, around 11.5 % cook with leaves and dung cakes and only 2.4 % use kerosene for cooking. The majority of rural households use firewood (84.4 %) and few of them (12.7 %) use leaves and dung cakes.

The use of modern source of cooking fuel such as butane gas, electricity and kerosene for cooking is uncommon in the rural areas (0.4 %). Use of kerosene is common in urban areas and stands at 13.8 % following firewood (65.4 %). Charcoal (7.7 %), electricity (2.4 %) and leaves (5.3 %) are also used by urban households. On the other hand, only 0.2 % of the households in rural areas are observed to use charcoal for cooking(MoWR, 2012).

The WabiShebele basin energy requirements for a large section of both the rural & urban population are met from the immediate environment from biomass, animal dung & agricultural residues. Biomass resources meet 99 percent (woody biomass 83%, animal dung 13%, agricultural residue 1.1%, & charcoal 1.2%) of total energy requirements in the basin at present & their dominant role is expected to continue in the future. in the case of (MoWR, 2005).

On the other hand there are perennial rivers in the upper WabiShebele river basin. In this region they receive average yearly rainfall of 1001-1500mm. Then, micro-hydropower is of interest as a renewable energy form of mankind for providing electricity for rural community.

Throughout the entire history of mankind, nature has offered us all its resources for free. However, most of these resources are limited by nature and the unwise and improper use of these resources is leading us to non-sustainable lifestyles and it is endangering the future of the planet. The ecosystem is under enormous pressure as its carrying capacity is being washed away due to our ever increasing consumption that results in overexploitation of resources and devastating pollution. As the general solution to these problems people have showed their concern to the environment by fighting all the odds against it; environmentalists have said their say and governments, in spite of lack of adequate action, have also proved their commitment to fight the challenges of our planet; educational institutions are producing educated personnel in the area and researchers have underlined the importance of sustainable development to the planet's future.

Consequently, replacing our traditional fossil fuel dependent energy systems with sustainable energy systems has got worldwide attention in the past few decades. Wind, solar, hydropower and geothermal energies are among the sustainable and/or renewable energy sources that could satisfy our energy demands while least affecting the balance of the ecosystem. While nuclear energy has been used as an alternative source of energy for over half a century, recently biofuels (such as ethanol, biodiesel, etc.) have also become widely accepted alternatives to fossil fuels. Generally, our day to day activities are creating grave social and environmental impacts to our planet Earth and the question of having ample and sustainable energy resources has become one of the biggest diplomatic challenges of our time. As the law of conservation of energy says, energy cannot be created or destroyed, but it can only be transformed from one form to another. However, our demand for convenient forms of energy (such as heat, electricity, etc.) is growing alarmingly and so is the pressure we bear on the planet. According to a World Bank report, the world average energy use per capita has increased from 1,338 kilograms in 1971 to 1,819 kilograms of oil equivalent in 2007. On the other hand, GWEC (2008) reported that the power sector is the largest source of emission accounting for 40% of CO_2 emissions and about 25% of overall emissions.(Dawit, 2010) Consequently, solving problems related to this sector is of vital importance to the environment, the global society and our economy.

In addition to the environmental protection & social equity the provision of adequate and affordable rural energy in general and electricity in particular- especially renewable energy-based have direct synergy with poverty reduction strategies that Ethiopia is now implementing. An affordable source of electrical as well as mechanical and thermal energy could encourage the establishment of agro-processing and cottage industries, which would create employment opportunities in rural areas and increase disposable income.

Enabling individual rural households to have access to amenities like affordable electricity, which is currently restricted to urban areas, would improve the quality of education, health, and other services. Additionally, electricity would improve the attractiveness of rural areas and would provide an incentive for better-trained persons to serve in more remote areas and encourage the establishment of government offices and associated services.

According to the report of (MoWR, 2012.) the Ethiopian Annual per capita consumption of electricity is 100 kWh per year. The same figure for the Sub-Saharan Africa is 510 kWh. This reveals that most of the energy usage is still from traditional energy sources such as wood and animal waste. Moreover it also informs the fact that with the country’s economic development and improvement of the per-capita income, there will be huge potential for consumption of electricity within the country.

This figure seems to be out of all bounds for these countries at the moment. The terrain in Ethiopia and the settlement pattern are such that distributed generation seems to be the only viable option to increase the level of electrification in any significant manner.

In Ethiopia the development of hydro potential until recently was very low, considering the long years of attempting such development. Major factors causing this low development, especially in connection with the grid, are the high investment cost for power generation and lack of local capacity other than that of the government institution, EEPCo, for expansion of the grid system. The challenges which face us on climate change are huge and will require a global agreement. But they also need small scale answers with individual households and businesses taking responsibility for doing something (Graham et al. 2010). So to address this climate warming issue replacing our traditional fossil fuel dependent energy systems with sustainable energy systems is a un-questionable issue. Wind, solar, hydropower and geothermal energies are among the sustainable and/or renewable energy sources that could satisfy our energy demands while least affecting the balance of the ecosystem. The purpose of this research is to assess the potential micro hydropower sites of less than 500 KW plant capacities inAdaba sub catchment.

1.3 Statement of problem

According to the CSA projection for the year 2016, the total number of population in Adaba is 176,229. out of this 18,620(10.5%) lives in urban Area & 157,609 (89.5%) lives in rural Area. The Rural Area population is characterized by scattered settlement pattern, & far away from the main grid, which leads for high expense of transmission line, this makes difficult for the inter connected system to electrify this rural villages.

As a result the majority of the population in the rural area has to rely on biomass based energy system, which has resulted in massive deforestation and soil erosion in the region. Moreover in Adaba sub catchment Considerable amounts of effort and time are now exerted for the collection of fuels which divert productive human capital from agriculture and other income generating activities.

But in this region there are perennial rivers combined with good topography, which makes Micro-Hydropower suitable near these villages. Having these facts in mind the potential for micro hydropower for this region is going to be assessed so that the energy problem will be solved to some degrees & the use of biomass energy sources reduced by substituting renewable energy system.

1.4 Research question

- ✓ Is micro hydropower suitable option for sustainable energy systems in Adaba sub catchment?
- ✓ What really does the Ethiopian micro hydropower sector looks like?
- ✓ How the cost does for Micro-Hydropower varies?

1.5Objective

1.5.1 General objective

The overall objective of this study is to assess the potential for Micro-Hydropower development in Adaba sub catchment in the upper Wabi Shebele river basin, So as to minimize the deforestation of woods for the purpose of energy by replacing traditional fossil fuel dependent energy systems with sustainable energy systems.

1.5.2 Specific objectives

Some of the specific objectives adopted to meet the main objective include:

- ✓ To select the possible potential site & identify the demand center
- ✓ To estimate the potential of selected site.
- ✓ Ranking of the sites.
- ✓ To produce GIS based map showing the location of the viable hydropower site.

Chapter 2

2. Literature review

2.1General

Hydropower is generated from water moving in the hydrological cycle, which is driven by solar radiation. Incoming solar radiation is absorbed at the land or sea surface, heating the surface and creating evaporation where water is available. A large percentage close to 50% of all the solar radiation reaching the Earth's surface is used to evaporate water and drive the hydrological cycle. The potential energy embedded in this cycle is therefore huge, but only a very limited amount may be technically developed. Evaporated water moves into the atmosphere and increases the water vapor content in the air. Global, regional and local wind systems, generated and maintained by spatial and temporal variations in the solar energy input, move the air and its vapor content over the surface of the Earth, up to thousands of kilometers from the origin of evaporation. Finally, the vapor condenses and falls as precipitation, about 78% on oceans and 22% on land. This creates a net transport of water from the oceans to the land surface of the Earth, and an equally large flow of water back to the oceans as river and groundwater runoff. It is the flow of water in rivers that can be used to generate hydropower, or more precisely, the energy of water moving from higher to lower elevations on its way back to the ocean, driven by the force of gravity. (Kumar et al. 2011)

2.2 Warming of the earth

Greenhouse gases such as carbon dioxide (CO_2) absorb heat (infrared radiation) emitted from Earth's surface. Increases in the atmospheric concentrations of these gases cause Earth to warm by trapping more of this heat. Human activities especially the burning of fossil fuels since the start of the Industrial Revolution have increased atmospheric CO_2 concentrations by about 40%, with more than half the increase occurring since 1970. Since 1900, the global average surface temperature has increased by about 0.8 °C (1.4 °F). In the case of an institution(National Academy of science)

Over the last decades, the temperature in Ethiopia increased at about 0.2° C per decade. The increase in minimum temperatures is more pronounced with roughly 0.4° C per decade. Precipitation, on the other hand, remained fairly stable over the last 50 years when averaged over the country. However, the spatial and temporal variability of precipitation is high, thus large-scale trends do not necessarily reflect local conditions. Most of the global climate models project an increase in precipitation in both the dry and wet seasons. Studies with more detailed regional climate models, however, indicate that the sign of the expected precipitation change is uncertain. The temperature will very likely continue to increase for the next few decades with the rate of change as observed. (Marius keller, 2009)

2.3 History of hydropower

Humans have been harnessing water to perform work for thousands of years. The Greeks used water wheels for grinding wheat into flour more than 2,000 years ago. The evolution of the modern hydropower turbine began in the mid-1700s when a French hydraulic and military engineer, Bernard Forest de Bélidor wrote Architecture Hydraulique.

In 1880, a dynamo driven by a water turbine was used to provide arc lighting– a technique where an electric spark in the air between two conductors produces a light – to a theatre and storefront in Grand Rapids, Michigan, and in 1881, a dynamo connected to a turbine in a flour mill provided street lighting at Niagara Falls, New York; both of which used direct current technology. The breakthrough of alternating current, the method used today, allowed power to be transmitted longer distances and ushered in the first U.S. commercial installation of an alternating current hydropower plant at the Redlands Power Plant in California in 1893. The Redlands Power Plant utilized Pelton waterwheels driven by water taken from the nearby Mill Creek and a 3-phase generator which ensured consistent power delivery. In the case of an institution (Office of energy, efficiency & renewable energy, 2016)

Early hydropower plants were much more reliable and efficient than the fossil fuel-fired plants of the day (Baird, 2006). This resulted in a proliferation of small- to medium-sized hydropower stations distributed wherever there was an adequate supply of moving water and a need for electricity. As electricity demand grew, the number and size of fossil fuel, nuclear and hydropower plants increased. In parallel, concerns arose around environmental and social impacts (Thaulow et al, 2010 as cited in Kumar et al. 2011).

Hydropower plants (HPP) today span a very large range of scales, from a few watts to several GW. The largest projects, Itaipu in Brazil with 14,000 MW and Three Gorges in China with 22,400 MW both produce between 80 to 100 TWh /yr (288 to 360 PJ/yr). Hydropower projects are always site-specific and thus designed according to the river system they inhabit. Historical regional hydropower generation from 1965 to 2009 is shown in Figure 2.1.

The great variety in the size of hydropower plants gives the technology the ability to meet both large centralized urban energy needs as well as decentralized rural needs. Though the primary role of hydropower in the global energy supply today is in providing electricity generation as part of centralized energy networks, hydropower plants also operate in isolation and supply independent systems, often in rural and remote areas of the world. Hydro energy can also be used to meet mechanical energy needs, or to provide space heating and cooling.

Hydropower plants do not consume the water that drives the turbines. The water, after power generation, is available for various other essential uses. In fact, a significant proportion of hydropower projects are designed for multiple purposes. In these instances, the dams help to prevent or mitigate floods and droughts, provide the possibility to irrigate agriculture, supply water for domestic, municipal and industrial use, and can improve conditions for navigation, fishing, tourism or leisure activities. One aspect often overlooked when addressing hydropower and the multiple uses of water is that the power plant, as a generator of revenue, in some cases can help pay for the facilities required to develop other water uses that might not generate sufficient direct revenues to finance their construction.

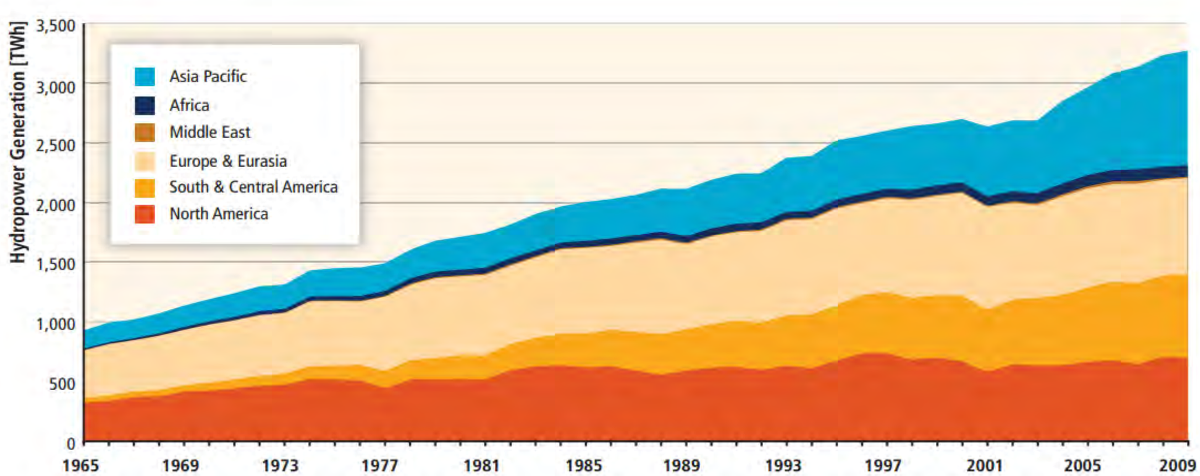

Figure 2.1 Hydropower generation (TWh) by region (online).

2.4 Hydropower Resource of Ethiopia

The birth of modern energy in Ethiopia dates back to the late 19th century and the country is endowed withenormous water, solar, wind and geothermal energy potentials. However, today, more than a centurylater, only a tiny part of the potentials are utilized and only 48.3% of the population has access toelectric energy As of July 2012.**(**EEPCO, 2015**).**

The power system is reliant on hydropower, which accounts for 94% of generation. Ethiopia is estimated to have the capacity to produce 45,000 MW of hydropower, out of this it is only 2100 MW or less than 5% of the total exploitable amount is exploited(Dereje 2013) Existing Power plants & their detail information is given on Table 2.1.

However, given the vulnerability of focusing only of hydropower development, the Government is diversifying and developing geothermal and wind power as well. Of the above exploitable potential in the country, about 1500 MW to 3000 MW would be suitable for small scale power generation including Pico and Micro hydropower. (Abebe, 2011).

Generally, there are 12 major basins in the country with 8 of them being river basins, 3 dry basins while the remaining one being lakes basin (MoWR, online). Most of these water basins have very special nature due to the landscape of the country in that in many cases it is possible to use the water of those basins as a source of Ethiopian Energy Systems: Potentials, Opportunities and Sustainable Utilization hydropower and use the same water for irrigation in the downstream. Another special feature of the Ethiopian rivers is the fact that most of the major rivers are sourced in the central part of the country and stream to all directions with some of the river basins covering very large areas. Wabishebelle River, for instance, covers an approximated area of more than 202,000km2 while the Blue Nile basin covers about 200,000km2 of land (Ibid.) the hydropower and irrigation potentials of the main river basins of the country are summarized in Table 2.2.

Table 2.1 Existing Power plant(Ethiopian electric power corporation, online)

No	Power Plant	completion of construction	catchment Area(Sq.km)	Dam height (m)	Dam crest Length	Storage capacity Of the Reservoir_(Mm^3)	Maximum water Level (m.a.s.l)	Installed capacity_(MW)	Firm Energy_(GWh)	Rated discharge (m^3/s)	Net design head (_m)
1	Beles	2010	14,200	35	125	37,307	1787	460	1540	160	330
2	Tekeze	2009	30,390	188	450	9,310	1140	300	971	184	120
3	Gibe II	2009	Weir	46.5	171.1	Weir	Weir	420	1244	4*24	485
4	Gibe I	2004	51	41	1700	688	1671	210	622	4*25	223.4
5	Gibe III	2015	34,150	234							
6	Wakena	1988	5,300	42	2000	763	2522.9		434	4*15	297
7	Fincha	1972	170	22	340	406	2217	134	740	4*7.4	594
8	FinchaAmertiNeshe	2011	29.5	38	1000	2.33	2232	97	230	2*9.3	587.5
9	Tis Abay I	1953	15,300	Weir	Weir	9100	1787	12	33.7	11.4	46
10	Tis Abay II	2001	15,300	Weir	Weir	9100	1787	72	442		
11	Koka	1960	200	23.8	458	1500	1590.7	72	80	3*48	42
12	Awash 2	1965	2.73(weir)	10	88	22.4	1539.0	42	135	65.6	60
13	Awash 3	1971	0.063(Weir)	20	125	22.4	1475.3	32	135	66.2	60
14	Sor	1990						5			
15	Yadot	1990						0.35			
16	Dire Dawa	2004						40			
17	Awash 7kilo	2004						35			
18	Kality	2004						11.2			
19	Aluto	1970						7.2			
20	Adama wind	2012						51			
21	Ashegoda	2012						30			
22	Adwa							3			

Table 2.2 Hydropower & irrigation potential of the basin (Dawit, 2010)

No.	River basin	Hydropower potential (GWh)	Irrigation Potential ('000ha)	Potential hydropower sites
1	Abbai (Blue) Nile	55000/78820*	1800	132
2	Omo-Ghibe	26026/36560*	90.4	23
3	Baro-Akobo	19826	631	39
4	GenaleDawa	9270	1070	23
5	Tekeze	8384	186.9	15
6	Wabishebelle	7457	209.3	18
7	Awash	5589	206	43
8	Rift Valley Lakes	12240/800*	131	6
9	Mereb	na**	5	na
10	Aysha	na	0	na
11	Denakil	na	na	na
12	Ogaden	na	na	na
	Total	**143,792/166,706***	**4329.6**	**299**

* different data from different references: both values are put whenever the difference is considerable

** data's are not available

2.5Energy Law of Ethiopia

The energy sector policy of Ethiopia's government is to enhance and expand the development and utilization of hydrological resources for power generation with emphasis on mini hydropower development. It is also to promote self-reliance in the fields of technological and scientific development of energy resources, and introduce energy conservation and energy saving measures in all sectors with ecologically and environmentally sound practices. The policy places high priorities on hydro-power resource development, as the water resources are the countries most abundant and sustainable energy forms. It is by the electricity proclamation No. 86/1997 that Ethiopian Electric Agency (EEA) established in June 1997, as an autonomous federal government organ, becoming fully operational since the beginning of 2000 and accountable to the ministry of infrastructure. The agency has power and duty to supervise and ensure, including issue, suspend and revoke license for the generation, transmission, distribution and sale of electricity. To study and recommend a tariff and collet license fees in accordance with rates to be prescribed by regulations (Federal NegaritGazeta, P.No. 86/1997 as cited in Nardos 2011).

The rural Electrification fund(REF) was established on 6th February 2003 by proclamation No. 317/2003, which is an institution responsible to provide loan and technical services for rural electrification projects to be carried out by private operators, cooperatives and local communities and more specifically for those projects operating on renewable energy sources; and to encourage the utilization of electricity for production and social welfare purposes in rural areas. “rural electrification project” means a single activity of designing, constructing, generating, transmitting, and performing other related activities to achieve the distribution of electricity in rural off-grid areas (Federal NegaritGazeta, P.No. 317/2003 as sited in Nardos, 2011).

According to the investment proclamation No. 116/1998, electricity generation from hydropower is open to both local and external developers without limit on capacity. But electricity generation from sources other than hydropower is reserved for the government and local developers. Development of non-hydro plants larger than 25MW is left to the government; and those below this threshold are open to the local private sector. The government remains the sole operator of the national grid (Federal NegaritGazeta, P.No. 116/1998 as cited in Nardos 2011)

2.6Rural Electrification in Ethiopia

Rural electrification is the process of providing electrical services to rural areas: generally regions with sparse population where agriculture is the dominant livelihood.(Muluken 2014)

The three most important rural energy sources, in their order of importance, are fuel wood, dung and agri-residue; while the three most important end-uses are “mitad” baking, other cooking and lighting. The implication of this is that, if rural households are provided with electricity, even for lighting, the gain in terms of environmental protection of rural areas is significant. In view of the above there is a huge market for investors in the area of rural electrification. And the huge market could be taken as the other opportunity for the development of the energy sector in general and rural electrification in particular. But the rural electrification is facing some problems. In extending electrical power to low income areas where domestic consumers are poor and the demand of electricity for productive purposes is absent; low levels of demand, low revenues, and high initial costs are obstacle to investment. Studies indicate that the key constraint to energy supply for rural communities is access to the initial capital needed to buy the equipment to harness the resource. This forced rural communities to choose energy options that are cheap on a day-today basis, but offer poor quality energy and are expensive over the longer term (Aklilu, 2010. as sited in Ermias, 2014)

According to (Munasinghe, 1990 as cited in Muluken 2014),rural electrification can be achieved through:

- ✓ Extensions of national or regional distribution systems, or grids, to rural areas.
- ✓ Isolated generators electrifying Mini-grids to supply a community, or
- ✓ Isolated generators electrifying a single house or facility.

2.7 Basic Concept of Hydropower

A hydro scheme requires both water flow and a drop in height (referred to as 'Head') to produce useful power. The power conversion absorbs power in the form of head and flow, and delivering power in the form of electricity or mechanical shaft power. Main parts of an hydro plant is shown on Fig 2.2. No power conversion system can deliver as much useful power as it absorbs –some power is lost by the system itself in the form of friction, heating, noise, etc.

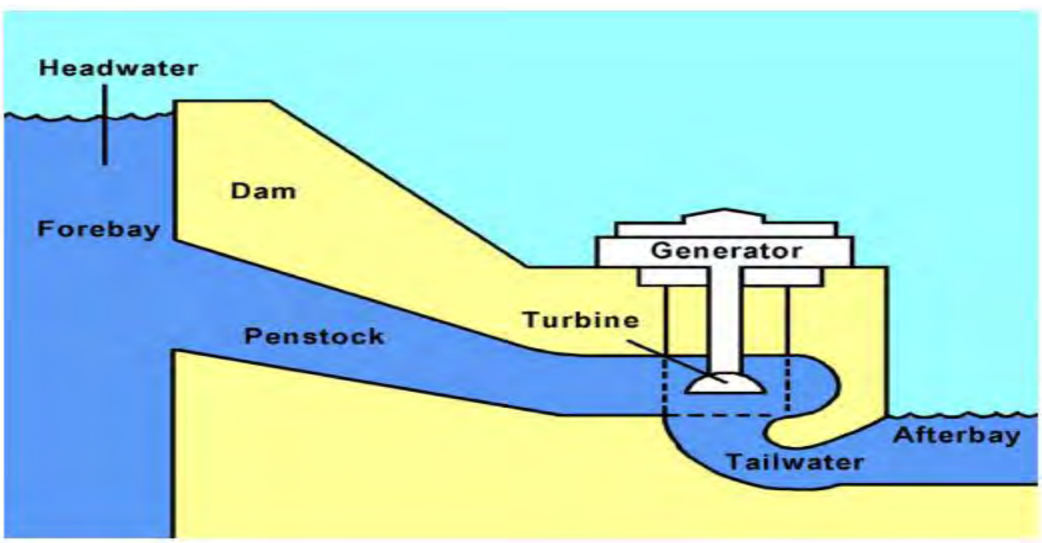

Figure 2.2 Main parts of Hydropower plant (U.S. Department of the interior Bureau of Reclamation power Resources office, 2005)

The power conversion equation is :

Power input = Power output + Loss or

Power output = Power input × Conversion Efficiency

The power input, or total power absorbed by the hydro scheme, is the gross power, (P_{gross}). The power output is the net power (P_{net}). The overall efficiency of the scheme (Fig.2.3) is termed E_o.

$P_{net} = P_{gross} \times E_o$ in kW

The gross power is the product of the gross head (Hgross), the design flow (Q) and a coefficient factor (g = 9.8), so the fundamental hydropower equation is:

$P_{net} = g \times H_{gross} \times Q \times E_o$ kW (g=9.8)

Where the gross head is in meters and the design flow is in cubic meter per second. Eo is derived as follows:

$E_o = E_{civil\ work} \times E_{penstock} \times E_{turbine} \times E_{generator} \times E_{drive\ system} \times E_{line} \times E_{transformer}$

Usually $E_{civil\ work}$: 1.0 - (Channel length × 0.002 ∼ 0.005)/ H_{gross}

$E_{penstock}$: 0.90 ∼ 0.95 (it's depends on length)

$E_{turbine}$: 0.70 ∼ 0.85 (it's depends on the type of turbine)

$E_{generator}$: 0.80 ∼ 0.95 (it's depends on the capacity of generator)

$E_{drive\ system}$: 0.97

E_{line} : 0.90 ∼ 0.98 (it's depends on the transmission length)

$E_{transformer}$: 0.98

$E_{civil\ work}$ and $E_{penstock}$ are usually computed as 'Head Loss (Hloss)'. In this case, the hydropower equation becomes:

$P_{net} = g \times (H_{gross} - H_{loss}) \times Q \times (E_o - E_{civil\ work} - E_{penstock})$ kW(DEUMB, 2009)

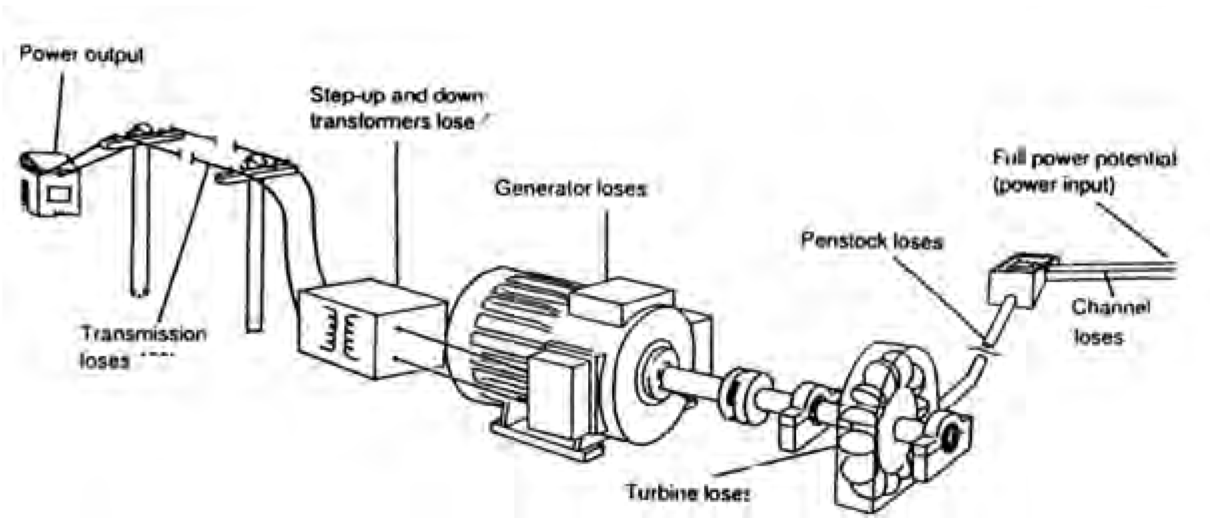

Figure 2.3 different parts of hydro power plant where Loss occurs(DEUMB, 2009)

2.8 Advantages of hydropower

Hydropower is a fueled by water, so it's a clean fuel source. Hydropower doesn't pollute the air like power plants that burn fossil fuels, such as coal or natural gas.

Hydropower relies on the water cycle, which is driven by the sun, thus it's a renewable power source.

Hydropower plants provide benefits in addition to clean electricity. Impoundment hydropower creates reservoirs that offer a variety of recreational opportunities, notably fishing, swimming, and boating. Most hydropower installations are required to provide some public access to the reservoir to allow the public to take advantage of these opportunities. Other benefits may include water supply and flood control. In the case of an institution (U.S. Department of Energy, 2006).

Hydropower offers many advantages over other power sources:

- Hydropower plant use renewable resources.
- Hydropower generators have low outage rates and low maintenance and operating costs.
- Hydro Turbines can provide start up power quickly if a system wide failure occurs.
- They can be operated automatically and by remote control
- Hydropower generators have a long life expectancy.

In particular small and micro hydropower has the following additional advantages:

- ✓ Its suitability for decentralization development, fully using local material and appropriate technology with the participation of the local people.
- ✓ its mature technology and small investment risk;
- ✓ little environmental impact during construction ,with some positive impact onthe environment;
- ✓ the obvious social benefit to a developing local economy and improvements inthe material and spiritual life of local residences. (Tulu, 2007)

2.9 Disadvantages of hydropower

Even though there is a huge hydropower potential in the country and the energy system is hydropower dominated, the amount of available water in reservoirs is greatly seasonal and sometimes heavily affected by shortage of rain which in turn affects the total energy generated from the available plants. This seasonal variation of rainfall and the consequent reservoir level variation, for instance, has been a challenge for the Ethiopian energy system over the years. (Dawit, 2010)

The Ethiopian electric power system has experienced several power shortages, and hence rationing, in the past few years during the dry season while considerable damage was resulted by floods passing reservoirs during wet seasons. The low reservoir level in the country during dry seasons is mainly the result of low water flow into the reservoirs as well as due to the high evaporation in the region. Added to that, a considerable available water variation is recorded in the country due to lack of enough rainfall. As can be seen in the Figure 2.4, the annual stream flow of the country shows a significant variation showing an increase or decrease of up to more than 120% between consecutive years. Consequently, searching for other alternatives that could contentedly supplement the hydropower plants is an advantage to the energy security of the country. Wind, solar and geothermal energies are among the best alternative candidates in the case of Ethiopia.(Amenu&Killingtveit, 2001. as cited in Dawit, 2010)

Hydropower does not discharge pollutants into the environment; however, it is not free from adverse environmental effects. Fish populations can be impacted if fish cannot migrate upstream past impoundment dams to spawning grounds or if they cannot migrate downstream to the ocean.

Upstream fish passage can be aided using fish ladders or elevators, or by trapping and hauling the fish upstream by truck. Downstream fish passage is aided by diverting fish from turbine intakes using screens or racks or even underwater lights and sounds, and by maintaining a minimum spill flow past the turbine.

Hydropower can impact water quality and flow. Hydropower plants can cause low dissolved oxygen levels in the water, a problem that is harmful to riparian (riverbank) habitats and is addressed using various aeration techniques, which oxygenate the water. Maintaining minimum flows of water downstream of a hydropower installation is also critical for the survival of riparian habitats.

Hydropower plants can be impacted by drought. When water is not available, the hydropower plants can't produce electricity. New hydropower facilities impact the local environment and may compete with other uses for the land. Those alternative uses may be more highly valued than electricity generation. Humans, flora, and fauna may lose their natural habitat. In the case of an institution (U.S. Department of Energy, 2006)

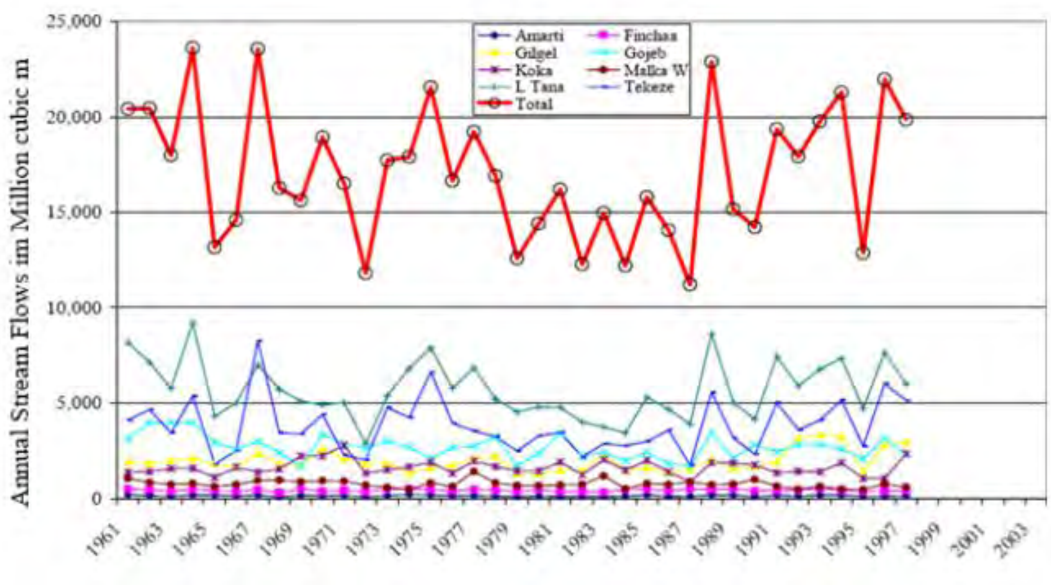

Figure 2.4 High variability of rainfall in Ethiopia (Walelu, 2006, as cited in Dawit)

2.10 Definition of Micro-Hydropower

The definitions of small-hydropower, mini-hydropower and micro-hydropower generations vary among different countries and organizations (zelalem, 1993). Classification of hydropower stations according to power capacity is given in Table 2.3.

Since we don't have a commondefined ranges for Micro-Hydropower here in Ethiopia It is necessary to make appropriate definition of Micro-Hydropower before proceeding to the evaluation of existing sites.

According to (Zelalem, 1993) Micro-Hydropower defined as having generation capacity of less than 100KW. (Keneni 2007; Abebe 2015)defined Micro-Hydropower as having generation capacity less than 500KW. On the other hand the Ethiopian Energy Agency defines Micro Hydropower as having capacity of 11-500KW, mini hydropower as 501-100KW, Pico hydropower as 1-10KW. But MOWR defines MHP to have a capacity of up to 100 KW. For this study the definition given by EEA is used.

Table 2.3 Classification of hydropower according to different country (KW).(TongJiandong et. al,2000.)

No	Name of country	Micro	Mini	Small
1	China	100	101-500	501-25000
2	Peru	5-50.0	51-500	501-5000
3	Philippines			15000
4	Romania			5-5,000
5	Sweden		100	101-15000
6	Thailand	200	201-6000	6001-15000
7	Turkey	100	101-1000	1001-5000
8	USA	500	501-2000	2001-15000
9	Colombia			20000
10	India	100	101-2000	
11	Malaysia	25	25-500	501-5000
12	Nepal			10000
13	Panama	100	101-1000	1001-10000
14	Ecuador	50	51-500	501-5000
15	Bolivia	100	101-1000	
16	Dominica	100	101-1000	1001-5000
17	Vietnam	50	51-500	501-5000
18	Japan			10000
19	France	500	501-2000	2001-8000
20	New Zeeland		10000	10001-50000
21	Indonesia			5000
22	Zimbabwe	5-500	501-5000	
23	Norway			10000
24	Greece	100	101-1000	1001-15000
25	Poland	100	101-1000	1001-15000
26	Finland	200	201-2000	

Small hydro plants are also classified according to the "Head" or the vertical distance through which the water is made to impact the turbines. The usual classifications are given in table 2.4.

Table 2.4 Hydropower classification according to Head(Dilip Singh, 2009.)

Type	Head range
High head	100-m and above
Medium head	30-100m
Low head	2-30m

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Schemes can also be defined as:-

- Diversion and Canal development
- Run-of-river schemes
- Schemes with the powerhouse located at the base of a dam
- Schemes integrated on a canal or in a water supply pipe

Most of the small hydro power plants are "run-of-river" schemes, implying that they do not have any water storage capability. The power is generated only when enough water is available from the river/stream. When the stream/river flow reduces below the design flow value, the generation ceases as the water does not flow through the intake structure into the turbines.

Small hydro plants may be stand-alone systems in isolated areas/sites, but could also be grid connected (either local grids or regional/national grids). The connection to the grid has the advantage of easier control of the electrical system frequency of the electricity, but has the disadvantage of being tripped off the system due to problems outside of the plant operator's control. (Dilip Singh, 2009.)

2.11 Micro-Hydropower in the world

Hydropower is a renewable, non-polluting and environmentally friendly source of energy. Hydropower is based on simple concepts. Moving water turns a turbine, the turbine spins a generator, and electricity is produced. Many other components may be in a system, but it all begins with the energy in the moving water. The use of water falling through a height has been utilized as a source of energy since a long time. It is perhaps the oldest renewable energy technique known to the mankind for mechanical energy conversion as well as electricity generation. In the ancient times waterwheels were used extensively, but it was only at the beginning of the 19th Century with the invention of the hydro turbines that the use of hydropower got popularized.

India has a century old history of hydropower and the beginning was from small hydro.

The first hydro power plant was of 130 kW set up in Darjeeling during 1897, marked the development of hydropower in the country. Similarly, by 1924 Switzerland had nearly7000 small scale hydropower stations in use. Even today, Small hydro is the largest contributor of electricity from renewable energy sources, both at European and world level. With the advancement of technology, and increasing requirement of electricity, the thrust of electricity generation was shifted to large size hydro and thermal power stations. However, it is only during the last two decades that there is a renewed interest in the development of small hydro power (SHP) projects mainly due to its benefits particularly concerning environment and ability to produce power in remote areas. Small hydro projects are economically viable and have relatively short gestation period. The major constraints associated with large hydro projects are usually not encountered in small hydro projects. Renewed interest in the technology of small scale hydropower actually started in China which has more than 85,000 small-scale, electricity producing, hydropower plants(http://practicalaction.org/.)

Hydropower will continue to play important role throughout the 21st Century, in world electricity supply. Hydropower development does have some challenges besides the technical, economic and environmental advantages it shares above other power generation (fossil fuel based) technologies. At the beginning of the new Millennium hydropower provided almost 20% (2600 TWh/year) of the electricity world consumption (12900 TWh/year). It plays a major role in several countries. According to a study of hydropower resources in 175 countries, more than 150 have hydropower resources. For 65 of them, hydro produces more than 50% of electricity; for 24, more than 90% and 10 countries have almost all their electricity requirements met through hydropower refer Table 2.5for worldwide installed small hydropower capacity.(http://www.uniseo.org/hydropower.html).

Table 2.5 installedSHP capacity(<10 MW) by world region (John, 2014; Wim, 2007)

REGION	CAPACITY(MW)	PERCENTAGE
Asia	32,641	68.00%
Europe	10,723	22.30%
North America	2,929	6.10%
South America	1,280	2.70%
Africa	228	0.50%
Australia	198	0.40%
TOTAL	**47,997**	**100%**

2.12 Micro-hydropower in Ethiopia

The topography of Ethiopia is well suited to the use of micro-hydro power. There are mountainous areas with many small rivers that are convenient for development of small and micro-hydro power generation. Limited by seasonality of rainfall, the potential for small-scale hydro power is estimated to be only 10 % of the total potential (1,500 - 3,000 MW). When it comes to Micro and Pico-hydro power, effect of seasonality on availability is very high.

Hence, the total potential for Micro and Pico-hydro power (of size less 500 kW) is estimated to be 100 MW (Muluken, 2014; Melessaw, 2009).So far, out of the total potential for MHPs in the country only a very small portion of it (less than 1 %) is developed. Most of the potential sites are located in the Western and South Western parts of the country with variations from place to place depending on mean annual rainfall received by the area which ranges between 300 mm to over 900 mm for mean annual water surplus amount & location refer Fig 2.5(Melessaw, 2009).

In Ethiopia several MHP schemes were installed in the period between 1950 and 1970, according to (Muluken 2014) during this period EEPCo installed 1.5 MW, but now almost all of them are not operational anymore refer Table 2.5: once the areas were connected to ICS, the MHP plants were shut down. Nonetheless, some of the plants are still in good condition and it would be technically feasible to rehabilitate those (Boli, Feibel, 2008 as cited in Muluken, 2014).

The same holds true for the Yaye MHP in sidama region which was commissioned in 2002, but after only operated for two years it was suffered from low river flow during the 2002/2003 dry season, EEPCo connected Yaye to its ICS and the MHP plant was shut down completely. (Gross, Bolli 2009, as cited in Muluken 2014).

GIZ ECO has implemented four pilot MHP sites in 2011 in the Sidama Zone/SNNPR with a capacity of 7KW(Gobecho I), 30KW (Gobecho II), 33KW (Erete) and 55KW (HageraSodicha) and supports the upgrading of a water mill in Jimma and 10 KW MHP plant in Kersa.

Furthermore, the GTP (2010-2015) includes the installation of 65 MHP plants (Ethio Resource Group2011 as sited in Muluken, 2014). No further details regarding capacity and location are available.

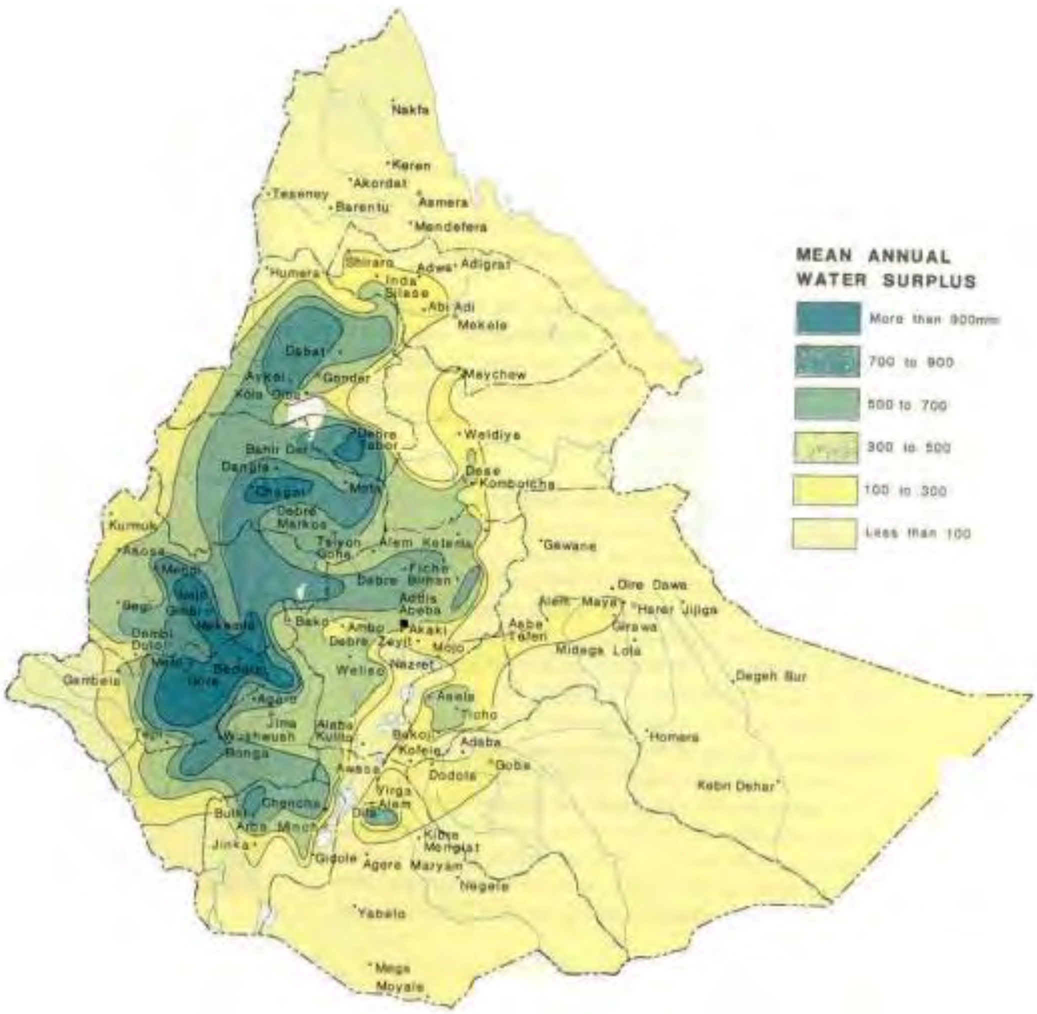

Figure 2.5 Mean Annual water surplus amount &location (Melessaw, 2009)

Table 2.6 Existing EEPCo hydro plants in the micro range (<500kW). (Keneni, 2007).

Name, Location	Head	Type of Scheme	Installed capacity (KW)	Commissioning Year	Current status
Yadot, Bale Zone	23	ROR	350	1991	Operational
Welega, Weliso town	16	ROR	162	1965	Not operational
Sotosomore, Jimma	30	ROR	147	1954	Not operational
Huluka, Ambo town	40	ROR	150	1954	Not operational
Deneba, Bunobedele	14	ROR	123	1967	Not operational
Gelenmite, DenbiDollo	42	ROR	195	1966	Not operational
Chemoga, DebreMarkos	55	ROR	195	1962	Not operational
DerbreBerhan		ROR	130	1955	Not operational
Jibo, Harar Zone		ROR	420		Not operational
Total Capacity		ROR	1872		
Operational		**ROR**	**350**		
Not operational		**ROR**	**1522**		

2.13 Advantage & Disadvantage of MHP

Advantage of micro hydropower

Clean energy source: Hydropower does not produce greenhouse gas emissions, which are the major cause of the international concerns about environmental problems. Hydroelectricity does not involve a process of combustion, therefore it avoids polluting emissions like carbon dioxide (responsible for global warming) that otherwise would be produced by conventional energy when burning fossil fuels. MHP is a clean energy source (it does not produce waste in the rivers, or air pollution) and renewable (the fuel for hydropower is water, which is not consumed in the electricity generation process)

Efficient energy source: It only takes a small amount of flow (as little as two gallons per

minute) or a drop as low as two feet to generate electricity with micro hydro. Since MHP is a decentralized energy source located close to the consumers, transmission losses can be reduced although electricity can be delivered as far as a mile away to the location where it is being used.
Reliable electricity source: Hydro produces a continuous supply of electrical energy in comparison to other small-scale renewable technologies. The peak energy season is during the winter months when large quantities of electricity are required. Power is usually continuously available on demand and the energy available is predictable.

No reservoir required: Micro hydro is considered to function as a 'run-of-river' system, meaning that the water passing through the generator is directed back into the stream with relatively minimal or no impact on the surrounding ecology.

Cost effective energy solution: according to (Dilip Singh, 2009) Building a small-scale hydro-power system can cost from $1,000 - $20,000 USD/kW, depending on site characteristics, power plant size and location. Maintenance costs are relatively small in comparison to other technologies. Given a reasonable head, it is a concentrated energy source. It is a long-lasting and robust technology – the life of systems can be as long as 50 years or more without major new investments (the average life considered for investment purposes however is about 30 years).
Power for developing countries: Because of the low-cost versatility and longevity of micro hydro, developing countries can manufacture and implement the technology to help supply much needed electricity to small communities and remote villages. No fuel and limited maintenance are required, so running costs are low (compared with diesel power). Localized power can be utilized for the benefit of the local economy. (Dilip Singh, 2009)

Disadvantage of micro hydropower
Site specific technology: In order to take full advantage of the electrical potential of small streams, a suitable site is needed. Factors to consider are: distance from the power source to the location where energy is required (this is not very common to find), stream size (including flow rate, output and drop), and a balance of system components: - inverter, batteries, controller, transmission line and pipelines.

Energy expansion not possible: There is always a maximum useful power output (size and flow from small streams for example) available from a given hydropower site, which limits the increase in power generation and the level of expansion of activities which can make use of the power.

Seasonal variations: In many locations the flow in a stream fluctuates seasonally and this can limit the firm power output to quite a small fraction of the possible peak output. During summer months there is likely to be less flow and therefore less power output. Advanced planning and investigations are needed to ensure adequate energy generation and power demands are met.

Environmental and ecological concerns: MHP, like any energy-production activity, has impacts on the local ecosystem (on the quality of river and river ecosystems, noise, landscape). However, new legislative frameworks, innovative technology, improved methods of operating MHP and above all the willingness of all actors to integrate environmental concerns are steadily reducing these local environmental impacts. MHP plants, if well equipped, with fish ladders and environmentally friendly runner blades, are not an obstacle even for fish migration. (Dilip Singh, 2009)

2.14 Classification Of micro-hydropower by facility type

Hydropower plants are often classified in four main categories according to operation and type of flow. Diversion & canal, Run-of-river (RoR), storage (reservoir) and pumped storage HPPs all vary from the very small to the very large scale, depending on the hydrology and topography of the watershed.

Diversion & canal

A Diversion & canal HPP draws the energy for electricity production mainly from the available flow of the river. Such a hydropower plant may include some short-term storage (hourly, daily), allowing for some adaptations to the demand profile, but the generation profile will to varying degrees be dictated by local river flow conditions. As a result, generation depends on precipitation and runoff and may have substantial daily, monthly or seasonal variations. When even short-term storage is not included, Diversion & canal HPPs will have generation profiles that are even more variable, especially when situated in small rivers or streams that experience widely varying flows.

In a Diversion & canal HPP, a portion of the river water is diverted to a channel or pipeline (penstock) to convey the water to a hydraulic turbine, which is connected to an electricity generator (Figure 2.6). Installation of Diversion& canal HPPs is relatively inexpensive and such facilities have, in general, lower environmental impacts than similar-sized storage hydropower plants.(Kumar et al 2011)

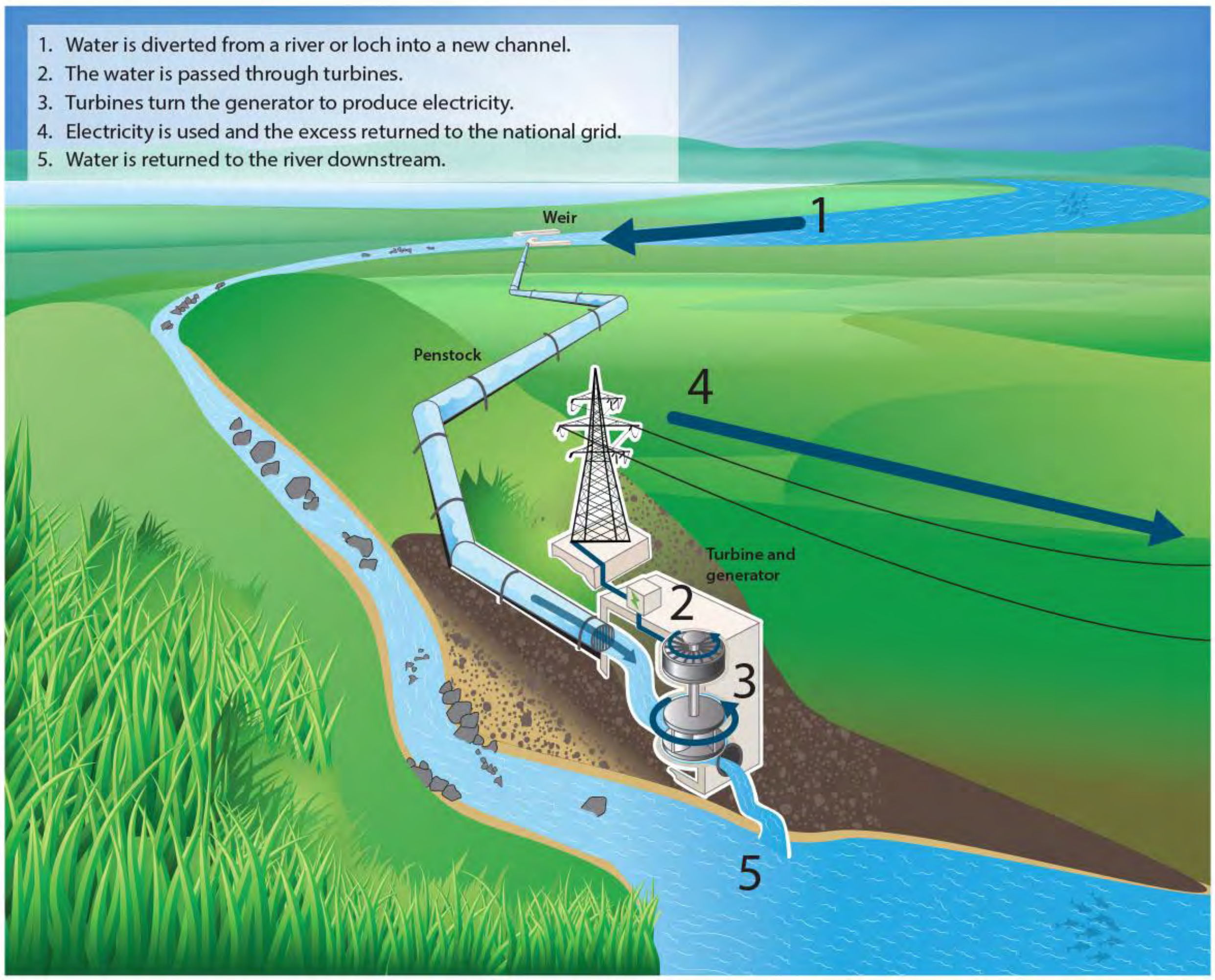

Figure 2.6Diversion & canal hydropower plant (online)

Run-off-River

In ROR hydropower the normal flow of the river is not disturbed, as there is no significant storage in the river system. In order to get head a weir or barrage is built across the river and the low head created is used to generate power. The power house for ROR hydropower is in the main course of a river(Fig 2.7).

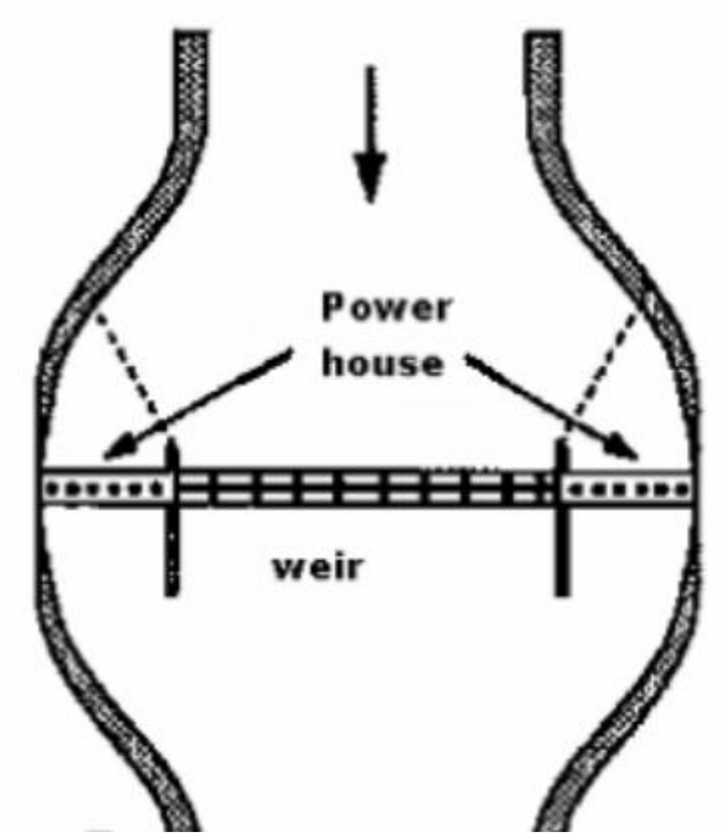

Fig 2.7 ROR hydro plant

Storage Hydropower

Hydropower projects with a reservoir are also called storage hydropower since they store water for later consumption. The reservoir reduces the dependence on the variability of inflow. The generating stations are located at the dam toe or further downstream, connected to the reservoir through tunnels or pipelines (Figure 2.8). The type and design of reservoirs are decided by the landscape and in many parts of the world are inundated river valleys where the reservoir is an artificial lake. In geographies with mountain plateaus, high-altitude lakes make up another kind of reservoir that often will retain many of the properties of the original lake. In these types of settings, the generating station is often connected to the lake serving as reservoir via tunnels coming up beneath the lake (lake tapping) (Kumar et al 2011).

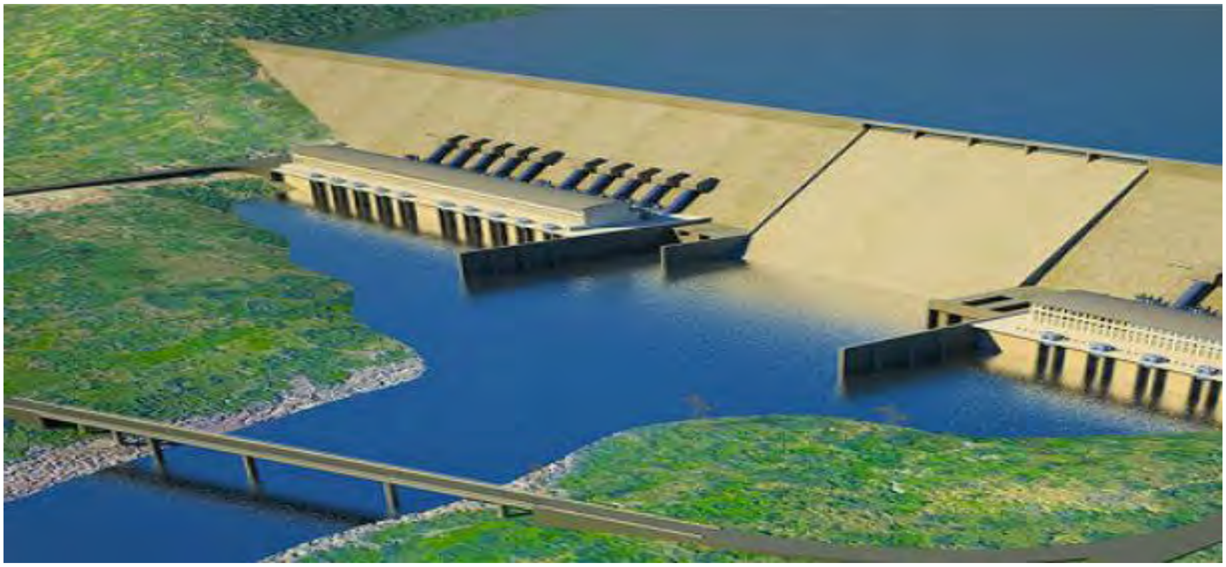

Figure 2.8Storage hydropower (http://nazret.com)

Pumped storage

Like peaking, pumped storage is a method of keeping water in reserve for peak period power demands. Pumped storage is water pumped to a storage pool above the power plant at a time when customer demand for energy is low, such as during the middle of the night. The water is then allowed to flow back through the turbine-generators at times when demand is high and a heavy load is place on the system Refer Fig 2.9.

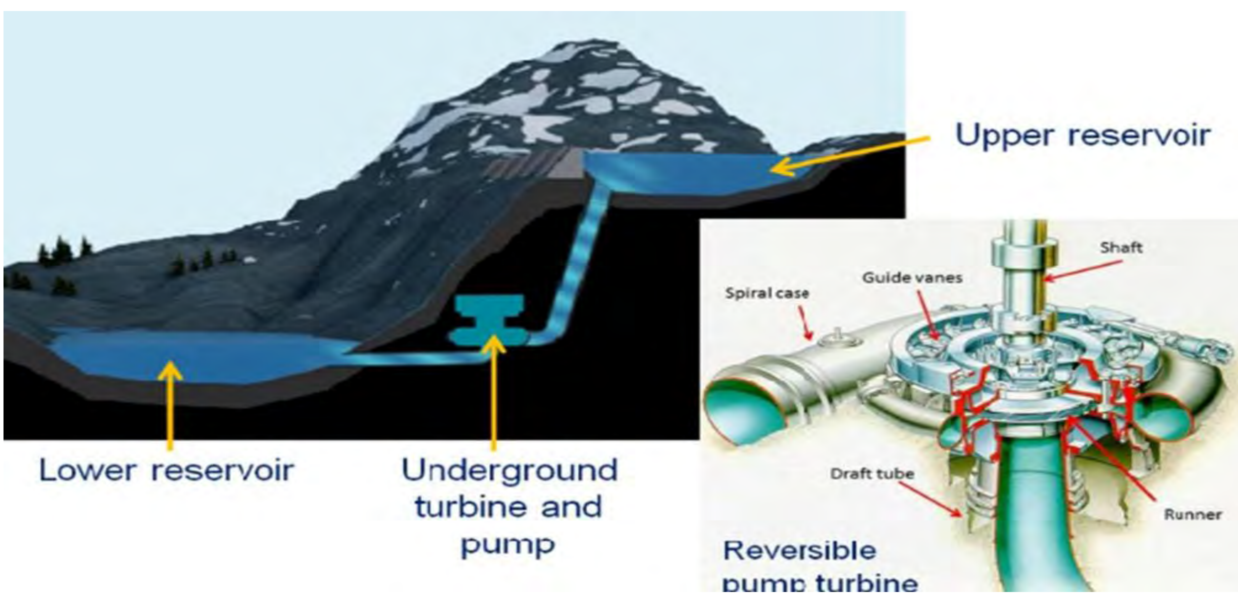

Figure 2.9 Pumped storage hydropower plant(http:/www.nortrade.com)

The reservoir acts much like a battery, storing power in the form of water when demands are low and producing maximum power during daily and seasonal peak periods. An advantage of pumped storage is that hydroelectric generating units are able to start up quickly and make rapid adjustments in output. They operate efficiently when used for one hour or several hours, because pumped storage reservoirs are relatively small, construction costs are generally low compared with conventional hydropower facilities. In the case of an institution (U.S. Department of the interior Bureau of Reclamation power Resources office, 2005)

2.15 Components Of Diversion & canal micro-hydropower plants

Figure 2.10below shows the major components of typical micro hydropower scheme. The water in the river is diverted by the weir through an opening in the river side (the 'intake') into a channel (this could be open or buried depending upon the site conditions). A settling basin is built in to the channel to remove sand and silt from the water. The channel follows the contour of the area so as to preserve the elevation of the diverted water. The channel directs the water into a small reservoir/tank known as the 'forebay' from where it is directed on to the turbines through a closed pipe known as the 'penstock'. The penstock essentially directs the water in a uniform stream on to the turbine at a lower level. The turning shaft of the turbine can be used to rotate a mechanical device (such as a grinding mill, oil expeller, wood lathe, etc.) directly, or to operate an electricity generator.

The machinery or appliances which are energized by the turbine (or MHP) are called the 'load'. When electricity is generated, the 'power house' where the generator is located transfers the electricity to a step-up 'transformer' which is then transmitted to the grid sub-station or to the village/area where this electricity is to be used.

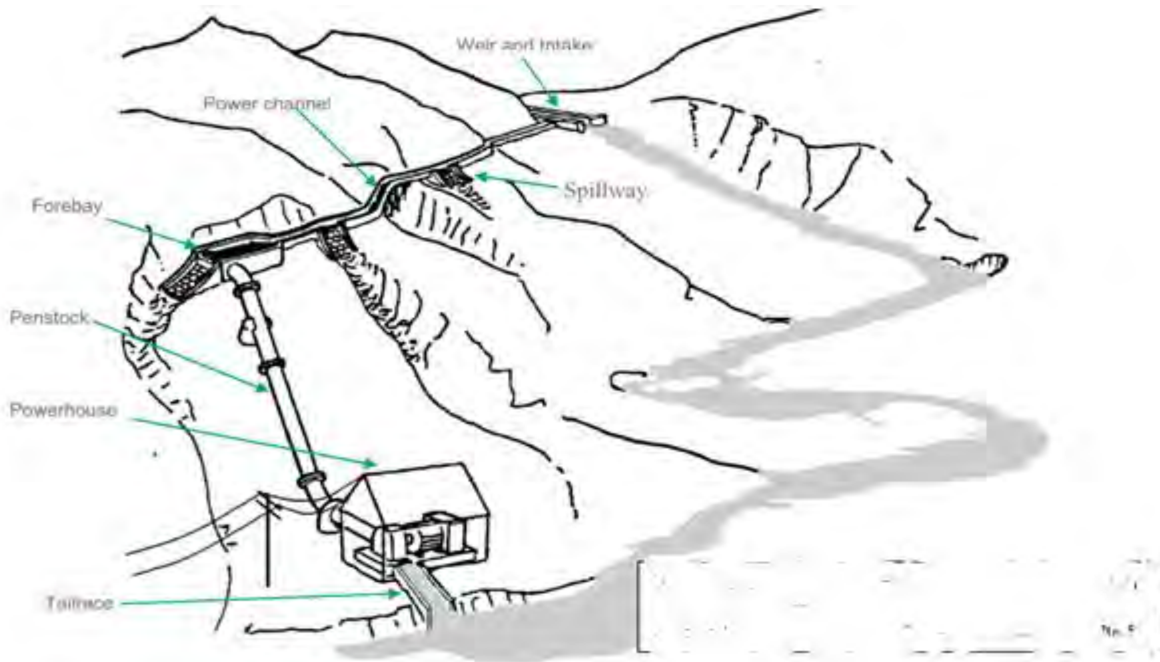

Figure 2.10 Schematic of a typical Diversion &canal hydro power plant (Dilip Singh, 2009)

2.15.1 Civil works

(Dilip Singh, 2009) stated that a micro hydropower station essentially needs water to be diverted from the stream and brought to the turbines without losing the elevation/head. Given below are some of the important factors that must be kept in mind while designing a micro hydropower system:

Available head: The design of the system has effects on the net head delivered to the turbine. Components such as the channel and penstock cannot be perfectly efficient. Inefficiencies appear as losses of useful head of pressure.

Flow variations: The river flow varies during the year but the hydro installation is designed for almost a constant flow. If the channel overflows there will be serious damage to the surroundings. The weir and intake must therefore be designed for such eventualities and divert only the required amount of flow irrespective of whether the river is in low or in high flow. The main function of the weir is to ensure that a constant flow in the channel is maintained when there is less flow in the river. The intake structure is designed to regulate the flow to within reasonable limits when the river is in high flow. Further regulation of the channel flow is provided by the spillways. **Sediment:** Flowing water in the river sometimes carry small particles of hard abrasive matter (sediment) which can cause wear to the turbine if they are not removed before the water enters the penstock. Sediment may also block the intake or cause the channel to clog up if adequate precautions are not taken.

Floods: Flood water will carry larger suspended particles and will even cause large stones to roll along the stream bed. Unless careful design principles are applied, the diversion weir, the intake structure and the embankment walls of the river may be damaged.

Turbulence: In all parts of the water supply line, including the weir, the intake and the channel, sudden alterations to the flow direction will create turbulence which erodes structures and causes energy losses.

Most common civil structures used in a MHP scheme are:

- Weir and intake

  A micro hydropower system necessitates that water from the river to be diverted and extracted in a reliable and controllable manner. The water flowing in the channel must be regulated during high river flow and low flow conditions. A weir can be used to raise the water level and ensure a constant supply to the intake. Sometimes it is possible to avoid building a weir by using natural features of the river. A permanent pool in river could also act as a weir. Another condition in site selection of the weir is to protect it from damage. Sometimes, in remote hilly regions, where annual flooding is common it may be prudent to build temporary weir using local resources and manpower. The temporary weir is a simple structure at low cost using local labor, skills and materials. It is expected to be destroyed by annual or bi-annual flooding. However, advanced planning has to be done for rebuilding of the weir. The intake of a MHP is designed to divert only a portion of the stream flow or the complete flow – depending upon the flow conditions and the requirement. MHP schemes use different types of intakes distinguished by the method used to divert the water into the intake. For MHP schemes, intake systems are smaller and simpler. (Abebe, 2011)

- Power canal

  The power channel or simply a channel conducts the water from the intake to the forebay tank. The length of a channel depends upon the topography of the region and the distance of powerhouse from the intake. Also the designing of the MHP systems states the length of the channel – sometimes a long channel combined with a short penstock can be cheaper or required, while in other cases a combination of short channel with long penstock would be more suitable.(Dilip Singh, 2009)

- Settling basin

  The water diverted from the stream and carried by the channel usually carries a suspension of small particles such as sand that are hard and abrasive and can cause expensive damage and rapid wear to turbine runners. To get rid of such particles and sediments, the water flow is allowed to slow down in 'settling basins' so that the sand and silt particles settle on the basin floor. The deposits are then periodically flushed. The design of settling basin depends upon the flow quantity, speed of flow and the tolerance level of the turbine (smallest particle that can be allowed). The maximum speed of the water in the settling basin can thus be calculated as slower the flow, lower is the carrying capacity of the water. The flow speed in the settling basin can be lowered by increasing the cross section area.(Dilip Singh, 2009)

- Spillways

  Spillways along the power channel are designed to permit overflow at certain points along the channel. The spillway acts as a flow regulator for the channel. During floods the water flow through the intake can be twice the normal channel flow, so the spillway must be large enough to divert this excess flow. The spillway can also be designed with control gates to empty the channel. The spillway should be designed in such a manner that the excess flow is fed back to the without damaging the foundations of the channel.

- Forebay tank

  According to (Zelalem, 1993) A forebay is constructed at the end of the headrace, in front of the penstock, in order to maintain a constant head on the turbine and to store surplus water. Usually the forebay is a small basin, but it should store enough water to maintain two or three minutes at maximum discharge without resupply in order to cope with sudden variations in load, to which headrace flow could not respond well.

  Forebay can also be a reservoir to store water –depending on its size (large dams or reservoirs in large hydropower schemes are technically forebay). A sluice will make it possible to close the entrance to the penstock. In front of the penstock a trash rack need to be installed to prevent large particles to enter the penstock. A spillway completes the forebay tank.

2.15.2 Penstock

The penstocks are pipes of large diameter, usually steel or concrete used for covering water from the source (reservoir or fore bay) to the power house. They are usually high pressure pipe lines designed to with stand stresses developed because of static and water hammer pressures created by sudden change in power demand (i.e. value closure and openings according to power rejection and demand). The provision of such a high pressure line is very uneconomical if it is too long in which case it can be divided into two parts, along low pressure conveyance (tunnel) followed by short high pressure pipe line (penstock) close to the turbine unit, separated by a surge chamber which absorbs the water hammer pressure rises and converts them into mass oscillations. (Pandey B., 2006. As cited in Murad, 2015.)
The other issue that should be raised is the Classification of penstock. Commonly classification of penstocks depends on the basis of the material of construction, Method of support, Rigidity of connection & support, & Number of penstocks.

2.15.3 Turbines

While there are only two basic types of turbines (impulse and reaction), there are many variations. The specific type of turbine to be used in a power plant is not selected until all operational studies and cost estimates are complete. The turbine selected depends largely on the site conditions. A reaction turbine is a horizontal or vertical wheel that operates with the wheel completely submerged a feature which reduces turbulence. In theory, the reaction turbine works like a rotating lawn sprinkler where water at a central point is under pressure and escapes from the ends of the blades, causing rotation. Reaction turbines are the type most widely used.

An impulse turbine is a horizontal or vertical wheel that uses the kinetic energy of water striking its buckets or blades to cause rotation. The wheel is covered by a housing and the buckets or blades are shaped so they turn the flow of water about 170 degrees inside the housing. After turning the blades or buckets, the water falls to the bottom of the wheel housing and flows out.in the case of an institution (U.S. Department of the interior Bureau of Reclamation power Resources office, 2005)Beyond the operating, turbines canal so be classified as high head, medium head or low head machines. The basic turbine classification is given in Table 2.7:

Table 2.7 Turbine classification(*Dilip Singh September 2009)*

	High head	Medium head	Low head
Impulse turbine	**PeltonTurgo**	**Cross-flow Multi-jet PeltonTurgo**	**cross-flow**
Reaction turbines		**Francis**	**Propeller Kaplan**

2.16Micro Hydropower Cost

Determining the capital cost of a MHP plant is an important part of the economic analysis procedure. One of the biggest challenges facing investment analysis of hydropower project in developing countries is the absence of a quick & reliable basis for estimating capital costs. The high variability of KW & KWH prices of small hydropower systems worldwide prevent realistic cost estimation for small hydropower plants under Ethiopian conditions where almost no experience in this field is available. (Zelalem, 1992)

According to (Graham et al. 2010)it is difficult to give guidance on the estimated costs of micro hydro schemes as hydro power, unlike other forms of renewable energy, is very site specific and hence costs vary enormously from site to site. However, the costs can usually be reduced considerably if the project is incorporated with existing irrigation diversion structures, where infrastructure, such as the weir, is still in existence and in good condition.

For this study I do not make a detail economic analysis, but just an over view of the overall cost that is required for MHP development. The following are general work items that take part on the construction of MHP plant.

The costs of a scheme can be divided into four parts:

1 Civil works:-being all physical/engineering works concerned with the abstraction and return of the water, as well as any building costs to house the machinery.

2 Machinery:-being all the machinery (plant) necessary to convert water power into electrical power from a turbine, waterwheel etc. to the generator. This may also include screens, where needed.

3 Electrical works:-which include the control panel and control system, wiring and grid connection and metering if required.

4 External costs:-being the cost of a consultant to design and manage the project plus costs of licenses, planning permission etc.

According to (Zelalem, 1993), assuming 7.5% contingency of the total cost the grand sum of installation cost after making all necessary increments is 83,900 birr or 4,400 Birr/kw. And according to (Abebe2015) the reasonable unit cost of investment for MHP is US$2000/KW. The costs of MH plants for electricity generation schemes ranged from USD 1136 (Pucara-peru) to USD 5630(pedro Ruiz-peru) with an average installed cost of USD 3085 (Small Khennas and Andrew Barnett-2000 as cited in Tamene 2004). And according to (Graham and Andy, 2010),generally, the cost per kilowatt of installed power increases as the project size reduces due to certain fixed costs and economies of scale. A number of estimates of capital cost have been given which fall in the range of £1,000 to £4,500 per installed kilowatt subject to the amount of civil engineering required.

2.17 Identification of potential site

Potential sites are identified on the topographical map with a scale of 1/50,000 by interpreting the head. The following parameters should be considered in the map study:

(1) Site identification considering river gradient and catchment area

Sites with high head, shortest waterway and high discharge level are naturally advantageous for hydropower generation. The information on the river gradient (elevation difference and river length) and the drainage area could be obtained in the map study.

(2) Identification based on waterway construction conditions

As far as the basic layout of a micro-hydro scheme is concerned, most civil structures are planned to have an exposed structure. Because of this, the topography at any potential site must be able to accommodate such exposed civil structures.

(3) Identification Based on Local Information

In cases where potential sites cannot be interpreted on the topographical map because of the small usable head or the presence of a fall or pool, etc. as well as existing infrastructures like intake facilities for irrigation and forest roads, potential sites are identified on the basis of information provided by a local public body and/or local residents' organization. (DEUMB, 2009)

Chapter 3

3. Materials &Methodology

3.1General

The quality and quantity of input data largely determines the performance of any hydrological model (Prabin, 2014). The model used in this research is Hydrological Engineering Center-Hydrologic Modeling System (HEC-HMS). The basic data required to develop a hydrological model in HEC-HMS are climatic datasets, land cover, soil types and topography. For this study generally I need three basic data's which are the stream flow data, rainfall data, & the map. The details of data used & methodologies are discussed in this chapter.

Some of organization and Bureaus that have been contacted and cooperated in providing the necessary data's are:

- Ministry of Water, Irrigation & Energy
- Ethiopian Mapping Agency &
- Ethiopian National Meteorological Service Agency

3.2 Description of the study area

3.2.1 General

Ethiopia has 12 Basins, 8 of which are River Basins, 1 Lakes Basin and the remaining 3 Dry, with no or insignificant flow out of the drainage system. The dome shaped nature of the country, with high raising mountains and a high tableland at the center that descends in all direction and the surrounding lowlands that circumscribe the plateaus is a peculiar nature of Ethiopia. The Ethiopian Rift Valley that dissected the highland plateau from North-east to Southwest which tilts from the center in the North-easterly and South-westerly direction is also responsible for creation of the Lakes Basin and determining the direction of flow of some rivers that terminate in the Rift-Valley System (http://www.mowr.gov.et/).

For this research work, the Adaba sub catchment in upper wabe shebele river basin was selected(Fig 3.1). This is a region where there are around five perennial rivers are found, before converged to one.

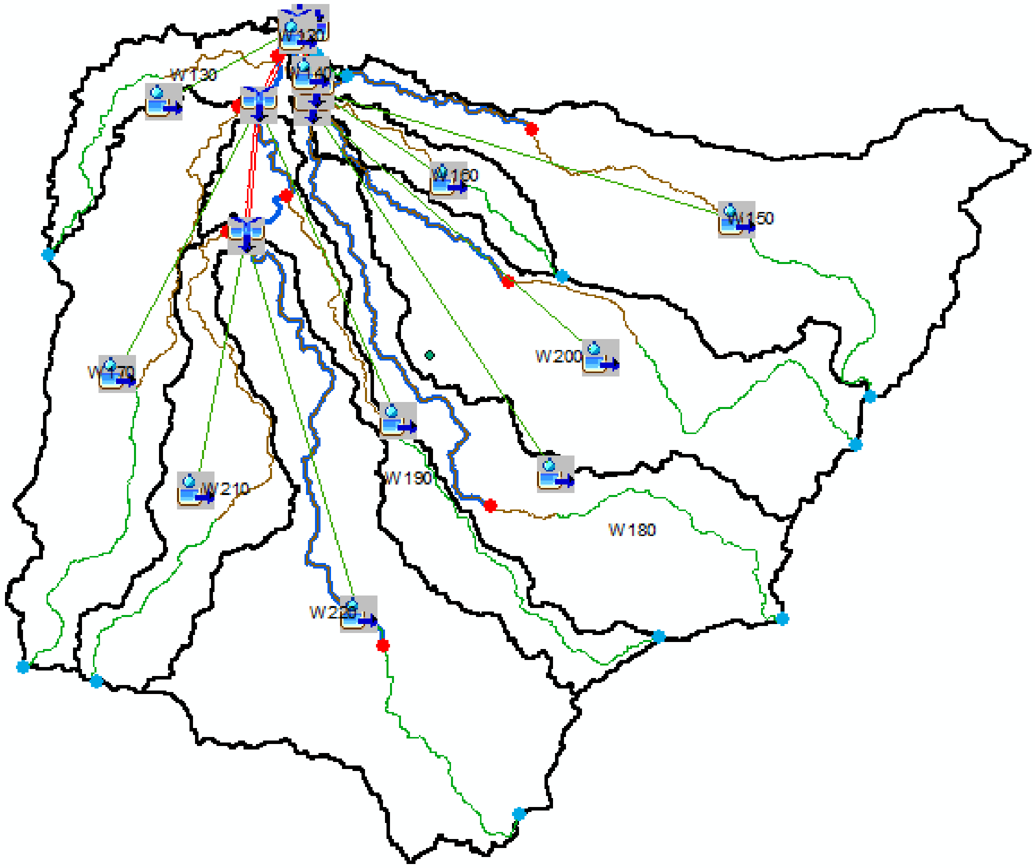

Figure 3.1 Adaba sub cathcment

3.2.2 Climate

3.2.2.1 Rainfall regimes & seasons in Ethiopia

In tropics, rainfall is an essential element in delineating homogeneous climatic behavior of a region. NMSA(1996) gives four rainfall regimes in Ethiopia: mono-modal, bimodal type I, bimodal type II, and diffused pattern(figure1).

Mono-modal- The area designated as region B on Figure 3.2 is dominated by a single maximum rainfall pattern with wet periods decreasing as we go northwards. This region is subdivided into 3 parts designated as B1, B2 and B3 where the wet period runs from February/March to October/November, April/May to October/November and from June/July to August/September respectively..

Bimodal type I – The area designated as region A on Figure 3.2 is characterized by a quasi-double maximum rainfalls pattern with a small peak in April and maximum peak in August. Therefore the region is dominated by semi-bimodal rainfall pattern.

Bimodal typeII- The area identified as region C on Figure 3.2 is dominated by double maximum rainfall pattern with peaks during April and October. Generally, the annual rainfall decreases from west to east. Region C is characterized by a bimodal rainfall pattern.

Diffused pattern- the area designated by D on Figure 3.2 is characterized by irregular rainfall pattern. Though erratic rainfall prevails from August/ September to January/ February the region does not have a well-defined rainfall pattern.

Based on the above rainfall patterns, generally the following seasons have been defined (NMSA, 1996. As cited in MoWR, 2005).

(a)Kiremt- the main rainy season that covers the period from June to September. The air flow during this season is dominated by a zone of convergence in low pressure systems accompanied by the oscillatory inter tropical convergence zone (ITCZ) extending from West Africa through north of Ethiopia towards India.

(b)Bega- generally dry season that covers the period from October to January. However there is occasionally untimely rain.

(c)Belg-a small rainy season that covers the period from mid-February to mid-May. However the rainfall is highly characterized by inter-annual and inter-seasonal variations.

According to the wabi shebele master plan study the Adaba region receive most of the rainfall during July, August, and September associated with the north ward passage of the ITCZ. From September to November, the ITCZ moves back to southward direction, causing a rapid end to the rainy season during September-October. By December and January the ITCZ moves further south wards in to Kenya.

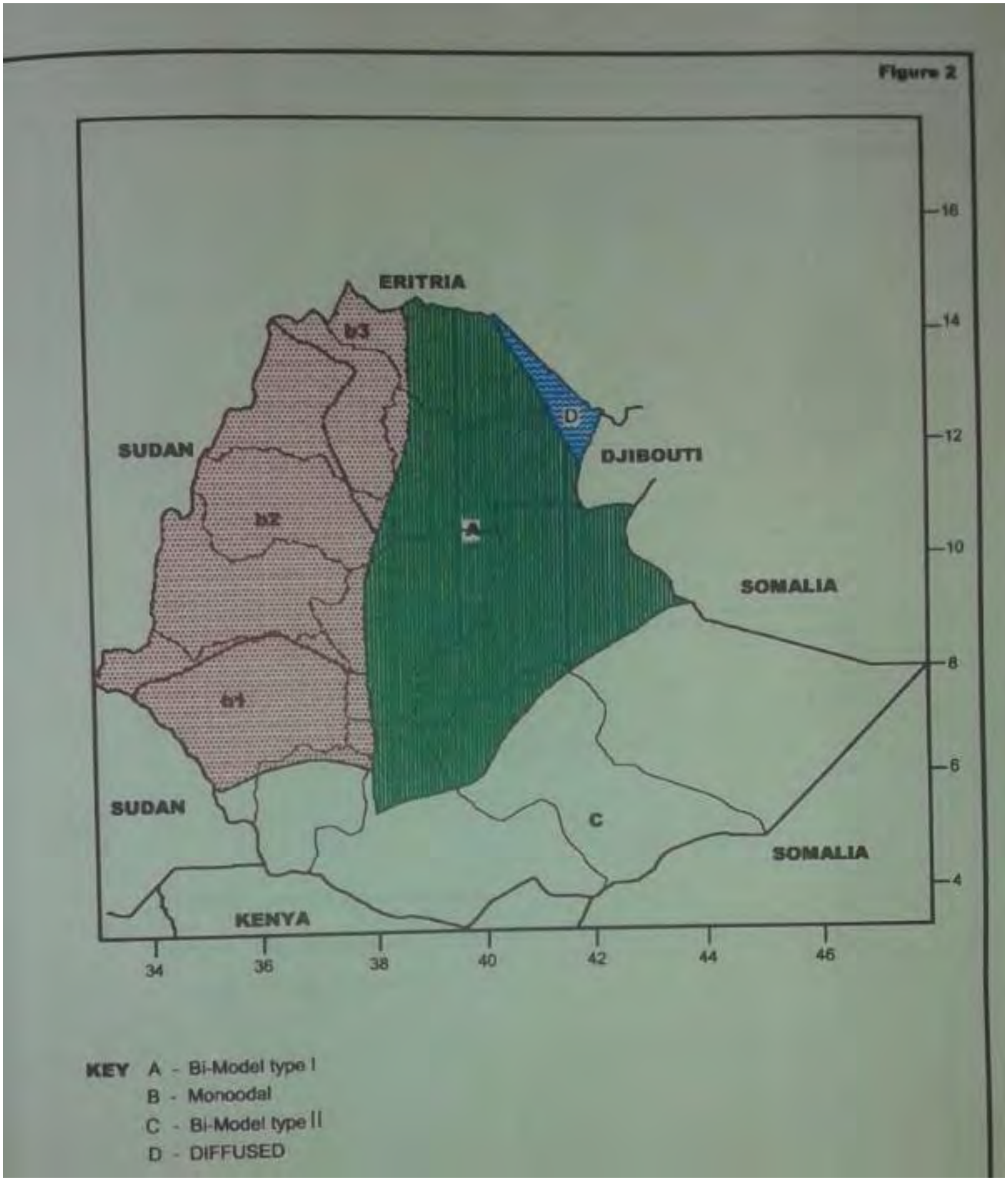

Figure3.2Rainfall regimes in Ethiopia.(MoWR, 2005.)

3.2.2.2 Rainfall

Mean annual rainfall of the basin is 425 mm varying from 900-1300 mm in the west-north-western (upper) part of the basin covering 15% area, 500-900 mm in the middle part covering 25% area and 150-500 mm in the balance 60% of the basin area in the lower valley. The mean monthly rainfall varies from 1563 mm in the west to 223 mm in the east of the basin. On the other hand The mean annual rainfall in Adaba is 797mm.typical distribution of monthly rainfall in upper and lower valley is shown in Fig. 3.3 & 3.4 respectively.

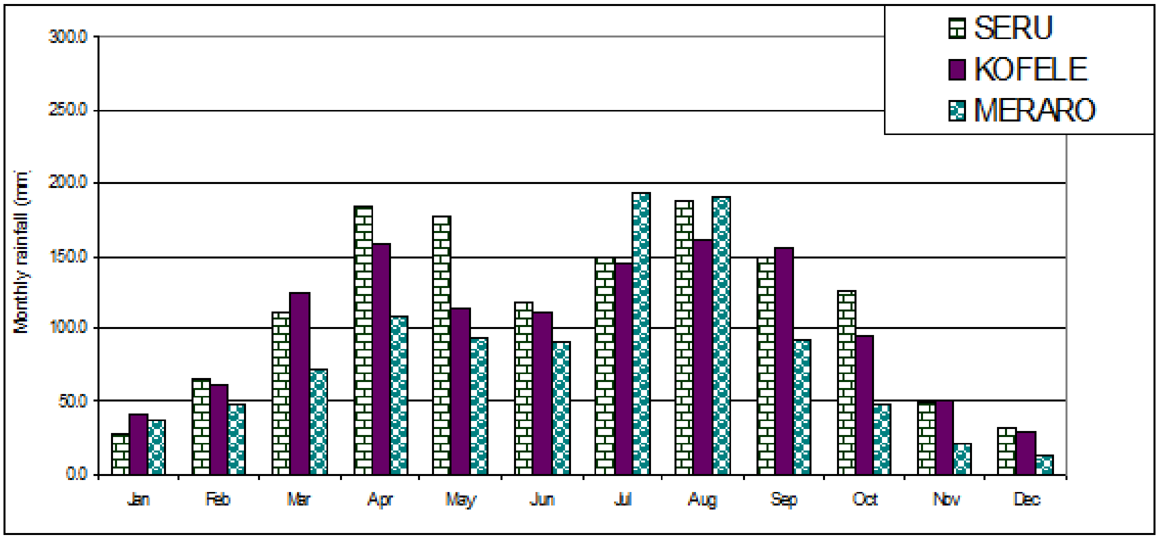

Figure 3.3 Typical monthly rainfall distributions in the upper valley **(MoWR, 2005)**

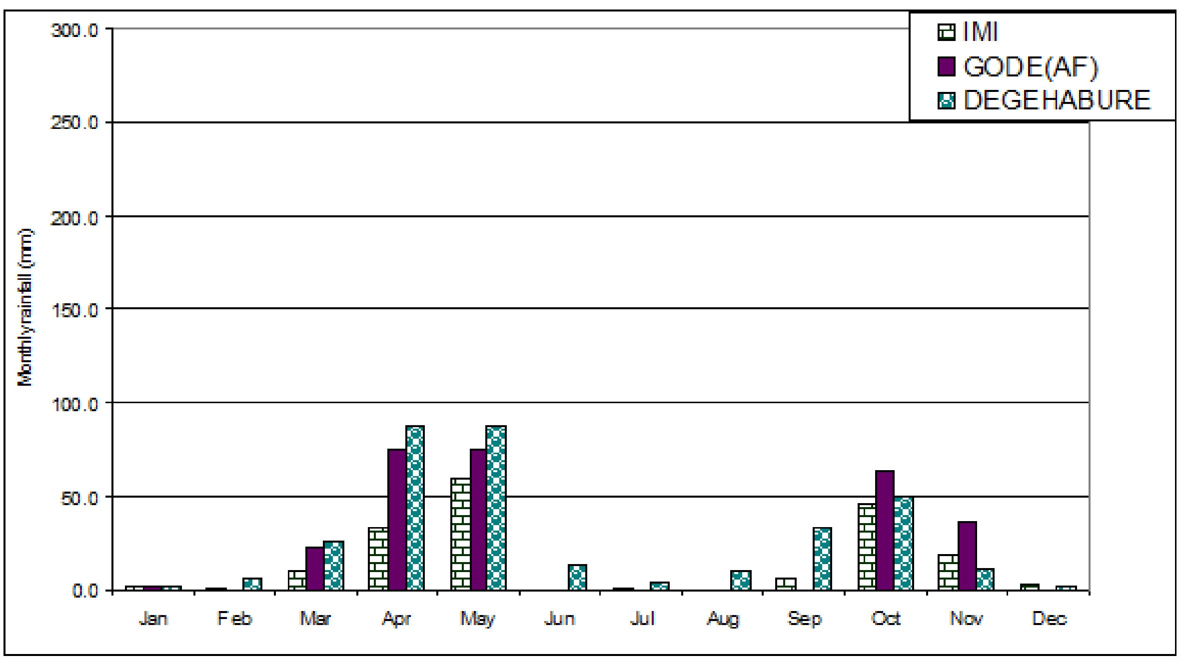

Figure 3.4 Typical monthly rainfall distributions in the lower valley.(MoWR, 2005)

3.2.2.3Agro-climatic zones

Eleven agro-climatic zones have been identified in Ethiopia (SCRP, 1986). The major climatic factors for classification are mean annual rainfall and altitude. Regardless of the amount of annual rainfall that a site receives, it is called Bereha (below 500m asl), Kolla (if it is 500- 1500 m asl), Weina-Dega (if it is in 1500- 2,300 m asl), Dega (if it is in 2,300- 3,200m asl), Wurch (if it is in 3200 m -3700 m asl) or high-Wurch (if it is > 3700 m asl). According to the above classification Adaba sub catchment is under DegaAgro-climatic zone. Because it's minimum elevation is 2347m & the highest elevation is 3025. (MoWR, 2005)

3.2.2.4Major Landforms in the Basin

The level lands are existing in all the development zones with coverage of minimum 32% to maximum 58% in Development Zones(DZ) 8 and 7 respectively(Fig 3.5). The sloping lands are existing in all DZ but minimum in DZ5 with coverage of 5.4%. The steep land covers maximum area in DZ1, DZ2, and DZ3 which covers Arsi, Bale highlands and East and West Harage zones. In development zones 6,7 and 8 its coverage is insignificant. The composite landforms are maximum in DZ 8, DZ6 DZ4 and DZ5 ranging from 61.7 to 24.81 %. (MoWR, 2005)

3.2.2.5 Land use Land cover

- Land cover

The dominant land cover types are intensively cultivated and moderately cultivated lands. These areas are dominating in Arsi-Bale, East & West Hararge of Chercher highlands and around Jijiga in DZ 1, 2, 3 and 5.In the basin the forest cover has shrunk to the highlands parts only. The area under open and dense forest cover only exists in the Adaba, Dodola, Agarfa, Tena, Chiro, Darolebu, Meta, Sawena and GaseranaGololcha i.e. in DZ 1, 2, 3 and 4. In the arid and semi-arid lowland areas the dominant land covers are open grasslands, open and dense shrub lands and open woodlands. On the extreme where the climate is arid, salt flats, exposed rock or sand surface are the predominant land covers. The wide occurrence of shrub and grasslands are associated with the extensive pastoral areas.(MoWR, 2005)

➢ Land use

The list on Table 3.1 shows the land use type, development zone, their respective area cover & the areal percentage of the land type.

Table 3.1 Land use catagories in the wabishebele river basin.(MoWR, 2005)

Landuse Type	Mapping unit	Dev. Zone	Ha	Km²	%
Agricultural-1	A1	1	1873045	18730.45	9%
Agricultural-2	A2	2,3,4,5	477807.43	4778.07	2%
Agro-Silvicultural	AS	1,2 and 3	254545.21	2545.45	1%
Open water	OW	1 and 3	3156.86	31.57	0%
Pastoral	P	5,6,7,8	6032062.79	60320.62	29.40%
Silvi-Pastoral	S	6,7 and 8	10636176.23	106361.76	53%
Silvi-Cultural	SP	1	183321.53	1833.22	1%
Unproductive	N	3,4,6 and 8	761886.41	7618.86	3.60%
Total			**20222000.45**	**202220**	**100%**

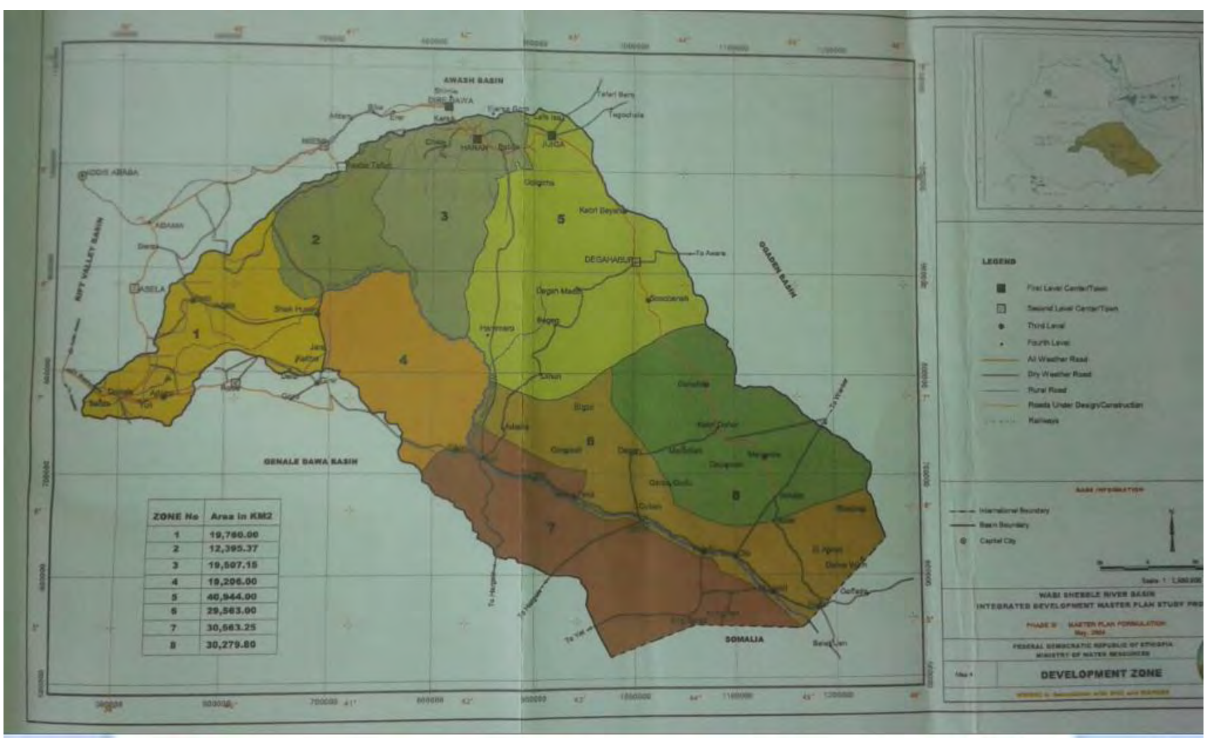

Figure 3.5 Development zone in wabishebele river basin (MOWE, 2005)

3.2.3 Location

3.2.3.1 Wabishebele

The Wabi Shebele River Basin lies between 4°45' and 9°45' North Latitude and 38°45' and 45°45' East Longitude.The basin is located in the southwestern part of the country and bounded by Genale-Dawa basin on the southwest, Rift Valley basin on the west and northwest, AyshaDewala basin on the northeast and Ogaden basin on the east. The size of the basin is 202,220 km^2 and it covers parts of Oromiya Regional State, substantial parts of the Somali Regional State, the entire Harari National Regional State, and a small portion of SNNP region**.**
Area wise, Wabi Shebele is the largest river basin among 12 basins in Ethiopia. It accounts for about 19% of the country's land area and 11 % of the country's population. However The Wabi Shebele Basin receives relatively low mean annual rainfall of about 425 mm as compared to most of Ethiopian basins, which calls for optimal and judicious use of this scarce resource.

3.2.3.2 Adaba sub catchment

The total area of this sub catchment is 1034km², According to the CSA projection for the year 2016, the total number of population in Ababa is 176,229 out of this 18,620(10.5%) lives in urban Area & 157,609 (89.5%) lives in rural Area. This catchments has 5 gauging & 4metrological stations located atthe center of the catchment; This sub catchment is located in the upper part of wabishebele river basin (Fig 3.6).

Around five runoff gauging stations were located in this sub catchment, Furuna gauging station, Lelliso gauging station,Wabi below the bridge gauging station, MeriboAdaba gauging station and Meribochanguity gauging station. Three precipitation station known as Adaba, Hunte, &Meskeldarkina rainfall station are also located in this sub catchment. The four runoff gauging stations can be described one by one as below.

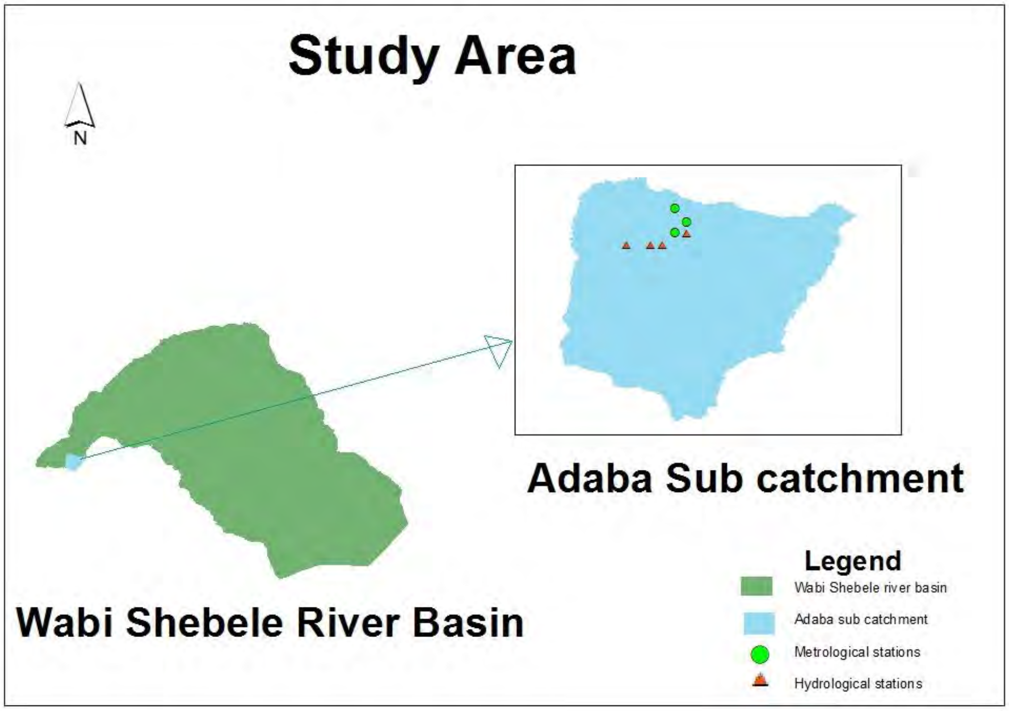

Figure 3.6 Study Area

- Lelliso gauging station

The discharge of Lelliso gauging station was recorded since 1984. The River(Fig 3.7) draining to this gauging station starts at the top most of the Adaba sub catchment and after different intermittent stream joining it; this stream converges to MeriboRiver. The geographical location of this gauging station is at latitude of 7:0:0 N and 39:23:0E longitude. Catchment included by this stream gauging station is estimated to be around 135 sqkm. The high flow period of this gauging station run from July to October while the low flow period will extend from November to June. (Musa, 2015)

Figure 3.7LelisoRiver

- Furuna gauging station

This river is located to the east of the the Adaba town. Furuna River(Fig 3.8) originates from witicho intermittent river and stretch down to the different stream and join Meribo River. The catchment area covered by this river is around 7.5 sqkm, which is the smallest sub catchment of the study area. Its gauging station record starts since 1989.

Figure 3.8FurunaRiver

- Meribo gaging stations

This is the largest flow contributing gauging station and it includes two stations. One of these stations is known as MeriboAdaba gauging station while the second one is called MeriboChanguity gauging staion. MeriboRiver(Fig 3.9)starts at Kaka intermittent river in Adaba sub catchment and also Aligebero intermittent river in Dodola sub catchment and then stretch to the downstream inAdaba sub catchment to join major river after a number of river join it, including both Furuna and Lelliso river discussed previously. The area covered by Meribo Adaba gauging station is 185 sqkm. The geographic location of this station is 7:0:0N and 39:20:0E.The record of both of the station starts since 1984.

Fig 3.9Meribo River

- Herero gaging station

The discharge of Herero gauging station was recorded since 1989. The river draining to this gauging station starts around Berisa, and after different intermittent stream joining it; this stream converges to MeriboRiver. The geographical location of this gauging station is at latitude of 7^0 N and 39.366^0longitude. Catchment included by this stream gauging station is estimated to be around 133sqkm. The downstream part of the herero river is shown in Fig 3.10

Figure 3.10 Herero River

3.3Data collection

Thirty years f of historical stream flow data is generally considered to be the minimum necessary to assure statistical reliability in the study of hydropower as stated In the case of an organization (US Army corps of engineers manual, 1981). However, for many sites, considerably less than 30 years of data is available. Most of The Gauging stations on small rivers have got historical data less than the minimum required i.e., 30 years. For sites with shorter records, different techniques can be applied to extend a period of record. For this thesis the method used is discussed in the next section.

3.3.1 Stream flow data

Stream flow data are required for the purpose of model calibration & validation. Since most of the micro hydropower stations are ungagged we use models to estimate the stream flow at different points, but to use these models estimation, the models shall be calibrated & validated. That is why we need gauged data's. For this study purpose stream flow discharges of 5 stations

(Table 3.2) have obtained from Ministry of water, Irrigation, & Energy of Ethiopia, Hydrology department. ReferApendix A1

Table 3.2 station's providing stream flow data (data source ministry of water, irrigation, & energy of Ethiopia.)

St. No	station name	coordinates		Altitude (m.a.m.s.l)	Measured period	% of Missing Data
		Latitude	Longitude			
1	Meribo	7^o	39.33^o	2355	1984-2008	13.217
2	MeriboChanguite	6.95^o	39.36^o	-	1984-2008	35.66
3	Herero	7.95^o	39.366^o	2350	1989-2008	28.8
4	Leliso	7^o	39.383^o	2345	1984-2015	29.47
5	Furuna	7.016^o	39.416^o	2405	1990-2008	20.38

3.3.2 Rainfall data

To establish a rainfall-runoff relationship for a catchment, historical meteorological data are required. In this study the main purposes of those data were to use as an input to the HEC-HMS model in the hydrological model setup and development. In Ethiopia, the source of raw metrological data is the National metrological service agency (NMSA). A request for daily rainfall data of 30 years of period was made to the agency. Following the approval of the agency's higher official daily data of up to 20 years period were collected. Table 3.3 shows the description of these stations.

Table 3.3 stations providing metrological data's (ENMSA).

St. No	station name	coordinates		Altitude (m.a.m.s.l)	Measured period	% of Missing Data
		Latitude	Longitude			
1	Adaba	7.017^o	39.4^o	2420	1989-2015	38.2
2	Hunte	7.05^o	39.4^o	2380	1984-2015	22.64
3	MeskelDarkina	7.033^o	39.4167^o	3000	1989-2014	34.3
4	Gasara	7.13^o	39.933^o	1680	1984-2015	9.6

3.3.3 Geomorphologic map

Geomorphological maps are one of the most appropriate and synthetic ways to analyze the distribution of landforms, surface and near-surface deposits including the processes that shape landforms (Huggett, 2007). Beyond this, land use/land cover and artificial features can be incorporated in a geomorphological map according to their importance. (Gebremariam, 2010 as cited in Bizuneh ,2014)

Geomorphological map preparation started with field survey which was carried out with the help of GPS. Additional provided data was: i) the collection of topographical maps, thematic maps like land cover maps and soil maps ii) the Digital Elevation Model (DEM) 30 m x 30 m resolution.

3.4Data processing

Before beginning any hydrological analysis it is important to make sure that data are homogeneous, correct, sufficient and complete with no missing. Errors resulting from lack of appropriate data processing are serious because they lead to bias in the final answer. In any case data should be appropriately adjusted for consistency, corrected for error, extended for insufficiency, and filled for missing using different techniques.

3.4.1Metrological Data processing

3.4.1.1Data screening

Before stepping to any work the data that have been collected from metrological agency should be screened. Rough data screening of the five metrological stations was done by visual inspection of daily rainfall data.

3.4.1.2Detection for Outliers

Outlier detection methods can be generally divided into three groups: univariate, two-variate, and multivariate detection methods. Multivariate outlier detection method consists of multivariate determination of each observation according to a blend of variables.

Standard deviation method (M±3×SD)

One of the most important and well-known univariate outlier detection methods isM±3×SD. this conventional method is based upon the principle that first, data mean and 3×SD are obtained;

then, data higher than M +3×SD or lower than M -3×SD are regarded as outliers and eliminated from data set (Sajad Mirzaei1 et. al)Figure 3.11 shows some of the outlier rainfall points of the metrological stations.

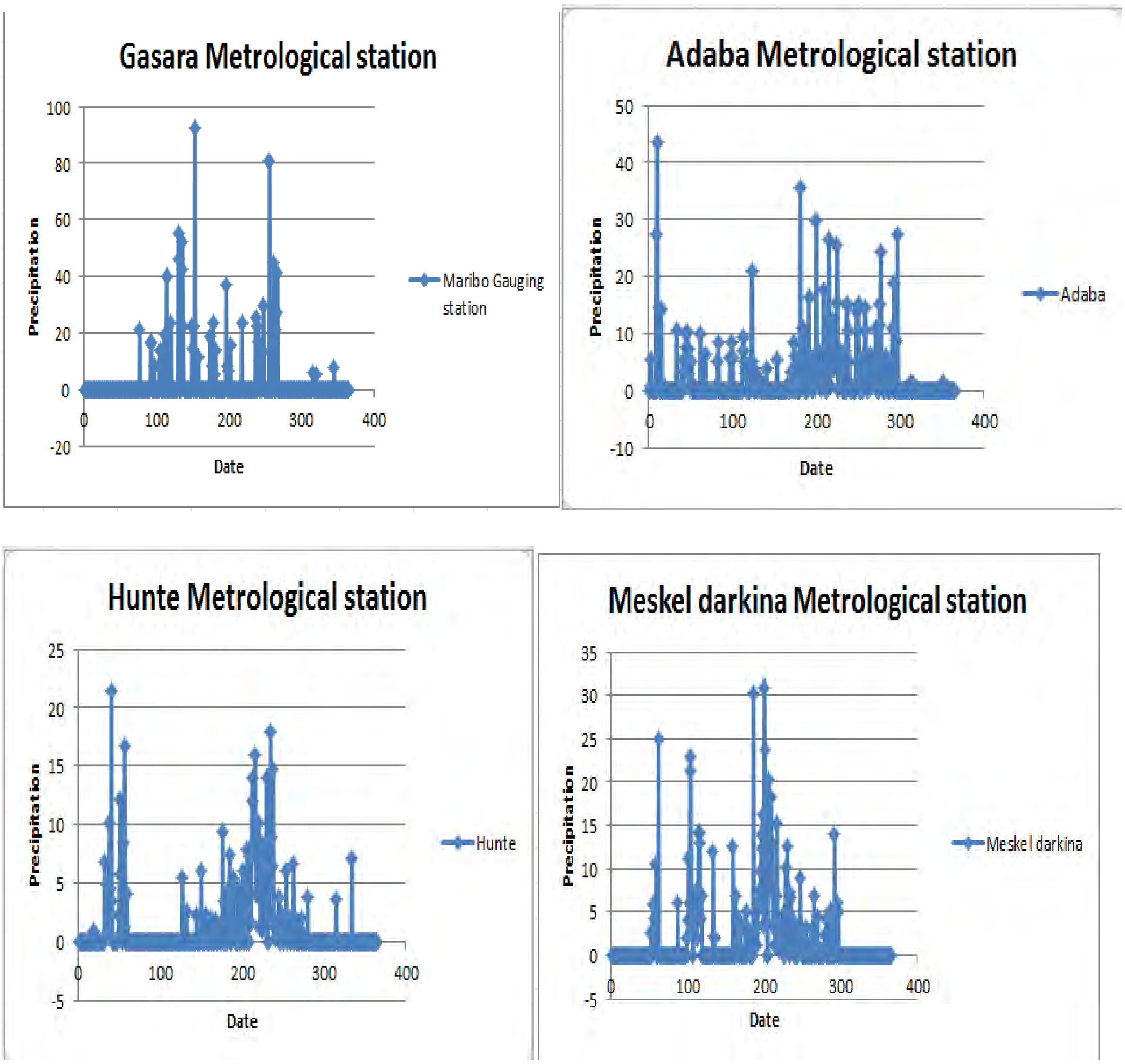

Figure 3.11 Metrological stations outlier detection

3.4.1.3 Filling in missing Data

Ordinarily, periods of recorded data at different location do not cover the same time span, and therefore, it is necessary to estimate missing values in order to obtain a complete set of data for analysis. (US corps manual, 1981) Most of the rainfall recorded from the stations has missing data ranging from 9 to 35 %. Therefore before using the data to runoff modeling it was must to fill the missed data. In filling missed data there are different techniques to be applied, generally two, using station own time series data, & neighborhood station data.

According to (Nardos, 2011) if the correlation coefficients is in the range $0.6<R^2<1.0$, it indicates good correlation then linear regression equation can be used to fill the missing data otherwise mean value should be used for filling the missing values. For this study First I have tried the neighborhood station data but the output of multiple regression techniques depicts that the stations have very low coefficient of determination which was less than the above mentioned value, so I decided to use the mean arithmetic techniques of its own station.

3.4.1.4Check for consistency & continuity

If the conditions relevant to the recording of a rain gauge station have undergone a significant change during the period of record, inconsistency would arise in the rainfall data of that station. This inconsistency would be felt from the time the significant change took place. The checking for inconsistency of a record is done by double mass curve technique (Subramanya.K, 1998)
The accumulated total of the individual gauge is compared with the corresponding totals for a representative group of nearby gauge. If a decided change in the regime of the curve is observed it should be corrected. In this study the stations which need correction are the Hunte & meskel Darkina(Fig 3.12).

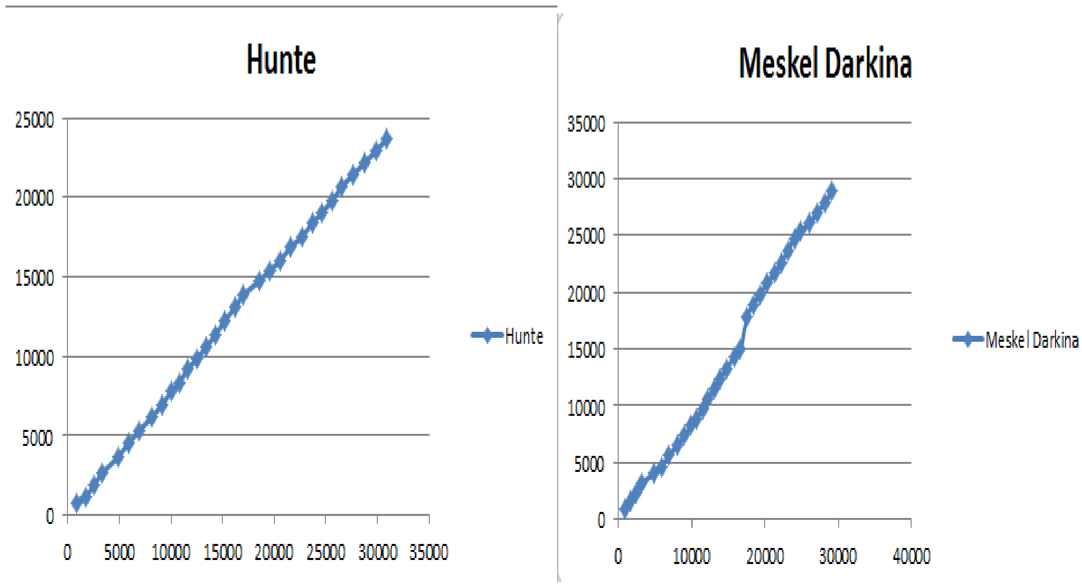

Figure3.12 Double mass curve for Hunte & Meskel darkina Metrological stations

3.4.1.5 Estimating Areal Precipitation

After the missing gaps are filled and the short records are extended, with the procedure mentioned above, the point rainfall was then converted to spatial rainfall. To do so there are different mechanisms:-

- Arithmetic average method:-When the rainfall is uniformly distributed over the area, the average rainfall may be taken as the arithmetic average of the recorded rainfall.
- Thiessen polygon method: - Rainfall varies in intensity and duration from place to place. Hence the rainfall recorded by each rain gauge station should be weighted according to the area it is assumed to represent.
- Isohytal method: - isohyets are a line joining places of equal rainfall intensities on a rainfall map of the basin. An Isohytal map represents an accurate picture of the rainfall distribution over the basin. If the network rainfall stations within the storm area are sufficiently dense, the Isohytal map will give a reasonably accurate indication of the rainfall distribution zones. In this study I used the Thiessen polygon method(Fig 3.13), & the Thiessen gauge waits are shown below for the four stations (Table 3.4) shows the wait of each station

Table 3.4Thissen weight for the stations

No	Station name	Area weight(Km²)	Thiessen Weight
1	MeskelDarkina	278.72	0.269
2	Hunte	86.17	0.083
3	Adaba	670	0.647
4	Gasara	0	0

After the Aerial precipitation is determined the gage wait for each sub basin is again calculated based on their Area. (Table 3.5)

Table 3.5 gage waits for each sub basin.

No	Sub basin	Gage wait
1	W130	0.03834535
2	w150	0.18480894
3	W160	0.02562051
4	W170	0.12335973
5	W180	0.12404119
6	W190	0.07298561
7	W200	0.1492351
8	W210	0.07335283
9	W220	0.20449819

3.4.2Hydrological data processing

3.4.2.1 Data screening

The initial step taken during the stream flow data screening as suggested by (Gordan et al. as sited in 1992 Mesgana 2013) was quick visual scan of the data time series to detect gross errors such as erroneous peak flow, missed recordings, & flow of constant rate. It helps to detect the year with magnitude change in the data, long period of missing records, and short-term missing data.

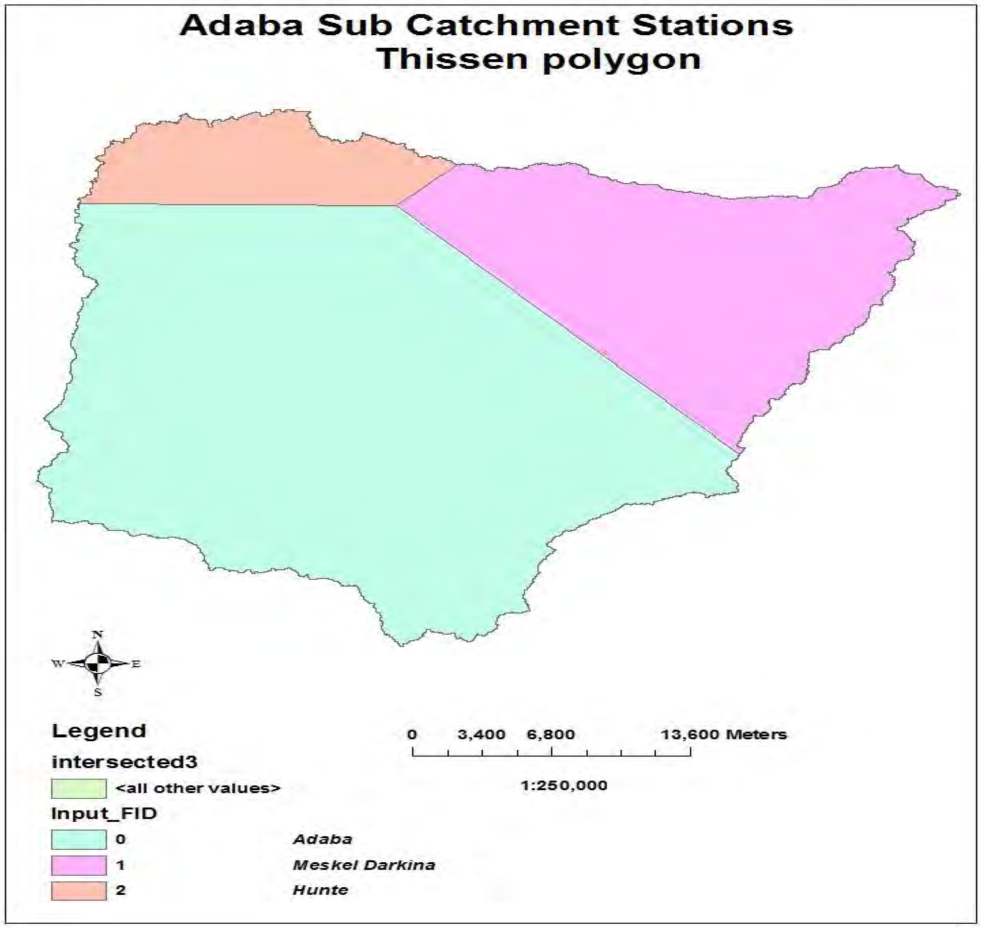

Figure 3.13 the metrological stations Thissen polygon

3.4.2.2 Detection for Outliers

The same method which was applied for the metrological data (Standard deviation method (M±3×SD)) were applied here to detect some data's which could outlier from the sample

3.4.2.3 Filling in missing Data

Again here as it is done for metrological data, filling missed data were also undertaken for the stream flow. There are around five gauging station, & almost all of them have missing data, though the range is different. The method here applied to fill the missing data is the same as the metrological one, as the coefficient of determination of the neighborhood stations was below the minima according to (Nardos, 2011)

3.4.2.4Check for consistency & continuity

The same procedure as the metrological stations was applied to check the consistency & continuity of the hydrological stations, after the stations were filled for missing data.

3.5 Base flow separation

Rainfall excess is converted into flow, which is a complicated process depending on many factors such as infiltration, soil type, soil storage characteristics, permeable soil depth, rainfall duration, rain intensity, soil and air temperature, evaporation, transpiration, humidity, wind speed, topography, land use and land cover characteristics, etc. and is composed of different components. Part of the rainfall excess moves on the earth's surface from topographically higher points to lower ones under gravitational effects. Another part infiltrates the soil and moves downward.(Hall, 1968) defines base flow as that portion of flow from the groundwater system Occurring under the effects of diverse geological, climatological, and morphological factors, base flow varies both in time and space (Singh, 1968).

A type of filter disintegrating the daily stream flow into quick flow and base flow components was made available by many researchers. A well-known one is the UK smoothed minima approach developed at UKIH (Institute of Hydrology, 1980). This is a widely used method (Wahl KL and Wahl TL, 1988, 1995; Mazvimavi*et al*., 2004 as cited in Hafzullah, Necati and Ali 2008).

3.6*Base* map of the study Area

The DEM is the important input to describe the topography of the watershed. The DEM used in this study is of 30 meters resolution i.e. 30*30 m grid size, with Transverse Mercator Projection (grid: UTM zone 37N), Clarke 1880 as spheroid and Adindan datum. & four topographic maps of scale 1:50000 map provided by the Ethiopian Mapping Authority (EMA). The schematic representation of the base map preparation is shown on fig 3.14

Table 3.6 List of topographic maps (data source Ethiopian mapping agency)

St. No	Topographical sheet number	Location name
1	0639A2	HERERO
2	0639B1	RIRA
3	0739C4	ADABA
4	0739D3	HAKO

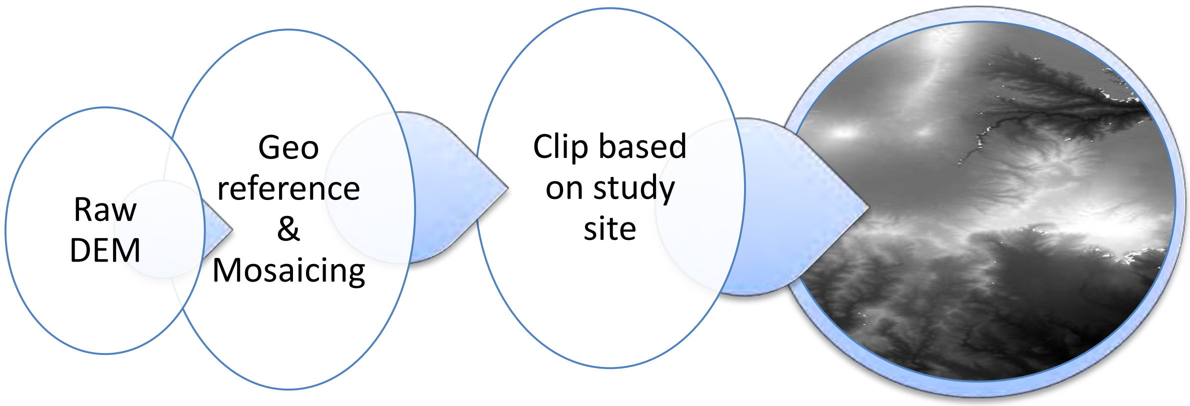

Figure 3.14 schematic representation of base map preparation

3.7 Modeling tools

The materials used for this research are

- ✓ ARC-GIS:-to analyze the physical and spatial information of the catchments.
- ✓ HEC-GeoHMS:- this is used to sub-divide the basin to more manageable form and determine basin characteristics for future use in HEC-HMS
- ✓ Arc Hydro tools:- this one is utilities based on the Arc Hydro data model for deriving hydrography data from DEMs, such as delineating watersheds, drainage networks and its derivatives for flow tracing
- ✓ DEM- since it describes the topography of the catchment it is used as an input data for ARC-GIS, where it is used for the catchment Preprocessing.
- ✓ HEC-HMS- it is a model designed to simulate the rainfall-runoff processes
- ✓ Hydrological data and metrological data.

3.8Discussions of models

A hydrologic system model is an approximation of the actual system; its inputs and outputs are measurable hydrologic variables and its structure is a set of equations linking the inputs and outputs. Central to the model structure is the concept of a system transformation (Chow, 1988). Models are normally characterized or classified to help describe and discuss their capabilities, strengths, and limitations. There is no universal method to characterize rainfall-runoff models, and models have been classified in several ways depending on the criteria of interest. Hydrologists have tried to classify rain fall runoff models according to their specific approach as well as their characteristics. The basic distinction between models is whether stochastic or deterministic representations and inputs are to be used. In the stochastic models, the chance of occurrence of the variable is considered thus introducing the concept of probability. In the deterministic models, the chance of occurrence of the variables involved is ignored and the model is considered to follow a definite law of certainty but not any law of probability (Raghunath, 1985).

3.8.1 HEC-GeoHMS

HEC-GeoHMS has been developed as a geospatial hydrology tool kit for engineers and hydrologist. The program is an extension of Arc GIS and allows users to visualize spatial information, document watershed characteristics, perform spatial analysis, delineate sub-basins and streams, construct inputs to hydrologic models, and assist with report preparation. Eight data sets can be derived from DEM that collectively describe the drainage patterns of the watershed.

3.8.2 HEC-HMS

HEC-HMS is hydrologic modeling software developed by the US Army Corps of Engineers Hydrologic Engineering Center (HEC), it is the physically based and conceptual semi distributed model designed to simulate the rainfall-runoff processes in a wide range of geographic areas such as large river basin water supply and flood hydrology to small urban and natural watershed runoff. The system encompasses losses, runoff transform, open channel routing, analysis of meteorological data, rainfall-runoff simulation and parameter estimation.

HEC-HMS uses separate models to represent each component of the runoff process, including models that compute runoff volume, models of direct runoff, and models of base flow. Each model run combines a basin model, meteorological model and control specifications with run options to obtain results.

The main reasons for the selection of HEC-HMS hydrological model for this research is that the model is physically based, spatially distributed and it belongs to public domain. It has been used in wide geographical area including climate change studies. The other reason is that there are rivers that are not gaged in this sub catchment, so by using HEC-HMS we can estimate all necessary parameters for run of modeling of the ungagged rivers from the neighborhood gaged catchment of the same Hydrological characteristics.

Chapter 4

4. Results and discussion

4.1Base flow separation

For this study I have used the Smoothed minima method to separate base flow & the steps which have been followed is discussed below (refer Appendix C, for base flow separation results). With the UKIH method, base flow separation is performed on consecutive daily stream flow time series. Given that the data are presented in the form of consecutive average daily flows, (Hisdal*et al*. (2003) as cited in Hafzullah, Necati and Ali 2008.)Presents the method, step by step, as follows:

(a) Divide the daily flow data into non-overlapping blocks of five days.

(b) Mark the minima of each of these blocks, and call them Q1, Q2, . . . , Qi. Consider in turn (Q1, Q2, Q3), Q2, Q3, Q4, . . . , Qi1, Qi, QiC1. In each case if

$0.9Q_i < \min(Q_{i-1}, Q_{i+1})$ Equation (1)

Then the central value is a turning point for the base flow line. Continue this procedure until the whole time series has been analyzed.

(c) Let the discharges at the turning points be Q1, Q2, . . .,Qm. Join the turning points by straight lines to form the base flow hydrograph. If, on any day, the base flow estimated by this line exceeds the total flow on that day, the base flow is set equal to the total flow. (Hafzullah, Necati and Ali 2008)

4.2 Digital Elevation Model (DEM) processing

Terrain Pre Processing

The terrain preprocessing (TPr) is the first step in using HEC Geo-HMS. A terrain model is used as an input device that describe the drainage patterns of the watershed with eight additional data sets, that allows for stream and sub-basin delineation. These eight additional data sets are Fill Sinks, Flow Direction, Flow Accumulation, Stream Delineation, Stream Segmentation, Catchment Grid Delineation, Catchment Polygon Processing, and Drainage line processing. The working of Geo-HMS and HEC-HMS in pictorial form has been shown in Figure 4.1 (a) and

Figure 4.1 (b). The results acquired after Geo-HMS are the catchment area of each sub-basin, slopeof each sub-basin, flow length, which will help us for the calculation of “Time of Concentration”. Nine sub basins, four reaches, four junctions and one outlet have been finalized in Geo-HMS analysis.

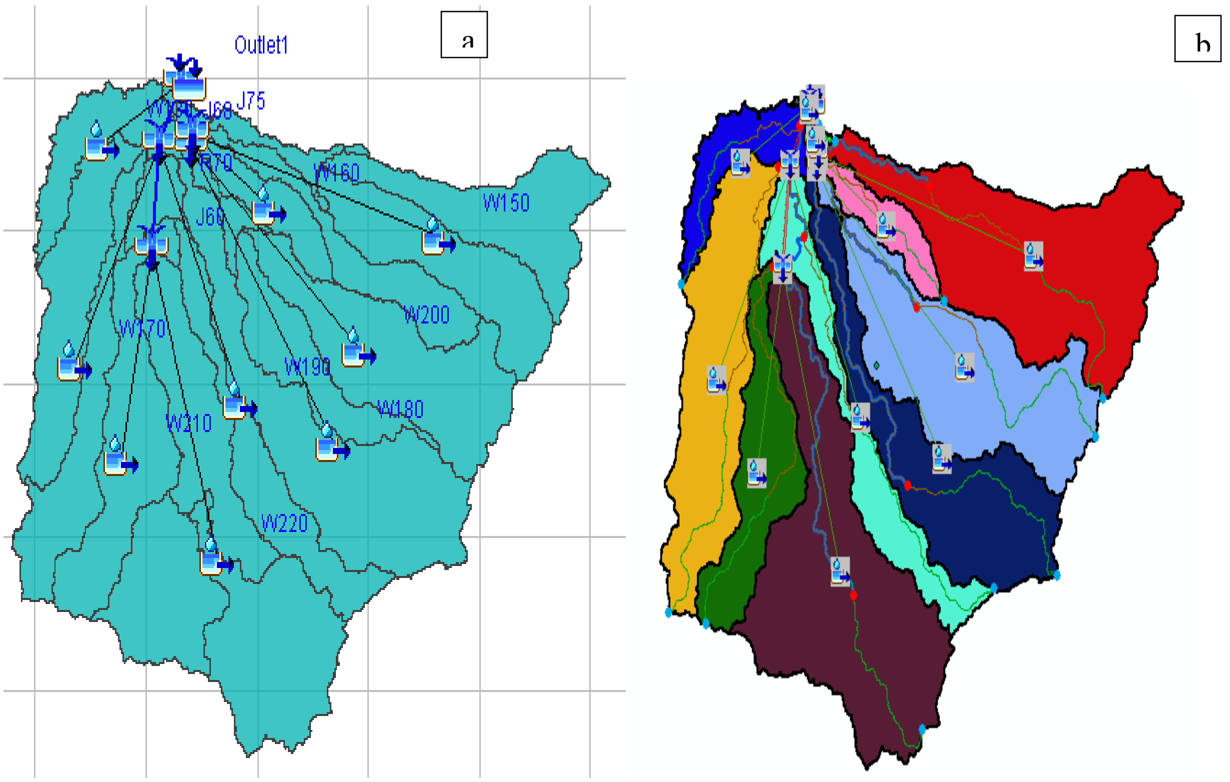

Figure 4.1 (a) HEC-HMS extracted from GeoHMS (b) final shape of HEC-GeoHMS

4.3 Watershed analysis

The analysis of morphometric characteristics of a catchment like drainage network, slopes, divides and sub-basin boundaries is the prerequisite for hydrological modeling (Johnson, 2009 as cited in Bizuneh 2014).Watershed analysis includes initial parameter estimation for rainfall-runoff modeling and determination of DEM derivatives for further analysis and simulation.

The Hydrologic Engineering Center Geospatial Hydrologic Modeling Extension (HEC-GeoHMS) and Arc Hydro tools coupled with Arc GIS 10.2 were used for watershed analysis and relate with the HEC-HMS model (Figure 4.2). The Arc Hydro tools are utilities based on the Arc Hydro data model for deriving hydrography data from DEMs, such as delineating watersheds, drainage networks and its derivatives for flow tracing (Johnson, 2009 as cited in

Bizuneh 2014). The tools provide raster, vector, and time-series functionality, and many of them populate the attributes of Arc Hydro features. Further, the Arc Hydro tool provides a data base (spatial data management) that uses various applications to develop hydrologic model (Johnson, 2009 as cited in Bizuneh 2014).

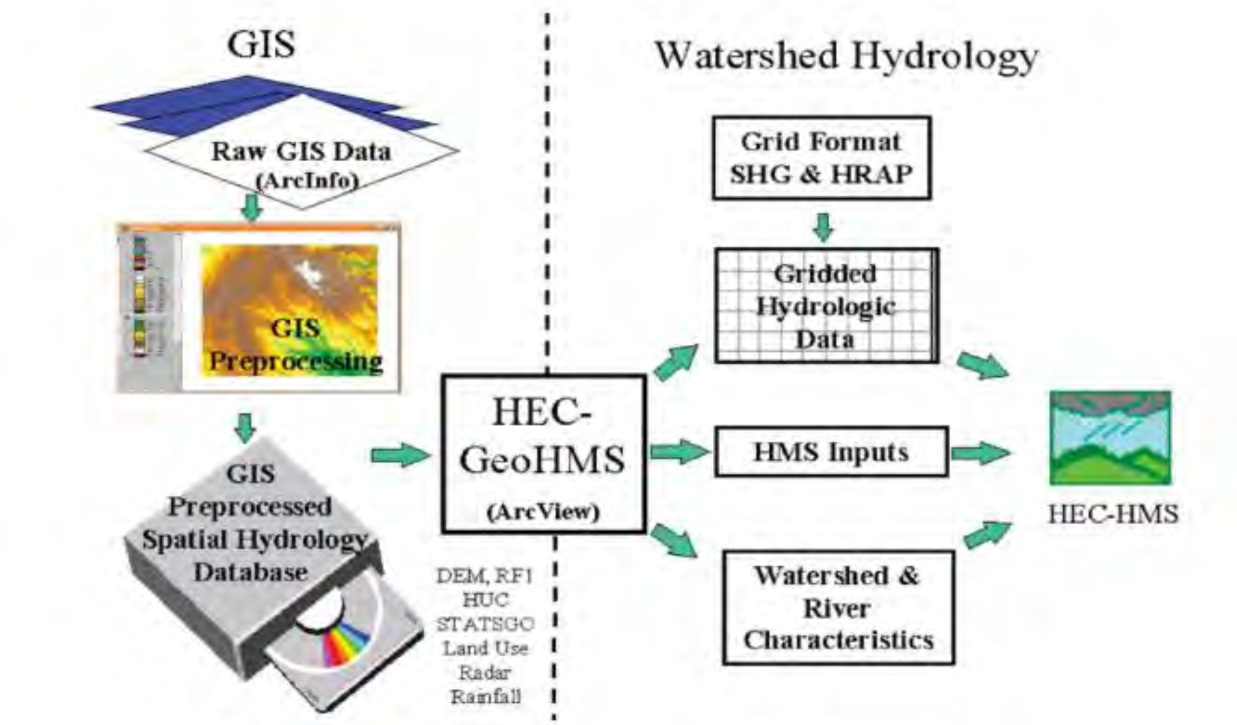

Figure 4.2 relation between GIS, HEC-GeoHMS, & HEC-HMS(HEC-GeoHMS user's manual, 2009)

The program features are terrain preprocessing, basin processing, hydrologic parameter estimation and Hydrologic Engineering Center Hydrologic Modeling Systems (HEC-HMS) model support (Feldman and Doan, 2009 as cited in Bizuneh 2014). The first two features are accomplished through a number of procedural steps. Terrain preprocessing includes filling sinks, assigning flow direction and flow accumulation, defining stream network, sub-watershed area sizes, basin slope and some other watershed characteristics that collectively describe the drainage pattern and geometry of a basin. Additionally, parameters like river slope, river length, watershed centroid, and longest and centroidal flow path were also determined (Fleming and Doan, 2009 as cited in Bizuneh 2014). Finally, the automatically generated drainage network from the DEM will be compared with the topographic maps at scale 1:50 000

4.4 Model setup & parameter estimation

The conceptual and physically based models can be categorized as lumped, semi distributed and distributed. Lumped model treats the catchment area as one or more homogeneous land segments where the inputs are averaged. Distributed model explicitly represents the spatial variability by dividing the catchment into grids and modeling each grid cell individually. Semi distributed model is a conceptual model that bridge the gap between lumped and distributed models. It utilizes conceptual relationships for hydrological processes that are applied to several relatively homogeneous sub-areas of the catchment area. (Er. Narayan Prasad Gautam, 2015) the main objective of this model in this study is to estimate the runoff depending on the given parameter that will be determined by calibration,

4.4.1 Basin Model

Following methods were selected for each component of runoff process such as runoff depth, direct runoff, base-flow and channel routing(Fig 4.3). These methods are selected on the basis of applicability and limitations of each method, availability of data, suitability for same hydrologic condition, well established, stable, widely acceptable, researcher recommendation etc.(Kishor, Panigrahi, & Chandra 2014)

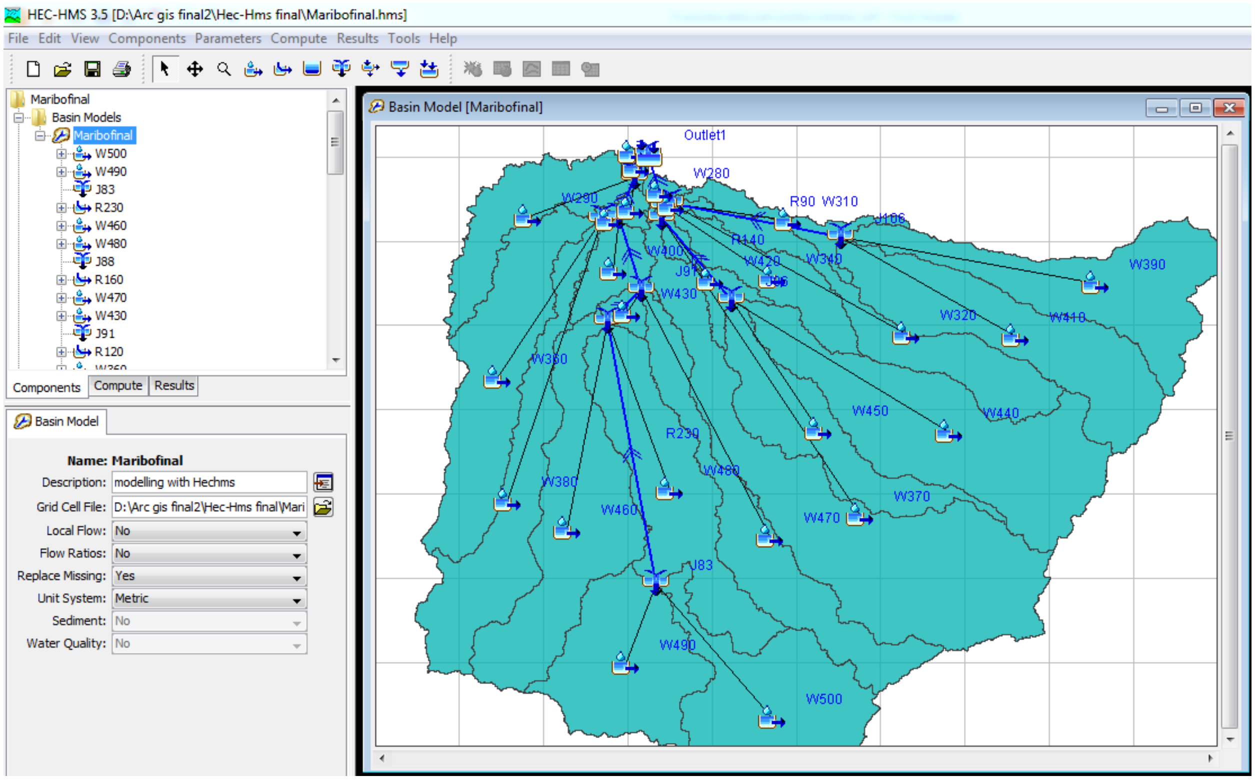

Figure 4.3 basin model

4.4.2Loss method

For this study Initial and constant loss rate is selected. this method include two Parameters of constant rate and Initial loss which show the physical characteristics of soil, land use and antecedence conditions of basin (Radmanesh et al., 2006 as cited in Saleh). If the basin be in saturated conditions, Ia tends to zero. When the soil of basin is dry, Ia will increase and show the maximum height of rainfall that cannot be changed into runoff. According to American standards, range of Ia in forest area is between 10 to 20 percent of rainfall and it varies between 0.1 to 0.2 inches in urban areas. Table 4.1 shows the result of Ia for the sub basins

Constant loss rate shows the final capacity of soil. (Saleh, 2012).SCS method, classify soil based on their infiltration capacity into four categories (KhalighiSigaroodi2004) calculated and published the rate for different groups of soils (Table 4.2). (Radmanesh et al., 2006 as cited in saleharkehi, 2012). Based on the soil map data analysis the major soil group in Adaba sub catchment is soil group D, which makes the constant loss to be in the range of (0-1.25)

Table 4.1Ia values for each sub basin

no	1	2	3	4	5	6	7	8	9
Sub basin	W130	w150	W160	W170	W180	W190	W200	W210	W220
Ia(mm)	0.008	0.038	0.005	0.026	0.026	0.015	0.031	0.015	0.042

Table 4.2 Classification of soils and their infiltration rate.

Hydrological soil groups	Soil texture	Infiltration(mm/hr)
A	Sand, Loamy sand or sandy loam	7.62-11.43
B	Silt loam or loam	3.81-7.62
C	sandy clay loam	1.25-3.81
D	Clay loam, silty clay loam, sandy clay, silty clay, or clay	0-1.25

4.4.3Modeling surface runoff

SCS unit hydrograph

The curve of runoff changes in terms of time is called hydrograph. It is able to prepare the maximum runoff, volume and the amount of retention of flooding in a watershed. In this study, SCS Dimensionless Hydrograph has been used to generate unit hydrograph for the selected event rainfall. It based on the converting time and flow axis to dimensionless hydrograph in flood hydrograph. It is implemented by dividing the real time of hydrograph by "time to peak", and also dividing the flow of hydrograph by "flow to peak. The method is based on the two assumptions which state firstly, flow at any time is proportional to the volume of runoff, and secondly, time factors affecting the hydrograph shape are constant. The parameters used in SCS dimensionless unit hydrograph are Time of concentration, Lag time, Duration of the excess Rainfall, Time to peak flow, Peak flow. (Reza, Andrew, &Ramani, 2013). The relevant equations used in this hydrograph are listed below:

Research by the SCS suggests that the UH peak & time of UH peak are related by:

$Up = C\frac{A}{Tp}$ Equation (2)

In which A = Watershed area; and C=conversion constant (2.08 in SI and 484 in foot-pound system). The time of peak (also known as the time of rise) is related to the duration of the unit Of excess precipitation a:

$T_p = \frac{\Delta t}{2} + t_{lag}$ Equation (3)

In which Δt = the excess precipitation duration (which is also the computational interval in the run); and t_{lag} = the basin lag, defined as the time difference between the center of mass of rainfall excess & the peak of the UH. (US corps of engineers Hydraulic engineering center 2000)

Estimating the model parameters for scs unit hydrograph

For ungagged watersheds, the SCS suggests that the UH lag time may be related to time of concentration, t_c as:

$t_{lag} = 0.6t_c$Equation (4)

Time of concentration is aquasi-physically based parameter that can be estimated as:

$t_c = t_{sheet} + t_{shallow} + t_{channel}$Equation (5)

Where t_{sheet}= sum of travel time in sheet flow segments over the watershed land surface; $t_{shallow}$ = sum of travel time in shallow flow segments. Down streets, in gutters, or in shallow rills and rivulets; and $t_{channel}$ = sum of travel time in channel segments.

Identify open channels where cross section information is available. Obtain cross sections from field surveys, maps, or aerial photographs. For these channels, estimate velocity by manning's equation:

$V = \frac{CR^{2/3}S^{1/2}}{n}$ Equation (6)

Where V=average velocity; R=the hydraulic radius (defined as the ratio of channel cross-section area to wetted perimeter); S = slope of the energy grade line (often approximated as channel bed slope); and C = conversion constant (1.00 for SI and 1.49 for foot-pound system.) values of n, which is commonly known as manning's roughness coefficient, can be estimated from textbook tables, Once velocity is thus estimated, channel travel time is computed as:

$T_{channel} = \frac{L}{V}$Equation (7)

Where L= channel length.

Sheet flow is flow over the watershed land surface, before water reaches a channel. Distances are short-on the order of 10-100 meters (30-300feet). The SCS suggests that sheet-flow travel time can be estimated as:

$t_{sheet} = \frac{0.007(NL)^{0.8}}{(P_2)^{0.5}S^{0.4}}$ Equation (8)

In which N= an overland flow roughness coefficient; L=flow length; p_2 = 2-year, 24hourrainfall depth, in inches; and slope of hydraulic grade, which may be approximated by the land slope. (Table 4.3)shows values of N for various surfaces.

Sheet flow usually turns to a shallow concentrated flow after 100 meters. The average velocity for shallow concentrated flow can be estimated as:

$$V=\begin{cases}16.1345\sqrt{S}\ for\ unpaved\ surface\\ 20.3282\sqrt{S}\ for\ paved\ surface\end{cases} \quad \text{Equation(9)}$$

From this the travel time can be estimated with Equation (8), the estimated value of t_{lag} for each sub basin is shown in Table 4.4.

Table 4.3 values of N for various surfaces

Surface description	N
smooth surfaces (concrte, asphalt, gravel, or bare soil)	0.011
Fallo (no residue)	0.05
cultivate soils:	
Residue cover≤20%	0.06
Residue cover≥20%	0.17
Grass	
Short grass prairie	0.15
Dense grasses, including species such as weeping love grass, blue grass, buffalo gass, and native mixtures	0.24
Bermudagrass	0.41
Range	0.13
woods†	
Light underbrush	0.4
Dense under rush	0.8

Notes

† When selecting N, consider cover to a height of about 0.1ft. This is only part of the plant cover that will construct sheet flow.

Table 4.4estimated Values for t_{lag}

sub basin	distance from centroid	t_{lag}(min)
W130	16374	730.545687
W150	14927	2519.785088
W160	7829	414.9165276
W170	13845	1672.936346
W180	7738	1666.992042
W190	3151	1357.388697
W200	7518	1343.18443
W210	11874	2765.642147
W220	11865	1262.951763

4.4.4 Modeling channel flow

Muskingum

The HEC-HMS includes different method for the channel flow or also known as routing model. The routing models available include:- Lag, Muskingum, Modified puls (storage routing), Kinematic wave, Muskingum Cung. Each of these models computes a downstream hydrograph, given the upstream condition.((HEC-HMS technical reference) For this study Muskingum model is selected The derivation of the original Muskingum routing model is based on Eqs. (10) and (11) for a channel or river reach without lateral inflow:

$\frac{dW}{dt} = 1 - Q$ Equation (10)

$W = K[xl + (1 - x)Q]$ Equation (11)

Where W is the water storage, $t$is time, I is the inflow, and Q is the outflow. Equation. (10) represents the mass balance, and Equation. (11) Expresses the channel storage volume, which is a simple linear combination of the inflow discharge of the upstream section and the outflow of the downstream section. In Equation (10) and (11), K and x are the two model parameters determined from observations; they represent the storage-time constant, which has a value reasonably close to the flow travel time through the river reach, and a weighting factor usually ranging from 0 to 0.5. Therefore, the key objective of the Muskingum model is to estimate the parameters K and x.

Estimation of K

The wave travel time K can be estimated by Equation. (12):

$K = \frac{L}{3600V_c}$ Equation 12

Where L is the reach length, and Vc is the flood wave celerity, which can be calculated by Equation. (13):

$V_c = \frac{dQ}{dA}$ Equation 13

The relationship between flood wave celerity and velocity can be obtained from the following formula (Todini 200 as cited in Xiao, Fan, & Zhao 2011): $V_c = \frac{5}{3}(1\text{-}\frac{4}{5}\frac{A}{BPsin\alpha})V_{av} = \lambda V_{av}$ Equation(14)

Where B is the water surface width, and α is the angle formed by dykes over a horizontal plane. λ is the wave celerity coefficient or shape coefficient of the channel cross section, whose values are 5/ 3 , 4/ 3 , and 13/ 9 for rectangular, triangular, and parabolic channel cross sections, respectively (Lin 2001 as cited in Xiao, Fan, & Zhao 2011). For this research case since the streams cross section are composite of the mentioned shape i took the average value of λ which is equals to 1.481. The calculated K values for each reach is shown in (Table 4.5)

Table 4.5 estimated K values for each Reach

Reach	V_{av}	L	Λ	V_c	K
R70	5.21	5808	1.48	7.70	0.20
R30	13.20	3766	1.48	19.53	0.05
R20	8.22	7666	1.48	12.16	0.17
R40	25.39	1205	1.48	37.58	0.008
R10	21.87	2149	1.48	32.36	0.018

Estimation of X

he parameter x of the Muskingum model is a physical parameter that reflects the flood peak attenuation and hydrograph shape flattening of a diffusion wave in motion (Rui et al. 2008 as cited in Xiao, Fan, & Zhao 2011).Once K is estimated, X can be estimated by trial & error. The feasible range of X is (0, 0.5)

4.4.5 Modeling Base flow

Base flow model simulates the storage and movement of subsurface flow which is the precipitation part that infiltrates the soil and moves downward. The program includes three alternative models of base flow namely: constant, monthly varying value, Exponential recession model, & Linear-reservoir volume accounting model. For this study the constant, monthly varying value is selected.

4.4.6 Metrological model

The meteorological component is the first computational element by means of which precipitation input is spatially and temporally distributed over the river basin. The spatio-temporal precipitation distribution is accomplished by the gauge weight method. The Thiessen's polygon technique was used to determine the gauge weights

4.4.7 Control Specification

The control specification model provides time related information for the model simulation such as the start and end of the computation period and the computation time interval (ScharffenbergandFleming, 2010 as cited in Bizuneh 2014). Since the available data are daily the computation time interval was one day.

4.5 Model calibration, Validation, sensitivity analysis & Evaluation.

4.5.1 Sensitivity analysis

The sensitivity analysis of the model was performed to determine the important parameters which needed to be precisely estimated to make accurate prediction of basin yield. Thus, at first the model was run with the model input values (the base data file), estimated by methods presented above and base output was collected. This was followed by varying each input parameter within prescribed range keeping the others constant and running the model. The output values were analyzed to determine their variations with respect to the base output set and this is a measure of the sensitivity.

4.5.2 Calibration

To improve the agreement between the simulated and observed data of the identified sensitive parameters Again, the automated calibration procedure in HEC-HMS uses an iterative method to minimize an objective function, such as sum of the absolute residuals, sum of the squared residuals, peak-weighted root mean square error etc. (HEC 2000 as cited in D. Roy, S. Begam, S. Ghosh and S. Jana). Thus, both manual and automated calibration methods were used for this study. The model has been calibrated with Maribo stream gauging station. The calibration was done for the period of 1984-1986, (Fig 4.4).

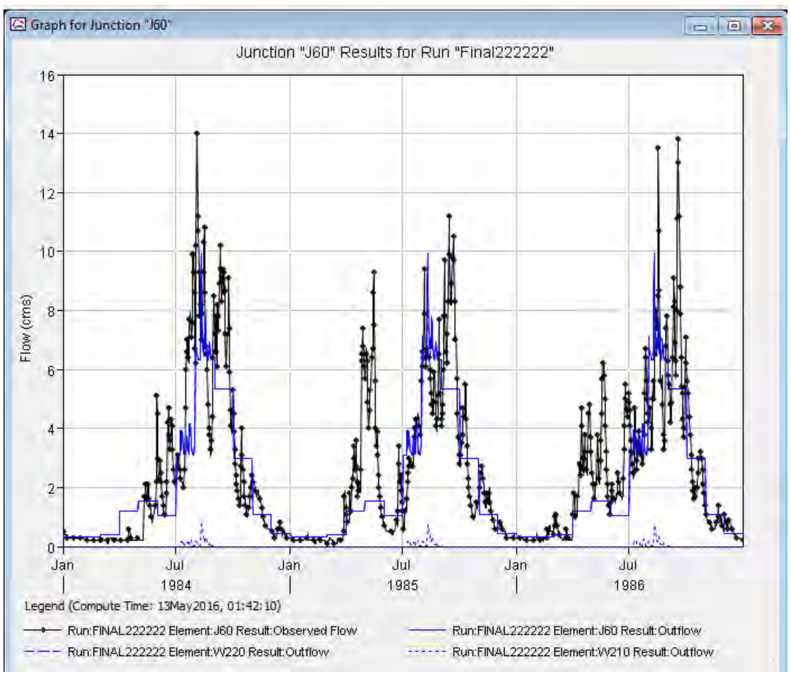

Figure 4.4 calibration at Maribo gaging station

4.5.3 Validation

The calibrated model was then used to estimate daily stream flow from the sub basins for the years 1987-1989 using the calibrated parameter. The observed and simulated flow for the sub basins indicates a close relationship (Figure 4.5). Thus the results indicate that overall estimation of stream flow by the model during the calibration period is satisfactory and therefore may be accepted for further analysis.

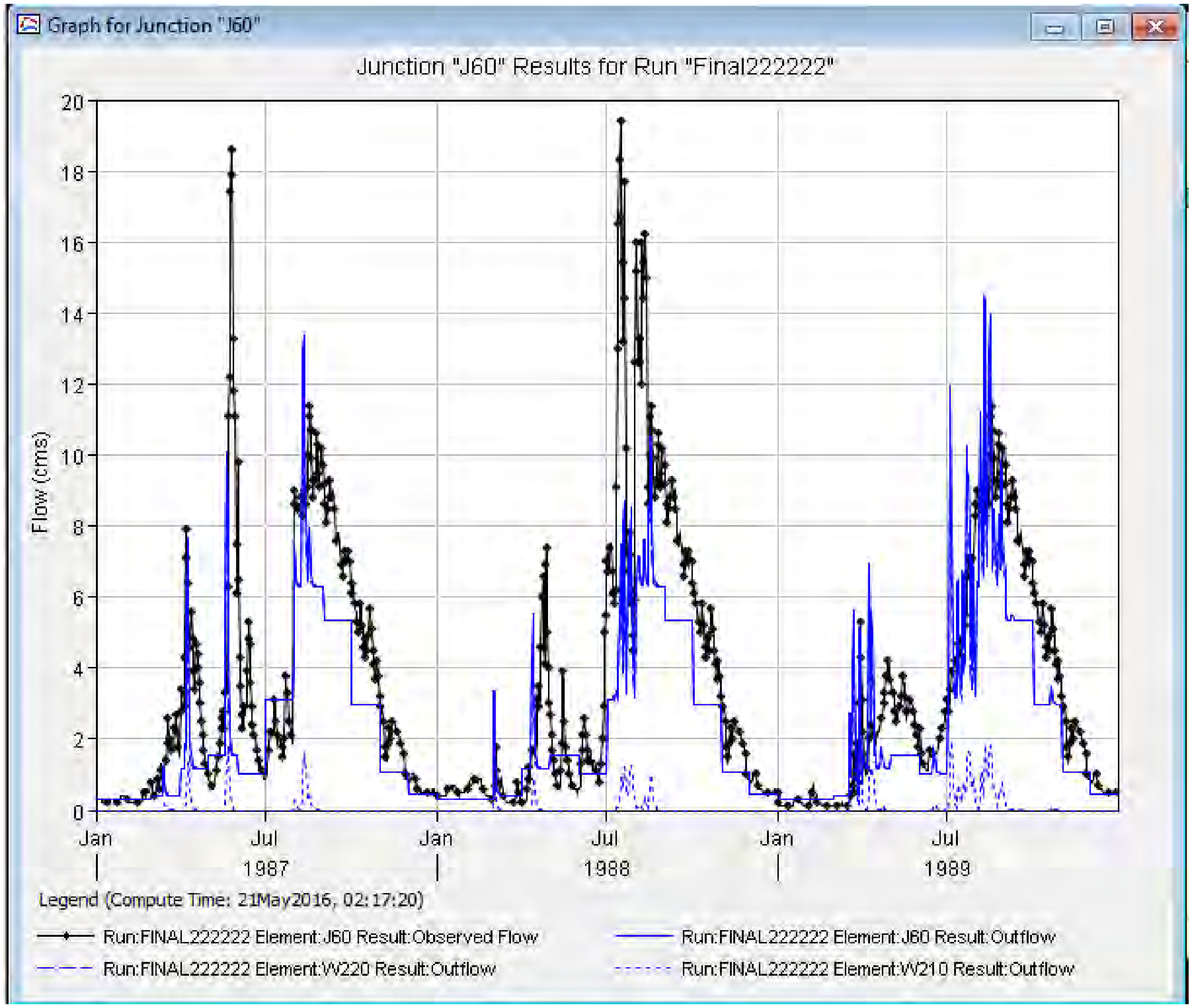

Figure 4.5 validations at Meribo gaging station.

4.5.4 Evaluation of model performance

The performance of the model was evaluated comparing daily simulated discharge with the observed discharge. The statistical tools such as Nash-Sutcliffe Efficiency (NSE), Coefficient of determination (R^2), Percent Volume Error (PVE), and percent error in simulated peak (PEP) were used to measure the model outputs.

a. **Nash-Sutcliffe Efficiency:**

It is calculated as,

$$NSE = 1 - \frac{\sum(Q_{obs(t)} - Q_{sim(t)})^2}{\sum(Q_{obs(t)} - Q_{obs,mean})^2} \qquad \text{Equation (15)}$$

Where NSE is Nash-Sutcliffe Efficiency, *Qobs(t)* is observed discharge at time t, Qsim(t) is simulated discharge at time t and Qobs,mean is average observed discharge. The “t” used in the calculation is the time period used (Nash &Sutcliffe, 1970). The value varies from - ∞ to 1. With the increase in performance, the numerical value increases and becomes maximum 1 in ideal case when simulated and observed hydrograph exactly match each other (Bhattarai, 2013, D. Roy et al. 2013. The EFF(NSE) values can vary from 0 to 1, with 1 indicating a perfect fit of the data. According to common practice, simulation results are considered to be good for values of EFF greater than or equal to 0.75, while for values of EFF between 0.75 and 0.36 the simulation results are considered to be satisfactory (Motovilov*et al*., 1999 as cited in D. Roy et al. 2013)

b. Coefficient of determination

Another widely used statistical measures Coefficient of determination (R^2) is given by,exactly match each other, a value of 1 is obtained. These objective functions are widely used in hydrological modeling(Masih et al. 2011).

$$R^2 = \frac{\left(\sum(Q_{obs(t)} - Q_{obs,mean}) \sum Q_{sim(t)} - Q_{sim,mean(t)})\right)^2}{\sum(Q_{obs(t)} - Q_{obs,mean})^2 \sum Q_{sim(t)} - Q_{sim,mean(t)})^2} \qquad \text{Equation (16)}$$

Where, Qobs and Qsim are observed and simulated discharge; Qobs and Qsim are mean observed and simulated discharge. Coefficient of determination varies from 0 to 1 where higher value denotes better fit of the regression line between simulated and observed discharges. When simulated and observed discharges exactly match each other, a value of 1 is obtained. These objective functions are widely used in hydrological modelling (Masih et al., 2011).Regarding the accepted range of R^2typically values greater than 0.5 is considered acceptable (Santhi et al. 2001; Van Liew et al., 2003, as cited in D. N. Moriasi et al. 2007).

c. Percent volume error

Finally, the volume balance was also calculated by using another statistical method called Percent Volume Error (PVE) which is given by,

$$PVE = \frac{V_{sim} - V_{obs,mean}}{V_{obs,mean}} * 100 \qquad \text{Equation (17)}$$

Where, Vsim and Vobs are average simulated and observed volume of stream flow. PVE shows by what percentage the simulated flow is underestimated or overestimated. Positive value means over estimation and negative shows underestimation. If simulated and observed flow is exactly same then an ideal case of 0 is obtained. The objective of the model improvement was set based on statistical tools as maximizing NSE and R^2, minimizing PVE& PPE (D. Roy et al. 2013).

d. Percent Peak error

This measures only the goodness-of-fit of the computed –hydrograph peak to the observed peak. It quantifies the fit as the absolute value of the difference, expressed as a percentage, thus treating overestimates and underestimates as equally undesirable. PPE is given by,

$$PPE = \frac{q_s(peak) - q_o(\text{peak})}{q_o(peak)} * 100$$

It does not reflect errors in volume or peak timing. This objective function is a logical choice if the information needed for designing or planning is limited to peak flow or peak stages. This might be the case for a floodplain management study that seeks to limit development in areas subject to inundation, with flow and stage uniquely related. (US Army corps of engineers, 2000). The satisfactory range for PVE & PPE as explained in (D. N. Moriasi et al. 2007) is ±15 - ±25.

Overall performance of the model is summarized in (Table 4.6)

Table 4.6 HEC-HMS performance during the calibration & validation periods

NO	Model efficiency criteria	Daily time step result		satisfactory range
		Calibration	Validation	
1	Nash sutcliffe Efficiency, NSE	0.79	0.72	>0.36
2	Coefficient of determination	0.64	0.695	>0.5
3	The relative error of the peak	25.6	32	±15 - ±25
4	The relative error of the volume	-12.6	-17	±15 - ±25

After the model was calibrated and validated for sufficient period in sub basin (W210) which is found at the center of the sub catchment & assumed to be representative of all the sub catchment, then the daily runoff modeling was done for all sub basins with the defined parameter by calibration & validation. It is these outputs from the model that are transferred to the specific points after the potential sites were selected. (The detail of transferring data to the potential site is discussed in section 4.9)

4.6 Selection of Potential Development Sites

The potential sites that are suitable for hydropower generation are selected by considering different parameters such as Site identification considering river gradient and catchment area, Identification based on waterway construction conditions, &Identification Based on Local Information In cases where potential sites cannot be interpreted on the topographical map because of the small usable head or the presence of a fall or pool, for this study the two basic parameters that are considered is discussed below.

(1) Level of firm discharge

While it is difficult to judge the suitability for development based on the absolute volume of firm discharge, a potential site with a relatively high level of firm discharge is more favorable site for a micro-hydro plant designed to supply power throughout the year.

(2) L/H [ratio between waterway length (L) and total head (H)]

A site with a smaller L/H value is more advantageous for small-scale hydropower. Figure 4.6 shows the relation of the ratio between the total head (H) and the waterway length (L) (L/H) among existing small-scale hydropower sites where the total head is not less than 10 m (the minimum head which can be interpreted on an existing topographical map). As clearly indicated in the figure, the L/H of micro hydro is generally not higher than 40 or is an average of 25. (DEUMB, 2009).

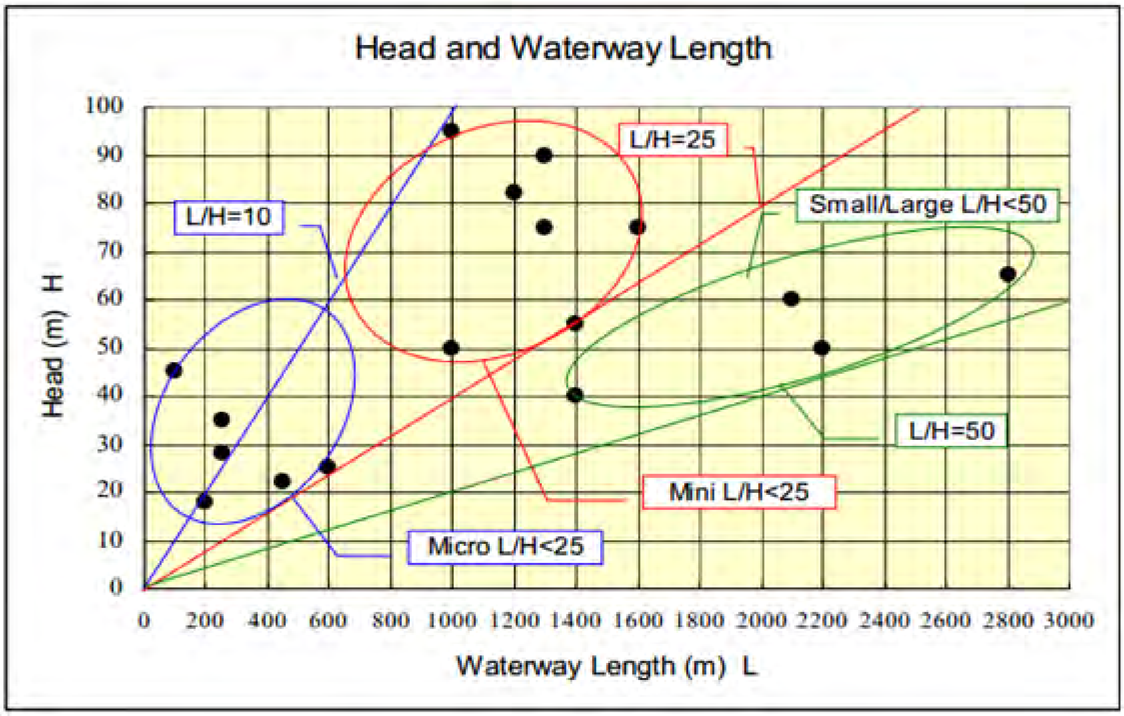

Figure 4.6 Relation between water way length &Head *(DEUMB, 2009)*

4.7 Stream flow and Head of flow

In the assessment of any hydropower plants potential, flow of water and head are essential. It is because the power generated by the HP plant is normally dependent on stream flow and head of flow.

i. Stream flow

Two methods are widely used for analysis of stream flow data as a primary estimation of power. Potential-flow-duration curve (FDC) and sequential stream flow routing (SSR). The flow-duration curve method is better method for all preliminary or screening studies. This method is also the best choice for high-head, run-of-river projects where head is generally fixed or even for low-head projects where head varies with discharge. For multipurpose storage projects, the SSR method is more appropriate and also can be used for examining the feasibility of including power at new water conservation or flood control projects. For peaking and pumped storage projects, hourly SSR routing is required (Mohammed et al., 2003)

Flow-duration curves (FDC) are used to summarize stream flow characteristics and can be constructed from daily, weekly, or monthly stream flow data. These curves show the percentage of time that flow equals or exceeds various values during the period of record, for example Q (50, daily) is the flow which is reached or exceeded statistically in 50% of the time, meaning in an average year at 50% of 365 days.

The disadvantages of the flow-duration curve is that it does not present flow in chronological sequence, does not describe the seasonal distribution of stream flow, and does not account for variations of head independent stream flow. However, these curves are useful for evaluating the power output of run-of-river projects and for other power projects where head varies directly with flow (EM1110-2-1701, 1985, As sited in Nardos 2011,)

There are two computing ordinates for plotting flow duration curve, viz. the rank-ordered technique and the class-interval technique. The rank-ordered technique considers a total time series of flow that represent equal increments of time for each measurement value, such as mean daily, weekly, or monthly flows, and ranks the flows according to magnitude. To develop the flow-duration curve, the observed stream flows should be arranged in descending magnitude. Then the data are ranked, the largest beginning with order 1, from 1 to total number of data. The probability of excedence is estimated by the following relation

$p_i = \frac{m}{n+1} * 100$ Equation (18)

Where

P_i=plotting position

m= rank

n= total number of data

The FDC is plotted according to the above procedure (Mohammed et al., 2003) whereas in class-interval technique the time series of flow values are categorized in to class intervals, the classes range from the highest flow value to in the series to the lowest value in the time series. A tally is made of the number of flow in each, and by summation the number of values greater than a given upper limit of the classes can be determined. The number of flows greater than a given upper limit of the class can be divide by the total number of flow values in the data series to obtain the exceedence percentage. The value of the flow for the particular upper limit of the class interval is then plotted versus the computed exceedence percent (Warnick et al., 1984As sited in Nardos 2011).

Then after plotting FDC a flow value is selected to become a design flow for the generation of power. The flow rate value ($Q_{(\%daily)}$, $Q_{(\%weekly)}$,& $Q_{(\%monthly)}$,) taken for run-off-river scheme varies from literature to literature.

ii. Gross head & net Head

To estimate the hydropower potential of any site hydrological data and physical characteristic of the site should be known. Gross head is one of the physical characteristic of the site and measuring is carried out using different techniques. For this research work the gross head was read from the topographic map of scale 1:50000. Having established the gross head available, the head losses that include losses due to conveyance, plant losses such as entrance loss, rack loss, generator and turbine loss etc. were considered to get the net head

4.8 Comparison of Flow-duration curves from monthly and daily data

A flow-duration plotted using monthly average flow has a tendency towards a higher design discharge that sometimes produces quite larger errors. This is because the monthly average used will mask within-month variation. Consequently, it is necessary to obtain an average daily flow series, as far as possible, in order to build a daily flow duration curve (TongZheng Wang Hai ding-1996 as cited in Tamene 2004).

(Ramsahoye 1982 as cited in Tamene 2004) considers the determination of the average daily flows-duration curve as the unquestionably most important hydrological input for the dependent power, energy, and economic studies of hydropower project. Especially when it is desired to provide investors and lenders with certainty, conservative estimate of stream flow is considered appropriate. However, the analysis based on monthly data is useful to make preliminary insight in to the design. (Tamene 2004).

4.9 Transferring data to the potential site

To estimate the hydropower potential of a site, the availability of stream flow data recorded for a long time throughout each year of concern is very important. But it may not be true that the sites for the hydropower plant and that for the gauge station are exactly at the same place. Therefore, it is important to transfer the data of the gauging station to the site. Equation (17) is the common type of relation that is used to estimate flow duration at site (Gulliver and Roger-1991).

On the other hand the catchement area of our specific point shall be within acceptable range according to (Jiandong , 2000 as cited in Abebe , 2015)Small rivers are generally defined as rivers with catchment area of less than 500km^2, or in a precise sense, less than 300 Km^2. From the point of view of MHP development, we are interested rather in riverlets or streams with catchment areas of less than 100Km$^{2.}$

$$Q_{site} = [\frac{DA_{site}}{DA_{gauge}}]^c * Q_{gauge} \qquad \text{Equation (19)}$$

Where,

DA_{site}drainage area of the power plant site (m^2)

DA_{gauge}drainage area of the gauge (m^2)

Q_{site} discharge at the site (m^3/s)

C a parameter typically varies between 0.6 and 1.2

The thing here is the selection of the value of C. put this as follows:

1. If the drainage area (DA) of the site is within 20% of the drainage area of the gauge ($0.8 \leq \frac{DA_{site}}{DA_{gauge}} \leq 1.2$), use C=1. The estimated discharge at the site will probably be within 10% of the actual discharge, which is normally sufficient.
2. If the DA site is within 50% of the DA gauge, consider whether the data of the two gauges (upstream and downstream gauges, if any) can be combined. In addition, when a weighted average between upstream and downstream gauges is possible, the following linear interpolation (equation 18) may be applied for a site lying between upstream & downstream gauging station.

$$Q_{site} = \frac{(DA_{gauge1} - DA_{site})Q_{gauge1} + (DA_{site} - DA_{gauge2})Q_{gauge2}}{DA_{gauge1} - DA_{gauge2}} \qquad \text{Equation (20)}$$

> The daily flow data from the two gauges should be used to compile a new set of daily flow data for the site. A flow-duration curve is compiled from the new flow data. For this case, comparing watersheds may be helpful because Q_{site} may be of 30%. If there is a partial discharge station near the site, it can give an indication of the proper value of C. the ratio of partial discharge to gauge discharge on the same day versus gauge discharge is plotted. The average value may be used to estimate C. (Abebe, 2011)

As can be seen from the above equations, the drainage areas of both the gauge station and the plant site are required. For this purpose ArcGIS software and DEM data of the site found from Ministry of water, irrigation & energy is used. Using the hydrology tools of the spatial analyst toolbox of ArcGIS like Fill, Flow Accumulation, Flow Direction, Flow Length, Snap Power Point, Stream Link, Stream Order and watershed the drainage areas of both points at the gauge station and the site are found. Therefore, by using Equation (19), the stream flow data is mapped to the site.

Selected sites

Sites suitable for Micro hydropower development were selected considering different parameters mentioned in the previous section 4.5; Table 4.7 specifies their geographic location.

Flow head

Gross head is one of the physical characteristic of the site which is used to estimate the hydropower potential at the site, and measuring is usually carried out using different techniques, for this research work the gross head was read from the topographic map of scale 1:50000.

Table 4.7 Geographic location of the sites.

No	station identity	coordinates		Altitude (m.a.m.s.l)
		Latitude	Longitude	
1	H1	6.959^0	39.308^0	2433
2	H2	6.935^0	39.293^0	2572
3	H3	6.895^0	39.289^0	2825
4	M1	6.975^0	39.35^0	2437
5	M2	6.966^0	39.357^0	2450
6	M3	6.966^0	39.357^0	2478
7	M4	6.95^0	39.37^0	2585
8	M5	6.923^0	39.364^0	2699
9	M6	6.917^0	39.362^0	2789
10	M7	6.907^0	39.361^0	2890
11	N1	6.98^0	39.371^0	2436
12	N2	6.955^0	39.381^0	2575
13	N3	6.987^0	39.399^0	2779
14	L1	6.927^0	39.424^0	2743
15	L2	6.992^0	39.42^0	2742
16	F1	6.987^0	39.449^0	2546
17	F2	6.986^0	39.459^0	2627

Power potential

In Micro-Hydropower development there are two category of developer, the developer in category 1 is more interested in generating only what energy is needed and in having that energy available for as much of the year as possible. The developer is not interested in recovering the maximum energy available from the stream. As a result the system will be designed for the minimum stream flow of the year.

Table 4.8 Gross head for each site

no	points	facility type	Head
1	H1	Diversion & canal	40
2	H2	Diversion & canal	60
3	H3	Diversion & canal	140
4	M1	Diversion & canal	40
5	M2	Diversion & canal	40
6	M3	Diversion & canal	60
7	M4	Diversion & canal	60
8	M5	Diversion & canal	70
9	M6	Diversion & canal	70
10	M7	Diversion & canal	60
11	N1	Diversion & canal	40
12	N2	Diversion & canal	60
13	N3	Diversion & canal	60
14	L1	Diversion & canal	80
15	L2	Diversion & canal	100
16	F1	Diversion & canal	40
17	F2	Diversion & canal	60

The primary motivation of those in category 2 is to produce the maximum energy available from the stream for the money invested. Category 2 developers are normally interested in flows between 20 to 35% exceedance.

Category 1 is used in the case where the power required for the site is known, but in category 2 we don't know the power demand but just interested to determine the maximum potential, which is the case for this research. (U.S. Department of Energy, 1983)

Flow duration curve

Flow-duration curves (FDC) are used to summarize stream flow characteristics and can be constructed from daily, weekly, or monthly stream flow data, for this study the FDC is constructed from daily flow data, the reason has been discussed in the previous section 4.6. & Q_{35} is taken as design flow.

Table 4.9 Results of the flow duration curve that was constructed from daily flows

no	potential points	facility type	Q_{35}
1	H1	Diversion & canal	0.95
2	H2	Diversion & canal	0.88
3	H3	Diversion & canal	0.6
4	M1	Diversion & canal	3.06
5	M2	Diversion & canal	3.01
6	M3	Diversion & canal	3.00
7	M4	Diversion & canal	2.81
8	M5	Diversion & canal	2.77
9	M6	Diversion & canal	2.76
10	M7	Diversion & canal	2.74
11	N1	Diversion & canal	0.53
12	N2	Diversion & canal	0.52
13	N3	Diversion & canal	0.52
14	L1	Diversion & canal	1.08
15	L2	Diversion & canal	1.07
16	F1	Diversion & canal	1.75
17	F2	Diversion & canal	1.66

Calculating theoretical available power

Once the flow duration curve for the sites selected is established, the next step was estimation of potential power and energy. Before any power plant is contemplated it is essential to assess the inherent power available from the discharge of the river and the head available at the site. The theoretical potential power of a river can be expressed as:

$P = \gamma qgh$ Equation (21)

Where

$$\gamma - The\ unit\ weight\ of\ water = 1000\ {}^{kg}/_{m^3}$$

$$Q - the\ flow\ or\ the\ discharge\ the\ river\ is\ experiencing\ {}^{m^3}/_{s}$$

$$H - The\ drop\ or\ potential\ hed\ in\ m$$

This expression written in terms of horse power & kilo watt would be

$$P = 1000qh/75 = 13.33QH(hp)$$

$P = 0.736 * 13.33Qgh = 9.8QH\ (KW)$ Equation (22)

The actual use of the above equation is difficult due to the fact that the discharge of any river varies over a wide range. High discharges are available only for short durations in a year. The corresponding available power would be of short duration. Thus the available power obtained from a given river can be classified according to the following types.

(i) **Minimum potential** power computed from the minimum flow available 100 percent of time, i.e., for all 365 days or 8760 hours, P100

(ii) **Small potential** power computed from the flow available for 95 percent of time, i.e., flow available for 8322 hours, P95.

(iii) **Average potential** power computed from the flow available for 50 percent of time, i.e., flow available for 6 months or 4380 hours, P50.

(iv) **Mean potential** power computed from the average of mean yearly flows for a period of 10 to 30 years, which is equal to the area of the flow-duration curve corresponding to this mean year. This is known as Gross river power potential, represented as Pm.

(v) **Maximum Potential** Power computed from the flow available for 25 to 35 percent of time(U.S. Department of Energy, 1983).

It can be seen that the evaluation of the average flow is a complicated problem and will be close to the correct value only when obtained from an average flow duration curve based on a flow data of a longer period. For this study, a flow data of 31 years is used. It is hoped that the value obtained is close to the correct value although the basin is still suffering from loss of long historical data.

Technically available power is obtained by including losses due to conveyance, plant losses such as entrance loss, rack loss, generator and turbine loss etc. For MHP, the overall efficiency, η, of 50% is multiplied with the theoretical power to obtain technically available power. The low overall efficiency is as a result of the following losses (Harvey, 1998. as cited in Keneni 2007; John, 2014).

- ✓ Channel loss = 5%
- ✓ Penstock losses = 10%
- ✓ Turbine losses =20%
- ✓ Generator losses = 15.4%
- ✓ Step-up and down transformer losses = 4%
- ✓ Transmission losses = 10%

Power output is obtained after all these losses are considered.

$$power\ output = power\ input * conversion\ efficiency$$

$$power\ output = 0.95 * 0.9 * 0.8 * 0.84 * 0.96 * 0.9 * power\ input$$

$$power\ output = 0.5 * power\ input$$

Therefore, overall efficiency, η for MHP=0.5

$$Power\ output = 9.88\eta QH(KW)$$

$$P = 9.8 * 0.5QH(KW)$$

$P = 4.9QH(KW)$ Equation (23)

This is the technically available power the sites have.

Table 4.10Technically available power of the sites

no	points	facility type	Head	$Q_{35}(m^3/s)$	P(KW)
1	H1	Diversion & canal	40	0.95	186.20
2	H2	Diversion & canal	60	0.88	258.72
3	H3	Diversion & canal	120	0.60	352.80
4	M1	Diversion & canal	40	3.06	599.76
5	M2	Diversion & canal	40	3.01	589.96
6	M3	Diversion & canal	60	3.00	882.00
7	M4	Diversion & canal	40	2.81	550.76
8	M5	Diversion & canal	70	2.77	950.11
9	M6	Diversion & canal	70	2.76	946.68
10	M7	Diversion & canal	60	2.74	805.56
11	N1	Diversion & canal	40	0.53	103.88
12	N2	Diversion & canal	60	0.52	152.88
13	N3	Diversion & canal	60	0.52	152.88
14	L1	Diversion & canal	80	1.08	423.36
15	L2	Diversion & canal	80	1.07	419.44
16	F1	Diversion & canal	40	1.75	343.00
17	F2	Diversion & canal	40	1.66	325.36
				Total	**8314.32**

The technically available power potential of the sites M1,M2,M3,M4,M5,M6, & M7 is greater than 500KW, but Since we are interested in Micro-Hydropower we use cascade power development, by dividing the available head in to two(Table 4.11).

Table 4.11Technically available power of the cascade sites

no	points	facility type	Head	$Q_{35(m^3/s)}$	P(KW)
1	M11	Diversion & canal	20	3.06	299.88
2	M12	Diversion & canal	20	3.06	299.88
3	M21	Diversion & canal	20	3.01	294.98
4	M22	Diversion & canal	20	3.01	294.98
5	M31	Diversion & canal	30	3.00	441.00
6	M32	Diversion & canal	30	3.00	441.00
7	M41	Diversion & canal	20	2.81	275.38
8	M42	Diversion & canal	20	2.81	275.38
9	M51	Diversion & canal	35	2.77	475.06
10	M52	Diversion & canal	35	2.77	475.06
11	M61	Diversion & canal	35	2.76	473.34
12	M62	Diversion & canal	35	2.76	473.34
13	M71	Diversion & canal	30	2.74	402.78
14	M72	Diversion & canal	30	2.74	402.78

Maximum potential energy of each site is calculated using equation 24

$Emax = P * 8760KWh$ Equation (24)

Where P = power for the sites under consideration. (Keneni, 2007).

Table 4.12 Technically available potential energy of the sites

no	points	Emax(KWh)
1	H1	1631112
2	H2	2266387.2
3	H3	3090528
4	M1	5253897.6
5	M2	5168049.6
6	M3	7726320
7	M4	4824657.6
8	M5	9511958.4
9	M6	9477619.2
10	M7	7056705.6
11	N1	909988.8
12	N2	1339228.8
13	N3	1339228.8
14	L1	3708633.6
15	L2	3674294.4
16	F1	3004680
17	F2	2850153.6
Total		72833443.2

4.10 Ranking of sites

To facilitate a favorable condition for local decision makers to make a reasonable decision as to which sites should be given the top priority for future micro hydropower development project implementation, ranking of the sites is essential. Even though there are so many criteria that govern the ranking of the sites, due to limitations of data only head and discharge, ratio between waterway length (L) and total head (H), the site condition: accessibility, i.e., distance from access road and distance to the demand center, and the energy output have been considered.

1. Energy Output

The last outcome that is needed from a river site is the energy output. In order to determine the rank a site has with respect to the others, the ratio of the energy output of that site to the highest of the energy of all sites is considered. For example, the energy output of the site of H1 is 1.63 GWh/year and the highest energy output from all the sites is 8.85GWh/year. Therefore, the ratio of energy output of H1 site to that of highest energy of all sites is 1.63/9.51 = 0.171. This value is added to the ratios of other parameters to obtain the ranking value of the site of H1.

2. Design Discharge

One of the main and, in fact the key parameter determining the potential of a river is the stream flow or the discharge. This parameter is used along with the others to determine the rank of the sites. Like that of the energy output, the ratio of the design discharge of a site to that of the annual maximum discharge of all sites is obtained and added to the other ratios so that the ranking value is computed. The Design discharge of H1 is 0.95m3/s while that of the maximum is 3.06m3/s. Thus, the discharge ratio is 0.95/3.06m3/s = 0.31.

3. Head

The potential energy that makes the turbine to rotate while the water strikes it is as a result of the head above the turbine. Especially in Ethiopian condition the head plays a great role in amplifying the power. Therefore, head is taken as one of the parameters to evaluate the rank of the sites. The highest head of the sites under study is found to be about 120m. The head of site H1 is 40m. This makes the ratio to be about 0.333.

4. L/H [ratio between waterway length (L) and total head (H)]

The ratio between the total head (H) and the waterway length (L) (L/H) of a site is a key parameter on minimizing the cost of a micro hydropower. A site with a smaller L/H value is more advantageous for small-scale hydropower. Therefore, L/H is taken as one of the parameter to evaluate the rank of the sites. In this case the ratio is obtained taking in to account the smallest L/H, so the inverse ratio of the L/H of the site to that of the minimum of the all site is taken to add up with the rest ratio, for H1, L/H=9 & the minimum L/H of all the site is 0.875, so the ratio is 1/ (9/0.875)=0.0972.

5. Distance from Access Road

Unless there is all weathered road at least in the nearby surrounding, it is obviously difficult to carry out development projects. Thus, it is necessary that accessibility or distance to accessible road have some weight in determining the rank of a site. In our example the distance of site H1 from the nearby road is about 4km. this is the same as step no4. The site which is very near or adjacent to all weathered road is site N1 having a distance of 2.7km from the road. The inverse ratio of this distance of the site of N1 to the distances of other respective sites is taken in the evaluation of the rank. For H1 the ratio is 1/ (4/2.7) = 0.675.

6. Distance of the Sites to the Demand Center

This factor is also very influential in determining the rank of a site. The effect is similar to the factor described in number 5 above. As much as possible the end users should be nearer to the hydropower station since expensive transmission lines are not appreciated in MHP installation. H1 is the site which is very near to the demand site with 1.2km. So the inverse ratio becomes 1(1.21/1.21) =1. The sum of the above five ratios reveals the rank of the site of H1.

$R = R_e + R_q + R_h + R_{l/h} + R_{dar} + R_{ddc}$ Equation (25)

R=0.171+0.31+0.33+0.0972+0.675+1=2.583

Where

R_e:-ratio of energy output of a site to the maximum of energy output of all sites.

R_q:-ratio of the Design discharge of a site to the maximum Design discharge of all sites.

R_h:-inverse ratio of head of a site to the minimum head of all sites.

$R_{l/h}$:- inverse ratio of the lowest L/H(ratio between waterway length (L) and total head (H))to that of a site.

R_{dar}:- inverse ratio of the lowest distance of the sites to the road & the average of all(Distance between the site and the nearby road).

Rddc:-inverse ratio of the lowest distance of the sites to the site under consideration (distance between the site & the demand center)

The ranking of all other sites is performed in similar manner. According to the result of the ranking process, the site M1 is found to score the first while H2 is at the bottom with its very low rank.

Table 4.13Ranking parameter

no	points	Head	Emax(GWh)	$Q_{35(m^3/s)}$	L/H	DA R * (Km)	D DC* (Km)
1	H1	40	1.63	0.95	9	4	1.21
2	H2	60	2.27	0.88	6.433333	6.65	3.66
3	H3	120	3.09	0.6	1.716667	11.1	7.96
4	M1	40	5.25	3.06	5.875	3.047	1.45
5	M2	40	5.17	3.01	4.975	4.28	3.04
6	M3	60	7.73	3.00	5.4	4.28	3.04
7	M4	40	4.82	2.81	2.625	6.605	1.42
8	M5	70	9.51	2.77	3.9	8.73	2.46
9	M6	70	9.48	2.76	1.7	9.317	3.7
10	M7	60	7.06	2.74	1.616667	10.34	3.48
11	N1	40	0.91	0.53	2.125	2.7	2.7
12	N2	60	1.34	0.52	3.333333	5.4	1.73
13	N3	60	1.34	0.52	1.883333	9.18	2.55
14	L1	80	3.71	1.08	0.875	9.25	3.28
15	L2	80	3.67	1.07	2.1375	10.19	3.8
16	F1	40	3.00	1.75	4.55	4	5.46
17	F2	40	2.85	1.66	3.9	4.5	5.8
		120	**9.51**	**3.06**	**0.875**	**2.7**	**1.21**
		Max			**Min**		

DAR* distance from access road
DDC** distance from demand center

Table 4.14 Ranking of the site

no	points	Re	Rq	Rh	RL/H	Rdar	Rddc	Rank		Demand center
1	H1	0.171	0.310	0.333	0.097	0.675	1.000	2.587	11	Keta
2	H2	0.238	0.288	0.500	0.136	0.406	0.331	1.898	17	Keta
3	H3	0.325	0.196	1.000	0.510	0.243	0.152	2.426	12	berenda
4	**M1**	**0.552**	**1.000**	**0.333**	**0.149**	**0.886**	**0.834**	**3.755**	**1**	**Keta**
5	M2	0.543	0.984	0.333	0.176	0.631	0.398	3.065	7	Keta
6	M3	0.812	0.980	0.500	0.162	0.631	0.398	3.484	4	Keta
7	M4	0.507	0.918	0.333	0.333	0.409	0.852	3.353	5	Chumlugo
8	M5	1.000	0.905	0.667	0.224	0.309	0.492	3.597	3	Chumlugo
9	M6	0.996	0.902	0.667	0.515	0.290	0.327	3.697	2	berenda
10	M7	0.742	0.895	0.500	0.541	0.261	0.348	3.287	6	berenda
11	N1	0.195	0.353	0.333	0.412	1.000	0.448	2.741	9	Ejersa
12	N2	0.290	0.350	0.500	0.263	0.500	0.699	2.601	10	Chumlugo
13	N3	0.257	0.310	0.500	0.465	0.294	0.475	2.301	14	Chumlugo
14	L1	0.318	0.288	0.667	1.000	0.292	0.369	2.933	8	Bucha
15	L2	0.217	0.196	0.667	0.409	0.265	0.318	2.072	16	Bucha
16	F1	0.316	0.572	0.333	0.192	0.675	0.222	2.310	13	Bucha
17	F2	0.300	0.542	0.333	0.224	0.600	0.209	2.208	15	Bucha

Map of the sites

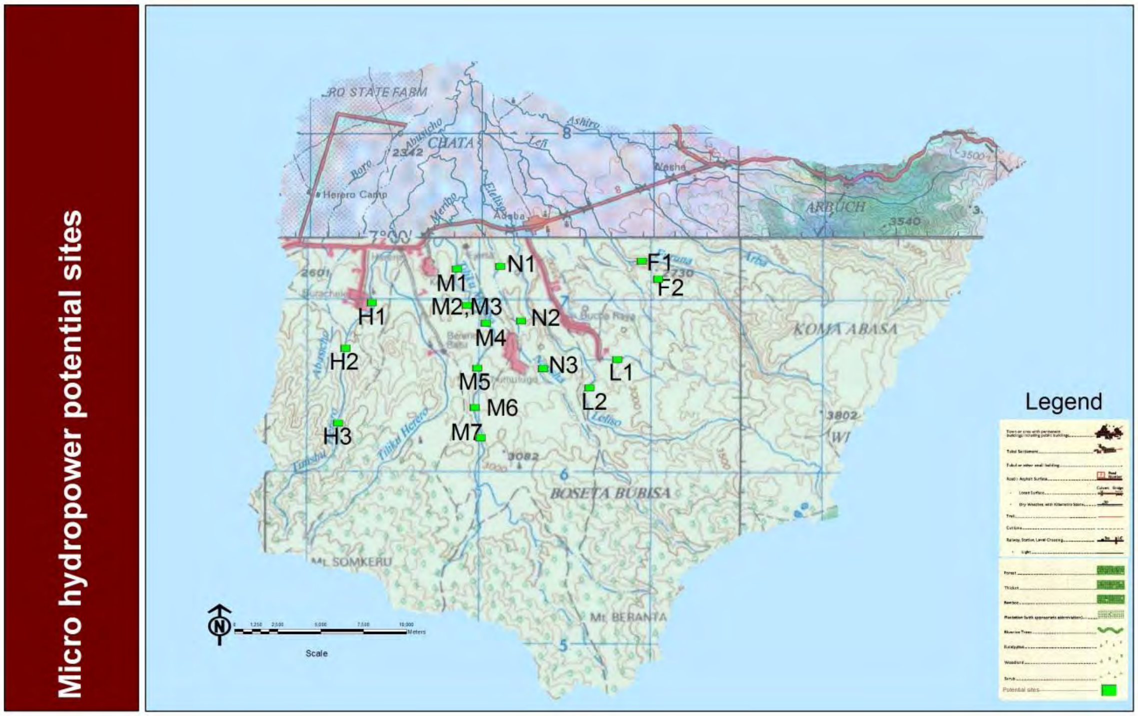

Figure 4.7 GIS based map of the potentially viable sites

Chapter 5

5. Conclusions& Recommendation

5.1 Conclusions

Ethiopia is blessed with a substantial amount of water resources potential and yet a nation where vast hydro power resources are still untapped. Due to the use of traditional energy system, deterioration of the environment is increasing from time to time. In view of the economic and social development, the stabilization of sustainable energy supply to the rural and urban area throughout the nation will keep the environment safe & additionally promote the modernization of peoples living in rural areas. Therefore, to bring into effect the poverty reductions at national level and to speed up the economic development. The Government has to initiate study & further action on Micro hydropower projects.

In this thesis the Micro hydropower potential of the selected sites of the Adaba sub catchment were assessed& the following are the summery

- ✓ To assess the potentials of these sites both primary data from field survey and secondary data from different institutions were gathered, analyzed and used.
- ✓ According to this study there are 17 sites which are identified as Micro hydropower potential sites with total potential capacity of 8.04MW.
- ✓ The type of hydropower schemes which is suitable for all the sites is Diversion & canal type hydropower plant.
- ✓ Rural villages in Ethiopia are very scattered, therefore, difficult to supply them power from the national grid. As we can see from this study MHP plants are very convenient in this regard.
- ✓ The environment in danger of deterioration could get solution by applying different viable sustainable energy systems such as MHP.

5.2 Recommendation

In order to boost the Micro Hydropower capacity in Adaba sub catchment& generally in Ethiopia, improvements can be made in the following areas:

- ✓ Looking at the enormous benefits of electrifying rural villages by SHP installation, it is recommended that the government of Ethiopia should give attention to promote the development of SHP plants.
- ✓ Private investors should be encouraged in various ways to pursue SHP development.
- ✓ High initial cost need to be overcome with easier/improved access to finance for project developments. Awareness of small hydropower should be raised among local banking institutions or micro- finance institutions in order to improve risk assessment and provide attractive loan conditions.
- ✓ Responsible parties shall do researches on Design and manufacture of low cost turbines.
- ✓ Relatively low return on investment discourages individual private investment in small hydropower, but cooperatives with members that will benefit from getting access to electricity may be potential developers, since their primary motive is not return on investment.
- ✓ Competitive water uses and demand may prevent small hydropower development. An increasing population could create more demand for water by upstream users. So integrated water resource management shall be considered early.
- ✓ Base flow of perennial rivers shall be measured.
- ✓ Last but not the least, there is the need to do more detailed hydrological, topographical and geological studies at each identified site. A more rigorous analysis of energy consumption and future demand would be useful.

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Appendix

Appendix A: Sample Data

A1 Sample daily flow at Maribo Gaging station

Annual Report of Daily Data: Instantaneous Daily Flow

Station Number : 061002 Year: 1984

Station Name : Maribo nr. Adaba

Time-Series Type : Flow (cumecs)

Latitude : 7: 0: 0 N Longitude : 39:20: 0 E Area : 185.0 sq km

date	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1	0.472	0.269	0.209	0.205	0.295	4.54	2.27	6.657	7.23	5.28	2.509	0.511
2	0.456	0.269	0.236	0.205	0.265	3.04	2.23	6.17	6.6	4.55	2.377	0.5
3	0.42	0.269	0.265	0.209	0.205	2.33	2.33	10.24	7.77	3.93	2.204	0.467
4	0.389	0.265	0.269	0.228	0.162	2.49	2.74	13.95	6.62	3.26	2.058	0.456
5	0.414	0.236	0.265	0.209	0.197	2.49	3.13	11.22	6.09	2.82	2.027	0.42
6	0.414	0.209	0.236	0.205	0.201	2.94	2.99	10.72	8.18	2.39	1.927	0.379
7	0.379	0.205	0.209	0.205	0.179	2.51	2.57	9.313	7.8	2.07	1.693	0.345
8	0.345	0.205	0.205	0.205	0.176	2.17	2.27	7.839	7.3	1.93	1.731	0.341
9	0.341	0.205	0.201	0.205	0.179	1.74	2.33	7.654	8.92	1.68	1.847	0.341
10	0.341	0.205	0.179	0.201	0.379	1.44	2.22	8.155	10.2	1.62	1.818	0.35
11	0.341	0.205	0.172	0.179	1.688	1.23	2.15	7.017	9.43	1.44	1.867	0.431
12	0.341	0.205	0.152	0.179	1.669	1.06	2.23	8.155	8.3	1.32	1.838	0.581
13	0.341	0.205	0.148	0.209	2.146	1.03	2.01	7.876	8.51	1.5	1.6	0.629
14	0.341	0.205	0.148	0.269	1.604	0.98	2.06	9.331	9.43	2.68	1.509	0.593
15	0.341	0.205	0.166	0.362	1.5	1.03	2.55	10.32	9.13	3.98	1.473	0.764
16	0.336	0.205	0.287	0.564	1.584	1.76	4.67	10.83	8.63	3.06	1.3	0.743
17	0.304	0.205	0.327	0.581	2.112	2.24	6.03	8.622	9.27	2.56	1.16	0.647
18	0.273	0.205	0.278	0.473	2.122	4.21	6.65	6.772	8.72	2.18	1.035	0.551
19	0.269	0.205	0.269	0.415	1.621	4.68	7.05	6.51	7.23	1.7	0.924	0.467
20	0.269	0.205	0.265	0.341	1.36	3.81	7.1	6.375	6.13	1.56	0.852	0.43
21	0.269	0.205	0.236	0.273	1.161	3.33	6.43	5.981	7.23	1.43	0.757	0.451
22	0.269	0.205	0.209	0.236	0.983	3.59	6.18	4.8	7.12	1.28	0.704	0.435
23	0.269	0.205	0.221	0.209	0.866	3.8	7.7	3.967	7.3	1.11	0.691	0.478
24	0.269	0.205	0.318	0.201	0.846	4.29	7.93	3.845	9.1	1.05	0.653	0.399
25	0.269	0.205	0.304	0.179	1.043	4.09	7.08	3.297	7.36	1.12	0.647	0.409
26	0.269	0.205	0.269	0.172	1.028	3.33	6.98	3.145	5.86	1.3	0.641	0.379
27	0.269	0.205	0.236	0.152	1.242	2.6	7.58	3.245	4.62	1.25	0.598	0.341
28	0.269	0.205	0.209	0.155	1.25	2.24	9.86	3.58	4.06	1.69	0.557	0.308
29	0.269	0.205	0.205	0.205	1.405	2.14	9.35	4.425	3.92	2	0.551	0.299

30	0.269		0.205	0.265	2.235	2.2	9.93	6.35	5.32	2.08	0.545	0.273
31	0.269		0.205		5.054		8.62	8.52		2.36		0.269
Mean	0.325	0.215	0.229	0.256	1.186	2.64	5.01	7.254	7.44	2.2	1.336	0.451
Flow (MCM)	0.871	0.538	0.613	0.665	3.176	6.85	13.4	19.43	19.3	5.89	3.464	1.209
Maximum	0.472	0.269	0.327	0.581	5.054	4.68	9.93	13.95	10.2	5.28	2.509	0.764
Minimum	0.269	0.205	0.148	0.152	0.162	0.98	2.01	3.145	3.92	1.05	0.545	0.269
Runoff (mm)	4.709	2.908	3.316	3.593	17.17	37	72.5	105	104	31.8	18.725	6.533

A2 Sample daily flow at Maribo Changuity station

Annual Report of Daily Data: Instantaneous Daily Flow

Station Number : 061015 Year: 1984

Station Name : Maribo Nr. Changuity (Closed)

Time-Series Type : Flow (cumecs)

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1	0.418	0.218	0.185	0.185	0.201	0.8	5.46	7.194	5.16	7.19	3.103	0.505
2	0.391	0.201	0.185	0.185	0.201	1.03	3.91	6.288	12.5	5.95	2.793	0.505
3	0.391	0.201	0.185	0.185	0.185	0.93	2.7	8.606	10.7	5.46	2.414	0.446
4	0.341	0.201	0.185	0.185	0.155	1.03	2.33	12.8	8.82	4.17	2.505	0.446
5	0.341	0.185	0.185	0.185	0.218	1.36	2.41	12.8	7.19	3.55	2.505	0.446
6	0.341	0.185	0.185	0.185	0.296	1.19	2.51	11.45	11.2	2.89	2.325	0.391
7	0.318	0.185	0.155	0.255	0.296	1.76	2.33	11.71	9.72	2.51	2.154	0.391
8	0.296	0.185	0.155	0.255	0.236	1.91	2.41	8.394	10.2	2.6	1.914	0.341
9	0.296	0.185	0.155	0.201	0.218	8.19	2.79	8.394	12.8	1.99	2.894	0.341
10	0.296	0.185	0.155	0.201	0.218	1.13	3.67	7.194	14.3	1.99	2.505	0.341
11	0.296	0.185	0.142	0.185	0.218	3.67	3.55	6.822	13.1	1.69	2.325	0.391
12	0.296	0.185	0.129	0.185	0.218	2.51	3	9.723	10.9	1.49	2.414	0.716
13	0.296	0.185	0.129	0.185	0.201	2.79	2.41	9.492	10.7	2.33	1.838	0.677
14	0.275	0.185	0.129	0.185	0.155	3.44	2.6	8.394	12.8	4.44	1.692	0.98
15	0.275	0.185	0.155	0.185	0.155	3.55	2.33	10.68	13.1	5.62	1.555	1.245
16	0.275	0.169	0.341	0.169	0.236	2.7	2.24	13.08	11.4	3.67	1.622	0.756
17	0.255	0.169	0.275	0.201	1.692	2.51	2.41	13.37	17.2	3.21	1.303	0.756
18	0.255	0.169	0.275	0.296	1.245	1.84	2.07	8.822	12.2	2.6	1.08	0.505
19	0.255	0.169	0.255	0.255	3.103	1.43	2.24	8.185	9.72	2.07	0.885	0.446
20	0.255	0.169	0.255	0.365	1.838	1.19	4.04	7.98	9.96	1.76	0.885	0.446
21	0.255	0.169	0.218	0.505	1.489	1.08	6.46	7.194	11.2	1.69	0.798	0.446
22	0.255	0.169	0.201	0.677	1.692	1.08	9.27	5.622	10.2	1.43	0.716	0.446
23	0.255	0.155	0.185	0.537	1.555	0.93	9.49	4.437	12.2	1.36	0.716	0.446
24	0.236	0.169	0.185	0.446	2.793	1.36	10.7	4.716	13.7	1.43	0.677	0.391
25	0.236	0.169	0.218	0.446	2.695	2.07	9.96	5.155	10.7	1.69	0.677	0.365
26	0.236	0.169	0.318	0.341	2.072	4.58	9.72	5.006	7.58	1.56	0.677	0.365
27	0.236	0.169	0.255	0.296	1.622	5.95	9.27	5.155	5.95	1.99	0.57	0.341
28	0.236	0.169	0.218	0.236	1.363	3.91	8.39	5.308	5.31	2.7	0.57	0.318
29	0.236	0.185	0.201	0.218	1.188	4.72	11.2	8.606	10.4	2.6	0.57	0.296
30	0.218		0.201	0.201	1.029	5.01	13.1	10.68	7.19	2.51	0.537	0.275
31	0.218		0.185		0.841		8.39	7.194		2.33		0.255
Mean	0.283	0.18	0.2	0.271	0.956	2.52	5.27	8.401	10.6	2.85	1.574	0.484

Flow (MCM)	0.758	0.451	0.535	0.703	2.559	6.53	14.1	22.5	27.5	7.64	4.08	1.297
Maximum	0.418	0.218	0.341	0.677	3.103	8.19	13.1	13.37	17.2	7.19	3.103	1.245
Minimum	0.218	0.155	0.129	0.169	0.155	0.8	2.07	4.437	5.16	1.36	0.537	0.255
Runoff (mm)	0	0	0	0	0	0	0	0	0	0	0	0

A3 Sample daily flow at Furuna Gaging station

Annual Report of Daily Data: instantaneous Daily Flow

Station Number : 061016

Station Name : Furuna at adaba

Time-Series Type : Flow (cumecs)

Area : 7.5 sq km

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1	-	-	-	-	5.307	2.03	2.15	5.305	3.21	2.55	2.542	2.152
2	-	-	-	-	4.507	2.03	2.17	4.771	3.12	2.55	2.482	2.16
3	-	-	-	-	4.05	2.03	2.29	4.984	2.99	2.53	2.414	2.208
4	-	-	-	-	3.763	2.03	2.45	4.984	2.91	2.43	2.355	2.208
5	-	-	-	2.152	3.466	2.02	2.42	4.61	2.84	2.41	2.347	2.16
6	-	-	-	2.152	3.278	1.97	2.41	4.008	2.83	2.41	2.355	2.192
7	-	-	-	2.359	3.048	1.97	2.41	3.773	2.77	2.35	2.414	2.583
8	-	-	-	3.702	2.78	2.03	2.35	3.607	2.77	2.29	2.465	3.605
9	-	-	-	4.063	2.629	2.14	2.3	3.496	2.81	2.28	2.414	4.225
10	-	-	-	4.832	2.559	2.2	2.36	3.711	2.69	2.28	2.355	4.207
11	-	-	-	4.974	2.542	2.16	2.53	4.028	2.57	2.28	2.347	3.263
12	-	-	-	4.184	2.49	2.15	2.62	3.986	2.55	2.27	2.339	2.991
13	-	-	-	3.681	2.499	2.15	2.72	4.575	2.57	2.22	2.289	2.907
14	-	-	-	3.24	2.629	2.15	3.06	5.334	2.66	2.22	2.281	2.852
15	-	-	-	2.973	2.825	2.15	4.01	8.487	2.63	2.22	2.281	2.907
16	-	-	-	2.717	2.944	2.15	5.55	7.534	2.62	2.22	2.281	2.954
17	-	-	-	2.62	2.816	2.16	7.21	6.294	2.65	2.29	2.273	2.871
18	-	-	-	2.568	2.577	2.21	7.29	6.091	2.83	2.41	2.224	2.963
19	-	-	-	2.62	2.473	2.21	7	6.035	2.94	2.55	2.216	3.019
20	-	-	-	2.727	2.355	2.16	8.92	4.404	2.84	2.67	2.216	3.349
21	-	-	-	3.029	2.289	2.15	13.2	3.805	2.76	2.69	2.208	3.944
22	-	-	-	3.259	2.273	2.15	15.6	3.628	2.7	2.69	2.16	3.629
23	-	-	-	3.68	2.224	2.15	16.2	3.597	2.69	2.67	2.152	3.307
24	-	-	-	3.638	2.208	2.18	15.9	3.526	2.69	2.57	2.152	3.152
25	-	-	-	3.955	2.16	2.38	13.8	3.506	2.68	2.54	2.152	3.132
26	-	-	-	6.001	2.152	2.4	10.8	3.794	2.64	2.48	2.152	3.123
27	-	-	-	8.77	2.144	2.29	10.5	3.783	2.68	2.41	2.152	3.066
28	-	-	-	13.26	2.097	2.22	11.2	3.731	2.67	2.36	2.152	3.047
29	-		-	10.12	2.089	2.21	10.1	3.466	2.57	2.36	2.152	2.972
30	-		-	6.891	2.089	2.16	7.77	3.307	2.55	2.42	2.152	2.834
31	-		-		2.081		6.45	3.277		2.53		2.708

Mean	-	-	-	-	2.753	2.15	6.64	4.498	2.75	2.43	2.282	2.99
Flow (MCM)	-	-	-	-	7.374	5.57	17.8	12.05	7.12	6.49	5.916	8.009
Maximum	-	-	-	-	5.307	2.4	16.2	8.487	3.21	2.69	2.542	4.225
Minimum	-	-	-	-	2.081	1.97	2.15	3.277	2.55	2.22	2.152	2.152
Runoff (mm)	-	-	-	-	983.1	742	2372	1606	950	866	788.79	1068

A4 Sample daily flow at Leliso Gaging station
Annual Report of Daily Data: instantaneous Daily Flow
Station Number : 061001
Station Name : Lelisso at Adaba
Time-Series Type : Flow (cumecs)

Latitude : 7: 0: 0 N Area : 135.0 sq km

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1	1.014	0.271	1.101	0.136	1.286	0.14	0.2	0.932	2.19	0.52	0.232	0.136
2	0.707	0.197	1.385	0.11	1.94	0.14	0.23	1.191	1.71	0.64	0.197	0.136
3	1.101	0.197	1.385	0.067	1.94	0.11	0.31	1.014	1.59	0.78	0.165	0.11
4	0.778	0.232	1.705	0.05	2.064	0.11	0.14	0.778	1.29	1.49	0.165	0.087
5	0.64	0.232	4.073	0.11	2.747	0.23	0.2	1.014	1.1	1.94	0.165	0.087
6	0.516	0.11	3.714	0.313	2.324	0.41	1.59	1.101	0.93	1.49	0.165	0.087
7	0.516	0.11	1.94	0.932	1.94	0.17	1.59	1.385	0.78	1.71	0.165	0.087
8	0.516	0.11	2.747	2.602	1.705	0.14	0.78	1.191	1.71	2.6	0.136	0.087
9	0.46	0.11	3.051	2.602	1.487	0.14	1.29	1.101	1.82	1.94	0.136	0.087
10	0.408	0.313	2.602	3.714	1.286	0.11	1.19	1.385	2.06	1.29	0.136	0.087
11	0.408	0.232	2.461	3.051	1.014	0.11	0.85	1.191	2.6	1.1	0.136	0.087
12	0.516	0.165	3.051	5.697	0.932	0.09	0.71	1.385	1.59	0.78	0.136	0.067
13	0.46	0.165	4.451	7.118	0.778	0.09	0.52	1.487	2.75	0.78	0.136	0.067
14	0.408	0.197	5.697	-	0.64	0.09	1.29	2.192	2.75	0.78	0.136	0.067
15	0.313	0.313	4.451	-	0.516	0.09	0.64	2.461	3.05	0.58	0.11	0.067
16	0.271	0.408	3.051	-	0.46	0.09	0.52	2.064	1.94	0.58	0.165	0.067
17	0.232	0.707	3.051	-	0.359	0.09	0.41	1.705	1.59	0.52	0.165	0.05
18	0.197	0.932	2.461	-	0.313	0.09	0.36	1.385	1.39	0.41	0.165	0.05
19	0.165	0.64	2.461	7.118	0.313	0.07	0.31	1.94	1.19	0.52	0.11	0.05
20	0.165	0.516	4.847	4.847	0.313	0.07	0.93	2.747	1.01	0.52	0.11	0.05
21	0.11	0.408	5.697	2.747	0.232	0.09	0.71	1.82	1.01	0.36	0.11	0.05
22	0.165	0.46	4.451	1.94	0.165	0.11	0.64	2.064	1.01	0.41	0.11	0.05
23	0.165	0.64	2.461	1.385	0.197	0.09	0.52	2.324	0.85	0.41	0.087	0.067
24	0.136	0.932	1.94	1.101	0.197	0.09	0.46	2.192	0.85	0.31	0.087	0.067

25	0.232	0.778	1.94	0.932	0.197	0.07	0.41	2.461	0.78	0.27	0.087	0.067
26	0.232	4.073	1.594	0.778	0.197	0.07	0.41	2.602	0.46	0.41	0.087	0.067
27	0.197	0.778	1.286	0.64	0.165	0.09	0.93	2.461	0.46	0.36	0.087	0.067
28	0.197	0.778	0.707	0.778	0.165	0.09	0.64	2.461	0.58	0.31	0.11	0.067
29	0.313		0.408	0.932	0.165	0.09	1.39	2.747	0.58	0.31	0.11	0.067
30	0.313		0.197	0.707	0.165	0.09	1.1	2.897	0.52	0.23	0.136	0.067
31	0.232		0.197		0.136		0.93	2.461		0.23		0.067

Mean	0.39	0.536	2.599	-	0.849	0.11	0.72	1.811	1.41	0.79	0.134	0.075
Flow (MCM)	1.044	1.296	6.96	-	2.275	0.29	1.92	4.85	3.64	2.12	0.348	0.2
Maximum	1.101	4.073	5.697	-	2.747	0.41	1.59	2.897	3.05	2.6	0.232	0.136
Minimum	0.11	0.11	0.197	-	0.136	0.07	0.14	0.778	0.46	0.23	0.087	0.05
Runoff (mm)	7.732	9.6	51.56	-	16.85	2.16	14.2	35.93	27	15.7	2.58	1.478

A5 Sample daily flow at Herero Gaging station

Annual Report of Daily Data: instantaneous daily Flow
Station Number : 061014
Station Name : harero at harero
Time-Series Type : Flow (cumecs)

Latitude : 7: 0: 0 N   Elevation : 2926.0 metres   Area : 133.0 sq km

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1	0.38	0.092	4.612	5.183	1.646	0.05	0.1	0.898	12.8	2.28	0.012	0.012
2	0.38	0.184	3.82	15.71	1.712	0.05	0.24	0.999	12.3	1.78	0.012	0.012
3	0.342	0.342	3.104	19.83	1.334	0.12	0.27	4.421	7.18	1.33	0.012	0.012
4	0.306	0.306	3.573	23.17	1.052	0.05	0.58	5.274	4.84	1	0.012	0.006
5	0.273	0.241	3.218	16.76	0.665	0.05	0.1	3.827	3.27	0.85	0.012	0.006
6	0.273	0.184	2.364	10.55	0.581	0.05	0.12	2.924	2.44	0.67	0.012	0.006
7	0.306	0.211	2.364	7.098	0.465	0.05	0.17	2.756	2.13	0.62	0.006	0.006
8	0.273	0.306	4.076	40.81	0.709	0.05	0.14	2.358	1.71	0.8	0.002	0.002
9	0.241	0.551	3.453	23.5	0.665	0.12	0.17	3.637	4.02	0.9	0.006	0.002
10	0.211	0.649	2.992	14.45	0.541	0.05	0.21	3.095	5.27	0.39	0.006	0.006
11	0.184	0.461	2.364	6.434	0.394	0.04	0.17	3.183	3.45	0.33	0.006	0.012
12	0.211	0.38	1.987	3.923	0.328	0.04	0.17	2.839	2.28	0.27	0.029	0.012
13	0.184	0.306	1.333	2.924	0.297	0.03	0.17	3.272	1.58	0.21	0.012	0.012
14	0.184	0.306	0.701	1.646	0.213	0.03	0.19	2.281	1.46	0.19	0.012	0.012
15	0.184	0.306	0.649	1.275	0.188	0.03	0.12	2.281	1.28	0.17	0.012	0.006
16	0.158	0.461	0.599	0.849	0.188	0.03	0.12	1.987	1.16	0.17	0.029	0.002
17	0.158	0.461	0.701	0.581	0.165	0.03	0.27	1.581	1.28	0.12	0.029	0.006

18	0.158	3.82	0.461	0.394	0.122	0.03	0.21	1.334	1.16	0.12	0.02	0.002
19	0.158	2.266	0.42	0.394	0.085	0.03	0.21	2.281	1.05	0.1	0.02	0.002
20	0.158	1.898	0.306	0.328	0.085	0.04	0.21	1.847	0.9	0.1	0.02	0.002
21	0.158	2.565	0.461	0.328	0.054	0	0.24	1.455	1.65	0.1	0.012	0.002
22	0.158	2.266	1.055	0.268	0.054	0.02	0.43	1.987	1.99	0.09	0.012	0.002
23	0.158	2.463	2.463	0.213	0.054	0.02	0.47	1.712	1.78	0.07	0.012	0.002
24	0.134	1.811	2.266	0.188	0.054	0.02	0.39	1.455	1.46	0.05	0.012	0.002
25	0.134	2.078	1.407	0.297	0.188	0.01	0.36	1.218	1.85	0.09	0.012	0.002
26	0.134	1.898	1.122	0.268	0.122	0	0.33	1.275	1.52	0.03	0.006	0.002
27	0.134	3.696	0.929	0.213	0.054	0	0.27	1.161	1.28	0.03	0.006	0.002
28	0.112	6.426	0.755	0.898	0.054	0.01	0.21	0.948	1.22	0.02	0.006	0.002
29	0.112		1.562	0.849	0.054	0.01	0.21	0.754	1.71	0.01	0.012	0.002
30	0.092		2.078	0.801	0.054	0.01	0.47	1.394	2.92	0.01	0.012	0.002
31	0.092		2.266		0.054		0.85	4.319		0.01		0.002

Mean	0.198	1.319	1.918	6.671	0.394	0.04	0.26	2.283	2.96	0.42	0.013	0.005
Flow (MCM)	0.53	3.191	5.138	17.29	1.057	0.09	0.71	6.113	7.68	1.12	0.033	0.013
Maximum	0.38	6.426	4.612	40.81	1.712	0.12	0.85	5.274	12.8	2.28	0.029	0.012
Minimum	0.092	0.092	0.306	0.188	0.054	0	0.1	0.754	0.9	0.01	0.002	0.002
Runoff (mm)		3.988	24	38.63	130	7.94	0.71	5.3	46	57.7	8.383	0.245

Appendix B: Data analysis

B1: sample Areal precipitation for the year 1984

			Gasara			Adaba			Hunte			Meskel darkina			
Year	Month	Day	Pcp.	contributing Area	A1*P1	Pcp.	contributing Area	A2*P2	Pcp.	contributing Area	A3*P3	Pcp.	contributing Area	A4*P4	Areal precipitatin
1984	1	1	0	0	0	0	670	0	0	86.17	0	0	279	0	0
1984	1	2	0	0	0	0.95	670	638	0.68	86.17	58.8	0.2	279	50.5	0.722
1984	1	3	0	0	0	1.05	670	702	0.37	86.17	31.5	0.2	279	59.2	0.766
1984	1	4	0	0	0	0.18	670	123	0	86.17	0	0.1	279	27.9	0.146
1984	1	5	0	0	0	0.29	670	194	0.16	86.17	13.5	0.6	279	178	0.372
1984	1	6	0	0	0	0.75	670	501	0.3	86.17	26.2	0	279	0	0.509
1984	1	7	0	0	0	0.29	670	194	1.15	86.17	99.3	0.2	279	41.8	0.324
1984	1	8	0	0	0	0.16	670	106	0.62	86.17	53.2	0	279	0	0.154
1984	1	9	0	0	0	1.63	670	1090	0.65	86.17	56.2	0	279	6.97	1.114
1984	1	10	0	0	0	3.12	670	2091	0.56	86.17	48.3	0	279	10.5	2.077
1984	1	11	0	0	0	0.69	670	465	1.18	86.17	102	2.6	279	711	1.235
1984	1	12	0	0	0	0.77	670	518	0.87	86.17	74.6	1.9	279	516	1.071
1984	1	13	0	0	0	0.82	670	547	1.04	86.17	89.9	0.5	279	145	0.755
1984	1	14	0	0	0	0.08	670	52.9	0.69	86.17	59.6	0.6	279	165	0.269
1984	1	15	0	0	0	0.74	670	497	0.34	86.17	29.2	1.4	279	401	0.896
1984	1	16	0	0	0	0.32	670	215	1	86.17	85.8	0.6	279	169	0.454
1984	1	17	0	0	0	0.85	670	568	0.9	86.17	77.9	0.6	279	164	0.782
1984	1	18	0	0	0	0.72	670	480	1.53	86.17	132	0.4	279	105	0.691
1984	1	19	0	0	0	0.28	670	190	0.52	86.17	45	2	279	557	0.766
1984	1	20	0	0	0	0.37	670	247	0.37	86.17	31.5	0.6	279	178	0.441
1984	1	21	0	0	0	0.51	670	342	0.34	86.17	29.6	0.6	279	165	0.519
1984	1	22	0	0	0	0	670	0	0.43	86.17	37.5	1.2	279	321	0.346
1984	1	23	0	0	0	0.41	670	272	1.24	86.17	107	0.7	279	183	0.542
1984	1	24	0	0	0	0.26	670	173	0.23	86.17	19.5	0.2	279	52.3	0.236
1984	1	25	0	0	0	0.15	670	98.7	1.23	86.17	106	0.8	279	225	0.415
1984	1	26	0	0	0	0.41	670	275	1.54	86.17	133	0.5	279	146	0.535
1984	1	27	0	0	0	0.83	670	554	0.7	86.17	60.3	0.8	279	228	0.814
1984	1	28	0	0	0	0	670	0	0.71	86.17	61.1	0	279	0	0.059
1984	1	29	0	0	0	0	670	0	0.78	86.17	67.4	0	279	0	0.065
1984	1	30	0	0	0	0	670	0	1.8	86.17	155	0	279	0	0.15
1984	1	31	0	0	0	0	670	0	1.67	86.17	144	0.1	279	40.1	0.178
1984	2	1	0	0	0	0.38	670	256	0.57	86.17	48.7	0.7	279	206	0.493
1984	2	2	0	0	0	1.08	670	721	0.73	86.17	63.3	1	279	285	1.034
1984	2	3	0	0	0	1.69	670	1135	1.42	86.17	122	0.9	279	238	1.445
1984	2	4	0	0	0	0.9	670	603	0.71	86.17	61.4	1.6	279	437	1.065

1984	2	5	0	0	0	1.05	670	702	0.94	86.17	80.9	1.4	279	382	1.125
1984	2	6	0	0	0	1.49	670	1001	0.36	86.17	30.7	0	279	2.14	0.999
1984	2	7	0	0	0	0.68	670	453	4.17	86.17	359	0.4	279	111	0.893
1984	2	8	0	0	0	1.95	670	1305	0.63	86.17	53.9	0.8	279	225	1.53
1984	2	9	0	0	0	1.82	670	1222	2.48	86.17	214	2.3	279	652	2.017
1984	2	10	0	0	0	1.22	670	820	2.52	86.17	217	2.2	279	617	1.599
1984	2	11	0	0	0	1.49	670	997	0.64	86.17	55.4	2	279	553	1.552
1984	2	12	0	0	0	2.66	670	1785	0.81	86.17	70.1	3.3	279	926	2.688
1984	2	13	0	0	0	0.7	670	469	1.13	86.17	97	1.8	279	508	1.038
1984	2	14	0	0	0	0.95	670	638	1.39	86.17	120	2.4	279	667	1.377
1984	2	15	0	0	0	0.65	670	434	0.2	86.17	17.6	2.9	279	813	1.221
1984	2	16	0	0	0	0.93	670	623	0.03	86.17	2.25	1.8	279	489	1.076
1984	2	17	0	0	0	1.34	670	895	0.98	86.17	84.3	1.3	279	354	1.288
1984	2	18	0	0	0	0.8	670	536	0.14	86.17	12	1.7	279	472	0.985
1984	2	19	0	0	0	1.78	670	1190	0.78	86.17	67.4	3	279	845	2.032
1984	2	20	0	0	0	2.28	670	1529	0.76	86.17	65.2	0.7	279	195	1.729
1984	2	21	0	0	0	1.54	670	1029	0.09	86.17	7.87	0.8	279	227	1.221
1984	2	22	0	0	0	1.59	670	1064	0.1	86.17	8.24	1	279	266	1.293
1984	2	23	0	0	0	0.99	670	666	0.1	86.17	8.62	1.2	279	341	0.981
1984	2	24	0	0	0	1.45	670	973	0.37	86.17	31.8	1.9	279	525	1.479
1984	2	25	0	0	0	1.99	670	1336	1.7	86.17	146	3	279	838	2.242
1984	2	26	0	0	0	1.16	670	776	0.1	86.17	8.99	2.4	279	669	1.405
1984	2	27	0	0	0	1.06	670	713	0.62	86.17	53.2	1.5	279	429	1.155
1984	2	28	0	0	0	2.93	670	1963	1.04	86.17	89.5	0.4	279	120	2.099
1984	2	29	0	0	0	0	670	0	0	86.17	0	0	279	0	0
1984	3	1	0	0	0	1.6	670	1072	0.42	86.17	36.3	1.1	279	303	1.364
1984	3	2	0	0	0	1.09	670	727	1.17	86.17	101	2.3	279	648	1.426
1984	3	3	0	0	0	3.16	670	2117	1.08	86.17	93.3	1.8	279	514	2.633
1984	3	4	0	0	0	3.29	670	2201	4.14	86.17	357	3.5	279	986	3.424
1984	3	5	0	0	0	1.02	670	683	0.83	86.17	71.9	2.5	279	706	1.412
1984	3	6	0	0	0	3.77	670	2523	1.15	86.17	99.3	1.5	279	406	2.926
1984	3	7	0	0	0	3.02	670	2023	0.98	86.17	84.3	5.1	279	1411	3.4
1984	3	8	0	0	0	1.81	670	1209	1.6	86.17	138	2.3	279	641	1.922
1984	3	9	0	0	0	1.07	670	717	0.68	86.17	58.8	2.6	279	737	1.462
1984	3	10	0	0	0	0.48	670	322	1.73	86.17	149	1.1	279	312	0.756
1984	3	11	0	0	0	0.58	670	385	1.11	86.17	95.5	1.3	279	364	0.816
1984	3	12	0	0	0	3.28	670	2198	0.68	86.17	58.8	3.5	279	976	3.123
1984	3	13	0	0	0	3.11	670	2080	0.58	86.17	49.8	1.9	279	526	2.567
1984	3	14	0	0	0	0.63	670	422	1.73	86.17	149	4.3	279	1204	1.715
1984	3	15	0	0	0	1.82	670	1219	0.67	86.17	58	1.6	279	448	1.667
1984	3	16	0	0	0	2.48	670	1658	2.7	86.17	233	1.7	279	462	2.273
1984	3	17	21.4	0	0	1.25	670	838	1.58	86.17	136	2.7	279	758	1.673

1984	3	18	0	0	0	2.81	670	1883	2.31	86.17	199	3.2	279	885	2.867
1984	3	19	0	0	0	1.17	670	781	1.05	86.17	90.1	2.8	279	791	1.605
1984	3	20	0	0	0	1.02	670	683	2.49	86.17	215	1.6	279	432	1.285
1984	3	21	0	0	0	1.89	670	1263	1.93	86.17	166	1.2	279	333	1.703
1984	3	22	0	0	0	1.36	670	908	0.64	86.17	55	2	279	564	1.476
1984	3	23	0	0	0	3.89	670	2606	1.4	86.17	120	1.4	279	394	3.015
1984	3	24	0	0	0	1.85	670	1236	1.89	86.17	163	1.3	279	369	1.709
1984	3	25	0	0	0	2.08	670	1390	0.53	86.17	45.8	1.9	279	517	1.888
1984	3	26	0	0	0	2.89	670	1933	1.93	86.17	166	2	279	554	2.564
1984	3	27	0	0	0	4.08	670	2730	0.86	86.17	74.4	3.8	279	1061	3.735
1984	3	28	0	0	0	4.38	670	2935	1.72	86.17	148	3.3	279	932	3.88
1984	3	29	0	0	0	3.64	670	2435	2.02	86.17	174	3.6	279	995	3.483
1984	3	30	0	0	0	2.55	670	1709	1.94	86.17	167	3.8	279	1049	2.825
1984	3	31	0	0	0	1.66	670	1109	0.81	86.17	70.2	2.9	279	805	1.917
1984	4	1	16.9	0	0	2.52	670	1689	1.98	86.17	170	2.4	279	681	2.455
1984	4	2	16.3	0	0	2.16	670	1446	1.44	86.17	124	3.3	279	906	2.393
1984	4	3	0	0	0	4.41	670	2952	5.78	86.17	498	1.1	279	313	3.635
1984	4	4	0	0	0	6.08	670	4076	1.69	86.17	145	2.5	279	705	4.76
1984	4	5	0	0	0	4.24	670	2839	0.81	86.17	69.7	1.1	279	314	3.114
1984	4	6	8.1	0	0	1.69	670	1132	1.01	86.17	86.9	3.6	279	1005	2.149
1984	4	7	0	0	0	5.26	670	3523	1.69	86.17	146	1.4	279	376	3.908
1984	4	8	0	0	0	2.93	670	1961	3.53	86.17	304	5.2	279	1446	3.586
1984	4	9	0	0	0	1.93	670	1294	2.98	86.17	257	6.2	279	1737	3.178
1984	4	10	0	0	0	3.75	670	2511	2.26	86.17	195	4.6	279	1268	3.84
1984	4	11	0	0	0	1.35	670	903	1.91	86.17	164	3.2	279	887	1.889
1984	4	12	0	0	0	3.58	670	2401	2.5	86.17	215	4	279	1115	3.606
1984	4	13	8.3	0	0	2.66	670	1779	0.89	86.17	76.8	3.9	279	1084	2.84
1984	4	14	14.1	0	0	1.85	670	1240	2.09	86.17	180	3.5	279	982	2.32
1984	4	15	0	0	0	2.23	670	1491	0.2	86.17	17.6	6.8	279	1908	3.301
1984	4	16	0	0	0	2.99	670	2003	1.88	86.17	162	2	279	548	2.622
1984	4	17	0	0	0	3.15	670	2112	1.27	86.17	109	1.9	279	534	2.663
1984	4	18	14.1	0	0	2.58	670	1731	1.68	86.17	145	3.1	279	858	2.642
1984	4	19	0	0	0	1.41	670	945	3.98	86.17	343	3.6	279	997	2.208
1984	4	20	19.3	0	0	4.25	670	2846	1.93	86.17	166	2.6	279	737	3.623
1984	4	21	0	0	0	2.49	670	1668	1.79	86.17	154	2	279	545	2.288
1984	4	22	0	0	0	2.22	670	1488	2.25	86.17	194	2.6	279	725	2.325
1984	4	23	0	0	0	2.85	670	1908	0.68	86.17	58.8	1.9	279	540	2.422
1984	4	24	0	0	0	2.75	670	1841	2.71	86.17	234	4.2	279	1158	3.124
1984	4	25	0	0	0	5.27	670	3533	3.3	86.17	284	6.1	279	1706	5.338
1984	4	26	0	0	0	4.86	670	3255	2.29	86.17	197	3.5	279	969	4.272
1984	4	27	0	0	0	2.52	670	1689	3.37	86.17	291	4.3	279	1200	3.073
1984	4	28	0	0	0	1.86	670	1245	0.68	86.17	58.4	2.1	279	585	1.825

1984	4	29	0	0	0	2.97	670	1992	0.86	86.17	74.4	1.7	279	471	2.452
1984	4	30	0	0	0	0.86	670	578	1.03	86.17	88.9	1.9	279	531	1.158
1984	5	1	0	0	0	1.48	670	994	0.6	86.17	51.3	0.2	279	48.8	1.057
1984	5	2	0	0	0	3.57	670	2390	0.89	86.17	76.8	1.2	279	347	2.718
1984	5	3	0	0	0	3.69	670	2475	1.12	86.17	96.3	1.6	279	456	2.926
1984	5	4	0	0	0	4.03	670	2699	1.73	86.17	149	3.6	279	1003	3.721
1984	5	5	0	0	0	2.17	670	1452	1.29	86.17	111	1.3	279	348	1.847
1984	5	6	0	0	0	2.98	670	1995	1.46	86.17	126	1.5	279	427	2.462
1984	5	7	0	0	0	2.11	670	1414	2.28	86.17	196	1.5	279	427	1.969
1984	5	8	0	0	0	4.17	670	2795	1.56	86.17	135	1.9	279	540	3.353
1984	5	9	0	0	0	5.69	670	3812	1.13	86.17	97.3	3.9	279	1085	4.826
1984	5	10	0	0	0	4.15	670	2779	1.42	86.17	123	1.6	279	432	3.221
1984	5	11	0	0	0	0.52	670	349	1.24	86.17	107	4.3	279	1197	1.597
1984	5	12	0	0	0	3.03	670	2031	2.4	86.17	207	4.2	279	1179	3.302
1984	5	13	0	0	0	1.17	670	783	1.17	86.17	101	2.4	279	679	1.511
1984	5	14	0	0	0	2.88	670	1932	1.24	86.17	107	2.4	279	674	2.622
1984	5	15	0	0	0	2.36	670	1583	0.97	86.17	83.5	3.3	279	922	2.501
1984	5	16	22.2	0	0	2.17	670	1453	1.86	86.17	160	0.8	279	221	1.772
1984	5	17	0	0	0	0.92	670	614	0.49	86.17	42.3	3	279	838	1.443
1984	5	18	0	0	0	2.77	670	1858	0.13	86.17	11.2	3.7	279	1025	2.797
1984	5	19	0	0	0	1.19	670	800	0.7	86.17	60.7	4	279	1108	1.903
1984	5	20	0	0	0	3.42	670	2292	1.41	86.17	121	4.9	279	1359	3.645
1984	5	21	0	0	0	1.18	670	790	1.83	86.17	158	4.8	279	1328	2.199
1984	5	22	0	0	0	0.12	670	77.6	1.93	86.17	166	4.1	279	1143	1.34
1984	5	23	0	0	0	0.54	670	363	1.55	86.17	133	1.9	279	526	0.988
1984	5	24	0	0	0	0.06	670	42.3	1.16	86.17	100	2.5	279	694	0.808
1984	5	25	0	0	0	0.76	670	508	0.78	86.17	67.1	2	279	569	1.105
1984	5	26	0	0	0	1.25	670	839	1.36	86.17	117	1	279	269	1.184
1984	5	27	0	0	0	1.01	670	677	1.21	86.17	104	1.9	279	533	1.27
1984	5	28	22.6	0	0	0.89	670	596	1.03	86.17	88.8	1.1	279	315	0.966
1984	5	29	14.6	0	0	0.95	670	635	0.91	86.17	78.7	0.9	279	244	0.925
1984	5	30	22.6	0	0	1.25	670	836	1.73	86.17	149	1.5	279	415	1.353
1984	5	31	0	0	0	2.27	670	1519	0.47	86.17	40.1	1.4	279	401	1.893
1984	6	1	0	0	0	0.66	670	439	0.98	86.17	84.2	2.4	279	679	1.162
1984	6	2	0	0	0	2.91	670	1950	0.53	86.17	45.7	1.1	279	315	2.232
1984	6	3	0	0	0	2.46	670	1650	1.73	86.17	149	1.4	279	380	2.106
1984	6	4	0	0	0	2.5	670	1675	1.15	86.17	99.4	1.2	279	344	2.047
1984	6	5	0	0	0	3	670	2010	0.94	86.17	80.9	0.9	279	243	2.255
1984	6	6	11.7	0	0	3.27	670	2190	1.23	86.17	106	1.4	279	388	2.594
1984	6	7	0	0	0	1.71	670	1146	1.78	86.17	153	3	279	842	2.069
1984	6	8	0	0	0	1.33	670	892	1.54	86.17	133	3.1	279	850	1.812
1984	6	9	0	0	0	1.88	670	1259	1.72	86.17	148	2.7	279	753	2.087

1984	6	10	0	0	0	2.47	670	1654	1.72	86.17	148	5.9	279	1633	3.319
1984	6	11	0	0	0	2.84	670	1903	2.73	86.17	235	5	279	1390	3.409
1984	6	12	0	0	0	1.63	670	1089	2.21	86.17	190	2.5	279	702	1.915
1984	6	13	0	0	0	2.23	670	1494	2.82	86.17	243	2.9	279	818	2.468
1984	6	14	0	0	0	3.34	670	2238	0.99	86.17	85.2	3.7	279	1029	3.239
1984	6	15	0	0	0	2.39	670	1601	3.03	86.17	261	1.9	279	539	2.32
1984	6	16	0	0	0	4.95	670	3317	2.55	86.17	219	2.2	279	613	4.009
1984	6	17	0	0	0	3.49	670	2338	3.23	86.17	278	2.7	279	743	3.246
1984	6	18	0	0	0	4.08	670	2730	1.87	86.17	161	2	279	569	3.343
1984	6	19	0	0	0	3.09	670	2067	4.01	86.17	345	2.1	279	580	2.891
1984	6	20	0	0	0	2.77	670	1856	2.28	86.17	197	3.5	279	989	2.939
1984	6	21	0	0	0	5.09	670	3407	2.83	86.17	244	3.9	279	1083	4.575
1984	6	22	18.6	0	0	3.02	670	2023	1.55	86.17	133	5.2	279	1446	3.481
1984	6	23	0	0	0	3.3	670	2211	1.66	86.17	143	4	279	1122	3.359
1984	6	24	0	0	0	2.7	670	1806	4.62	86.17	398	5.2	279	1447	3.528
1984	6	25	8.6	0	0	3.29	670	2204	3.65	86.17	315	3.3	279	920	3.323
1984	6	26	0	0	0	4.76	670	3186	6.58	86.17	567	4	279	1107	4.697
1984	6	27	0	0	0	5.18	670	3467	3.24	86.17	279	5.3	279	1464	5.035
1984	6	28	5.6	0	0	5.68	670	3806	3.87	86.17	334	5.3	279	1477	5.427
1984	6	29	0	0	0	2.87	670	1920	2.25	86.17	194	5	279	1392	3.387
1984	6	30	13.6	0	0	4.53	670	3035	4.48	86.17	386	2.8	279	780	4.06
1984	7	1	0	0	0	4	670	2683	2.78	86.17	240	6.3	279	1748	4.513
1984	7	2	0	0	0	4.29	670	2875	3.33	86.17	287	5.4	279	1491	4.496
1984	7	3	0	0	0	4.14	670	2776	5.11	86.17	440	5.1	279	1415	4.475
1984	7	4	0	0	0	5.26	670	3522	5.24	86.17	452	9.2	279	2572	6.325
1984	7	5	0	0	0	9.91	670	6643	7.34	86.17	633	5.5	279	1535	8.513
1984	7	6	0	0	0	7.63	670	5114	2.62	86.17	226	5.4	279	1505	6.614
1984	7	7	0	0	0	6.42	670	4304	4.16	86.17	359	5.4	279	1505	5.96
1984	7	8	0	0	0	7.16	670	4795	8.11	86.17	699	7.5	279	2090	7.329
1984	7	9	0	0	0	4.26	670	2855	3.33	86.17	287	9.2	279	2576	5.525
1984	7	10	0	0	0	6.34	670	4247	4.51	86.17	389	6.9	279	1929	6.343
1984	7	11	0	0	0	4.97	670	3331	8.05	86.17	693	7.1	279	1989	5.81
1984	7	12	0	0	0	6.72	670	4502	6.28	86.17	541	7.2	279	2013	6.818
1984	7	13	0	0	0	5.49	670	3679	5.4	86.17	466	6.8	279	1891	5.832
1984	7	14	0	0	0	8.85	670	5931	5.42	86.17	467	3.9	279	1075	7.221
1984	7	15	8.4	0	0	6.22	670	4167	5.21	86.17	449	4.8	279	1350	5.765
1984	7	16	0	0	0	12.4	670	8302	5.64	86.17	486	5.3	279	1467	9.909
1984	7	17	0	0	0	7.82	670	5239	6.08	86.17	524	8.2	279	2291	7.783
1984	7	18	6.7	0	0	4.24	670	2840	6.97	86.17	601	9.1	279	2528	5.767
1984	7	19	0	0	0	5.45	670	3653	3.7	86.17	319	9.4	279	2626	6.375
1984	7	20	15.6	0	0	4.99	670	3340	6.12	86.17	528	5.8	279	1623	5.305
1984	7	21	0	0	0	8.67	670	5807	6.31	86.17	544	6	279	1682	7.762

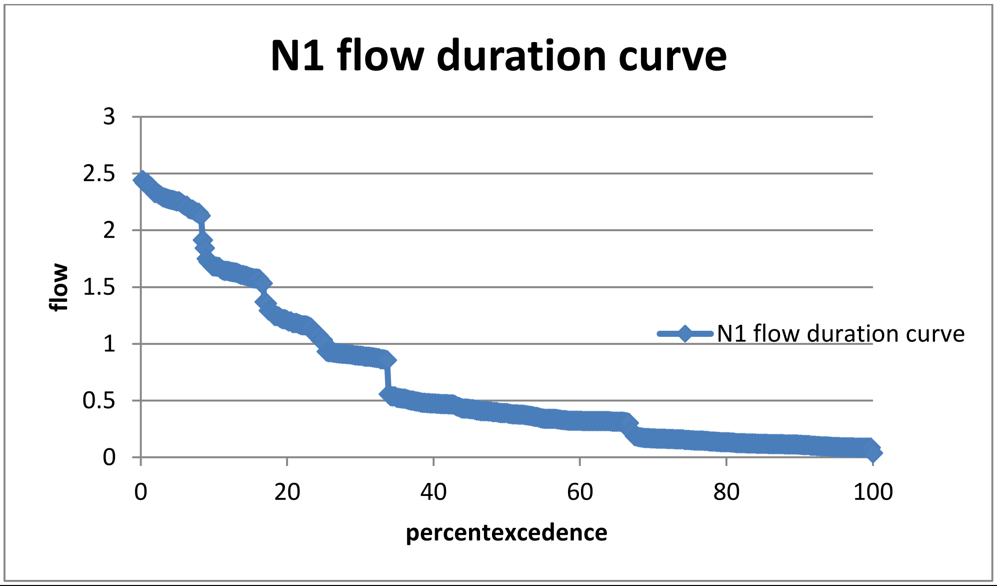
1984	7	22	0	0	0	5.76	670	3857	7.01	86.17	604	6.2	279	1738	5.991
1984	7	23	0	0	0	5.43	670	3640	11	86.17	952	2.7	279	757	5.169
1984	7	24	0	0	0	4.59	670	3072	4.48	86.17	386	7.2	279	2009	5.283
1984	7	25	0	0	0	3.82	670	2562	6.11	86.17	527	4.6	279	1280	4.222
1984	7	26	0	0	0	7.96	670	5334	7.28	86.17	627	5.3	279	1469	7.18
1984	7	27	0	0	0	7.06	670	4731	5.16	86.17	445	10	279	2877	7.781
1984	7	28	0	0	0	7.57	670	5070	4.33	86.17	373	6.4	279	1778	6.977
1984	7	29	0	0	0	8.65	670	5794	4.67	86.17	403	7.7	279	2152	8.067
1984	7	30	0	0	0	5.34	670	3577	4.21	86.17	363	6.1	279	1712	5.461
1984	7	31	0	0	0	5.5	670	3685	9.37	86.17	808	4.3	279	1191	5.492
1984	8	1	0	0	0	4.89	670	3273	7.96	86.17	686	6.7	279	1855	5.618
1984	8	2	0	0	0	6.37	670	4268	9.18	86.17	791	4.2	279	1177	6.025
1984	8	3	0	0	0	9.87	670	6613	6.44	86.17	555	5.5	279	1519	8.394
1984	8	4	0	0	0	4.56	670	3052	3.8	86.17	327	7.1	279	1987	5.185
1984	8	5	0	0	0	9.66	670	6469	8.25	86.17	711	3.4	279	946	7.852
1984	8	6	0	0	0	7.77	670	5203	6.38	86.17	550	5.3	279	1481	6.989
1984	8	7	0	0	0	7.92	670	5306	5	86.17	430	6.4	279	1782	7.265
1984	8	8	0	0	0	7.22	670	4837	5.74	86.17	495	4.9	279	1372	6.478
1984	8	9	0	0	0	7.98	670	5343	5.7	86.17	491	5.4	279	1499	7.086
1984	8	10	0	0	0	8.33	670	5578	11.1	86.17	956	6.3	279	1766	8.02
1984	8	11	0	0	0	10.6	670	7075	8.03	86.17	692	8.1	279	2248	9.677
1984	8	12	0	0	0	8.06	670	5397	5.43	86.17	468	9.5	279	2642	8.22
1984	8	13	0	0	0	8.05	670	5390	6.61	86.17	569	5.4	279	1491	7.2
1984	8	14	0	0	0	6.11	670	4090	6	86.17	517	5.1	279	1408	5.812
1984	8	15	0	0	0	8.71	670	5836	8.5	86.17	732	6.1	279	1710	7.999
1984	8	16	0	0	0	10.1	670	6777	3.46	86.17	298	5.7	279	1583	8.366
1984	8	17	0	0	0	8.02	670	5370	8.65	86.17	745	7.3	279	2031	7.871
1984	8	18	0	0	0	9.12	670	6107	5.59	86.17	482	4.5	279	1246	7.571
1984	8	19	0	0	0	6.8	670	4556	4.8	86.17	414	6.6	279	1846	6.586
1984	8	20	0	0	0	6.71	670	4492	5.92	86.17	510	9.6	279	2668	7.411
1984	8	21	0	0	0	7.09	670	4750	6	86.17	517	9.3	279	2590	7.592
1984	8	22	0	0	0	8.89	670	5956	5.86	86.17	505	9.1	279	2542	8.7
1984	8	23	0	0	0	10.7	670	7169	7.7	86.17	664	7.7	279	2140	9.637
1984	8	24	0	0	0	11.6	670	7802	4.91	86.17	423	8.1	279	2252	10.12
1984	8	25	0	0	0	7.86	670	5263	5.6	86.17	483	7.2	279	2015	7.499
1984	8	26	0	0	0	6.28	670	4208	6.34	86.17	547	6	279	1664	6.202
1984	8	27	22.4	0	0	6.87	670	4603	6.03	86.17	519	8.8	279	2445	7.312
1984	8	28	17.2	0	0	7.44	670	4981	4.62	86.17	398	8.5	279	2357	7.476
1984	8	29	9.9	0	0	7.44	670	4981	5.05	86.17	435	7.9	279	2204	7.363
1984	8	30	0	0	0	6.72	670	4499	5.11	86.17	441	6.5	279	1812	6.524
1984	8	31	14.5	0	0	5.33	670	3569	6.2	86.17	534	5.5	279	1529	5.442
1984	9	1	0	0	0	5.6	670	3749	3.37	86.17	291	1.5	279	425	4.314

1984	9	2	0	0	0	6.7	670	4492	5.4	86.17	465	2	279	549	5.321
1984	9	3	13.1	0	0	4.21	670	2823	4.83	86.17	416	3.4	279	935	4.033
1984	9	4	0	0	0	3.94	670	2640	3.08	86.17	266	3.2	279	892	3.67
1984	9	5	0	0	0	7.15	670	4791	2.9	86.17	250	1.3	279	352	5.21
1984	9	6	0	0	0	5.38	670	3606	4.26	86.17	367	2.1	279	598	4.417
1984	9	7	0	0	0	8.8	670	5896	3.04	86.17	262	1.6	279	457	6.392
1984	9	8	0	0	0	3.36	670	2251	4.85	86.17	418	2.5	279	699	3.254
1984	9	9	0	0	0	2.72	670	1821	1.74	86.17	150	1.8	279	504	2.392
1984	9	10	0	0	0	2.4	670	1611	2.89	86.17	249	1.5	279	416	2.199
1984	9	11	0	0	0	3.4	670	2278	4	86.17	345	0.6	279	180	2.709
1984	9	12	0	0	0	2.77	670	1858	4.12	86.17	355	1.3	279	375	2.5
1984	9	13	0	0	0	2.94	670	1967	2.83	86.17	244	1.8	279	504	2.624
1984	9	14	0	0	0	3.44	670	2305	4.53	86.17	391	1.9	279	521	3.108
1984	9	15	0	0	0	4.28	670	2866	2.97	86.17	256	1.2	279	322	3.327
1984	9	16	0	0	0	3.22	670	2159	2.92	86.17	251	2.5	279	684	2.99
1984	9	17	16.2	0	0	4.56	670	3055	1.69	86.17	145	2.2	279	626	3.697
1984	9	18	0	0	0	2.66	670	1785	0.92	86.17	79.3	1.6	279	457	2.242
1984	9	19	0	0	0	2.45	670	1638	2.21	86.17	191	3.3	279	928	2.664
1984	9	20	0	0	0	3	670	2010	1.51	86.17	130	2.8	279	774	2.816
1984	9	21	21.3	0	0	2.99	670	2004	2.91	86.17	251	1.9	279	527	2.689
1984	9	22	0	0	0	4.73	670	3167	0.92	86.17	79	1.6	279	437	3.559
1984	9	23	0	0	0	3.09	670	2071	3.01	86.17	260	1.4	279	392	2.631
1984	9	24	0	0	0	2.07	670	1386	1.14	86.17	97.9	1.3	279	371	1.792
1984	9	25	0	0	0	1.57	670	1054	0.64	86.17	55.1	1.7	279	480	1.536
1984	9	26	0	0	0	2.93	670	1961	0.92	86.17	78.9	0.7	279	197	2.162
1984	9	27	0	0	0	1.97	670	1322	1.73	86.17	149	0.4	279	116	1.533
1984	9	28	0	0	0	2.71	670	1818	1.05	86.17	90.7	0.1	279	23.6	1.867
1984	9	29	0	0	0	1.67	670	1121	1.52	86.17	131	0.8	279	214	1.416
1984	9	30	0	0	0	0.89	670	597	0.94	86.17	80.7	0.3	279	70.8	0.723
1984	10	1	0	0	0	3.53	670	2365	1.1	86.17	94.8	1.2	279	322	2.688
1984	10	2	0	0	0	3.59	670	2402	1.06	86.17	91.3	0.6	279	165	2.569
1984	10	3	0	0	0	3.05	670	2044	0.54	86.17	46.2	1.8	279	489	2.492
1984	10	4	0	0	0	3.45	670	2308	1.24	86.17	107	2.6	279	735	3.044
1984	10	5	0	0	0	3.44	670	2305	0.14	86.17	12.4	2.4	279	662	2.879
1984	10	6	0	0	0	1.72	670	1152	0.77	86.17	66.2	1.7	279	461	1.623
1984	10	7	0	0	0	1.41	670	945	0.81	86.17	69.6	1.5	279	427	1.392
1984	10	8	0	0	0	2.6	670	1742	1.12	86.17	96.5	1.1	279	298	2.064
1984	10	9	0	0	0	2.15	670	1441	1.27	86.17	110	0.6	279	180	1.672
1984	10	10	0	0	0	0.86	670	576	3.31	86.17	285	1.3	279	362	1.183
1984	10	11	0	0	0	0.44	670	291	0.42	86.17	36.5	1.8	279	504	0.804
1984	10	12	0	0	0	0.38	670	251	0.88	86.17	76.2	1.4	279	401	0.704
1984	10	13	0	0	0	1.71	670	1142	1.02	86.17	87.9	0.5	279	146	1.33

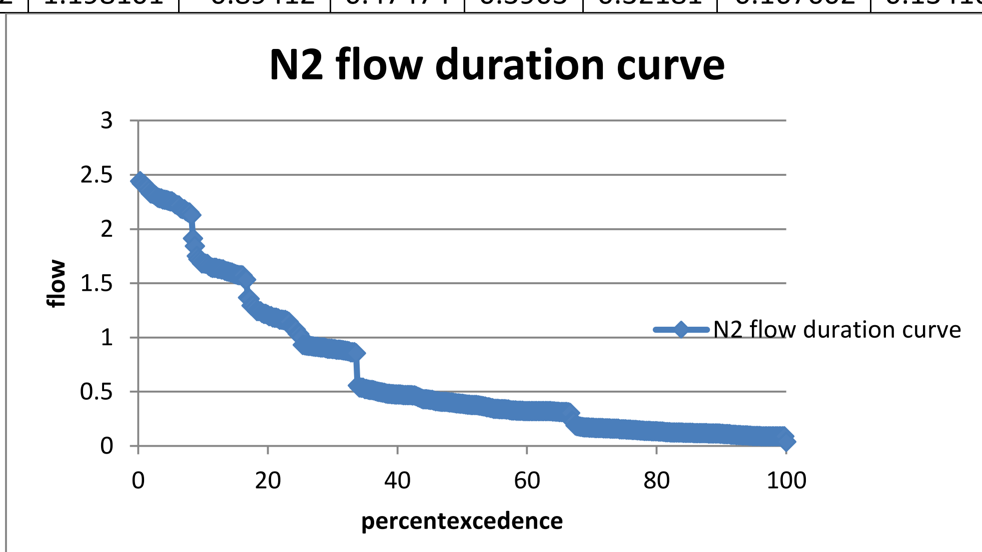
1984	10	14	0	0	0	1.33	670	891	1.37	86.17	118	1.5	279	407	1.369
1984	10	15	0	0	0	0.23	670	151	0.16	86.17	13.4	0.1	279	25.7	0.184
1984	10	16	0	0	0	0.5	670	335	0.11	86.17	9.65	0.1	279	27.9	0.36
1984	10	17	0	0	0	1.39	670	931	1.16	86.17	100	1.3	279	369	1.353
1984	10	18	0	0	0	1.99	670	1333	0.76	86.17	65.8	4.3	279	1211	2.522
1984	10	19	0	0	0	1.11	670	740	2.21	86.17	190	3.1	279	851	1.722
1984	10	20	0	0	0	2.17	670	1454	0.65	86.17	56.2	1.5	279	427	1.871
1984	10	21	0	0	0	1.2	670	804	1.5	86.17	129	0.5	279	152	1.049
1984	10	22	0	0	0	0.89	670	593	1.01	86.17	86.9	1.1	279	313	0.959
1984	10	23	0	0	0	0.99	670	663	0.72	86.17	62.4	0.8	279	212	0.906
1984	10	24	0	0	0	1.64	670	1099	0.9	86.17	77.9	1.1	279	307	1.433
1984	10	25	0	0	0	0.39	670	261	0.94	86.17	80.7	3.3	279	926	1.225
1984	10	26	0	0	0	0.56	670	372	0.59	86.17	50.7	0.1	279	30	0.437
1984	10	27	0	0	0	0.81	670	539	0.9	86.17	77.2	0.7	279	204	0.793
1984	10	28	0	0	0	1	670	670	1.08	86.17	92.7	0.1	279	38.6	0.774
1984	10	29	0	0	0	1.48	670	988	0.68	86.17	58.9	0.1	279	25.7	1.037
1984	10	30	0	0	0	0.01	670	3.35	0.06	86.17	5.51	0.8	279	221	0.222
1984	10	31	0	0	0	0.45	670	302	0.17	86.17	14.7	2.3	279	628	0.913
1984	11	1	0	0	0	0.16	670	105	0.17	86.17	14.2	0.6	279	162	0.271
1984	11	2	0	0	0	0	670	0	0.6	86.17	52.1	0.5	279	128	0.174
1984	11	3	0	0	0	0	670	0	0.92	86.17	79.4	0.4	279	110	0.183
1984	11	4	0	0	0	0	670	0	0.1	86.17	8.62	0.2	279	66.9	0.073
1984	11	5	0	0	0	0.01	670	8.38	0.17	86.17	14.6	0.1	279	29.7	0.051
1984	11	6	0	0	0	0	670	0	0.01	86.17	1.12	1.1	279	301	0.292
1984	11	7	0	0	0	0.81	670	544	0	86.17	0	0.4	279	115	0.637
1984	11	8	0	0	0	0.66	670	440	0.74	86.17	64.1	0.1	279	33.4	0.519
1984	11	9	0	0	0	0.9	670	603	0.28	86.17	24.4	0	279	1.86	0.608
1984	11	10	0	0	0	0.64	670	429	0.07	86.17	6.37	0.1	279	33.4	0.453
1984	11	11	5.8	0	0	0.3	670	201	0.66	86.17	56.6	0.4	279	102	0.347
1984	11	12	0	0	0	0.89	670	594	1.01	86.17	86.9	1.4	279	379	1.024
1984	11	13	0	0	0	0	670	0	0.05	86.17	4.5	0.7	279	201	0.198
1984	11	14	0	0	0	0	670	0	0.33	86.17	28.1	0.6	279	164	0.185
1984	11	15	5.6	0	0	0	670	0	0	86.17	0	0.8	279	219	0.212
1984	11	16	0	0	0	0.25	670	170	0.28	86.17	24.4	0.8	279	232	0.412
1984	11	17	0	0	0	0.55	670	366	0.27	86.17	22.9	0.3	279	91	0.464
1984	11	18	0	0	0	0.57	670	380	0.4	86.17	34.8	0.1	279	29.7	0.429
1984	11	19	0	0	0	0.15	670	98.3	0.09	86.17	7.87	0.5	279	132	0.23
1984	11	20	0	0	0	1.16	670	777	0.98	86.17	84.7	0.3	279	83.6	0.914
1984	11	21	0	0	0	0.27	670	183	0.57	86.17	49.5	1	279	286	0.501
1984	11	22	0	0	0	0	670	0	0.16	86.17	13.5	1.2	279	342	0.343
1984	11	23	0	0	0	0.57	670	380	0.58	86.17	50.2	1.9	279	543	0.94
1984	11	24	0	0	0	0.08	670	53.6	0.41	86.17	35.6	0.5	279	139	0.221

1984	11	25	0	0	0	0.1	670	67	0.26	86.17	22.1	0.7	279	203	0.282
1984	11	26	0	0	0	0.03	670	17.9	0.19	86.17	16.5	0	279	9.29	0.042
1984	11	27	0	0	0	0	670	0	0	86.17	0	0.1	279	16.7	0.016
1984	11	28	0	0	0	0.45	670	299	0.11	86.17	9.37	0.1	279	37.2	0.334
1984	11	29	0	0	0	1.57	670	1054	0.15	86.17	13.1	0.2	279	52	1.082
1984	11	30	0	0	0	0.19	670	125	0.31	86.17	26.6	0	279	0	0.147
1984	12	1	0	0	0	0	670	0	0.13	86.17	11.6	0.1	279	15.9	0.027
1984	12	2	0	0	0	0.02	670	11.2	0	86.17	0	0.2	279	51.8	0.061
1984	12	3	0	0	0	0.11	670	74.4	0.21	86.17	18.4	0.4	279	119	0.205
1984	12	4	0	0	0	1.63	670	1091	0.35	86.17	30.3	0.7	279	185	1.262
1984	12	5	0	0	0	0.75	670	503	0.01	86.17	1.12	0.5	279	151	0.633
1984	12	6	0	0	0	0.56	670	376	0.45	86.17	39	0.8	279	215	0.609
1984	12	7	0	0	0	0.39	670	261	0.75	86.17	64.4	0.6	279	163	0.472
1984	12	8	0	0	0	2.03	670	1362	0.21	86.17	18.4	0.3	279	71.7	1.403
1984	12	9	0	0	0	1.03	670	692	0.16	86.17	13.9	0.2	279	43.8	0.725
1984	12	10	7.9	0	0	0.58	670	391	0.05	86.17	4.5	0.4	279	113	0.492
1984	12	11	0	0	0	0.22	670	149	0	86.17	0	0.1	279	27.9	0.171
1984	12	12	0	0	0	0.07	670	48.4	0	86.17	0	0	279	0	0.047
1984	12	13	0	0	0	0	670	0	0.07	86.17	5.62	0	279	0	0.005
1984	12	14	0	0	0	0.27	670	179	0.07	86.17	5.99	0	279	0	0.178
1984	12	15	0	0	0	0.41	670	275	0.22	86.17	19.1	0	279	0	0.285
1984	12	16	0	0	0	2.29	670	1537	0.43	86.17	37.5	0	279	0	1.522
1984	12	17	0	0	0	0.97	670	651	0.06	86.17	5.25	0	279	0	0.634
1984	12	18	0	0	0	0.84	670	566	0.14	86.17	12.4	0.5	279	142	0.695
1984	12	19	0	0	0	0.75	670	503	0	86.17	0	0	279	4.29	0.49
1984	12	20	0	0	0	1.26	670	841	0.95	86.17	82	0	279	0	0.892
1984	12	21	0	0	0	3.04	670	2036	0.16	86.17	13.9	0.8	279	227	2.2
1984	12	22	0	0	0	1.19	670	800	0.05	86.17	4.5	0.3	279	70.8	0.846
1984	12	23	0	0	0	0.58	670	387	0.6	86.17	51.7	0.6	279	169	0.588
1984	12	24	0	0	0	0.68	670	454	0.04	86.17	3.37	0.2	279	68.6	0.508
1984	12	25	0	0	0	1	670	670	0.04	86.17	3.75	0.4	279	120	0.767
1984	12	26	0	0	0	0.77	670	517	0.84	86.17	72.3	1	279	279	0.839
1984	12	27	0	0	0	0.56	670	376	0.92	86.17	79.1	1.5	279	410	0.835
1984	12	28	0	0	0	0.48	670	320	0.13	86.17	11.2	0.1	279	32.2	0.351
1984	12	29	0	0	0	0.46	670	305	0	86.17	0	0.8	279	214	0.502
1984	12	30	0	0	0	1.03	670	692	0.23	86.17	19.9	0.9	279	257	0.937
1984	12	31	0	0	0	1.5	670	1005	0.02	86.17	1.5	0.4	279	116	1.085

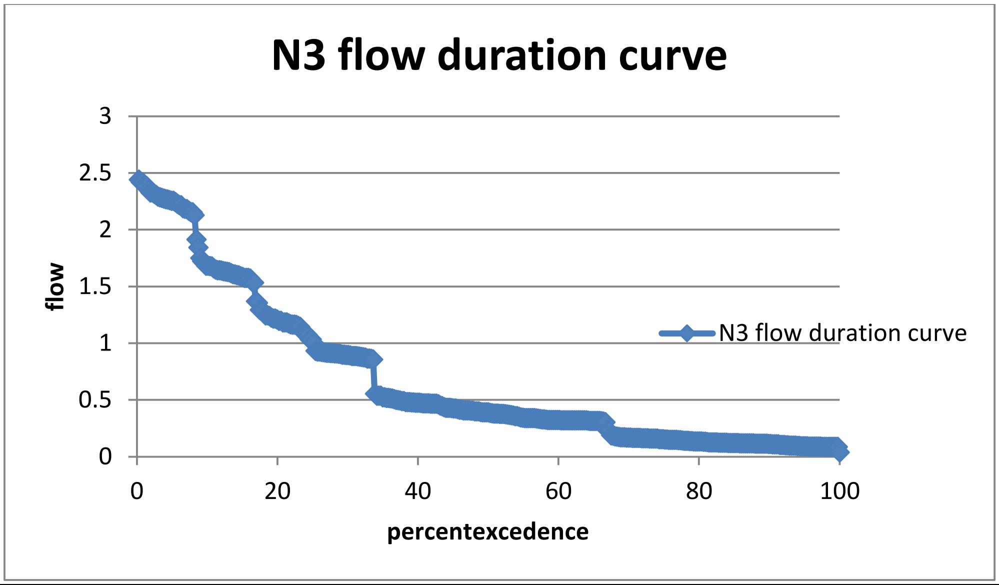
B2: flow duration curve for each site



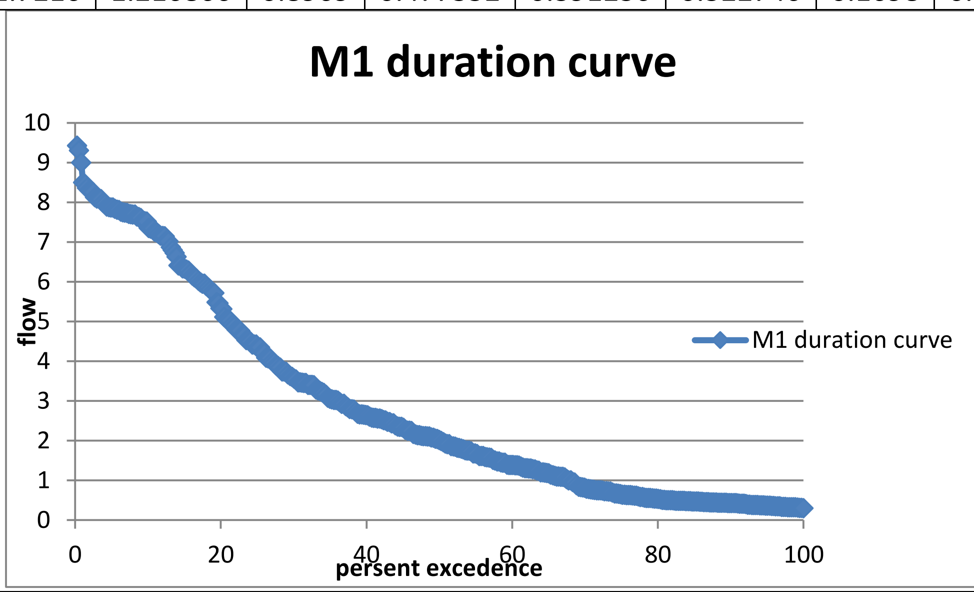
P(%)	10	20	30	40	50	60	70	80	90	100
Discharge (m^3/s)	1.6822	1.198101	0.89412	0.47474	0.3903	0.32181	0.167002	0.134165	0.111648	0.038



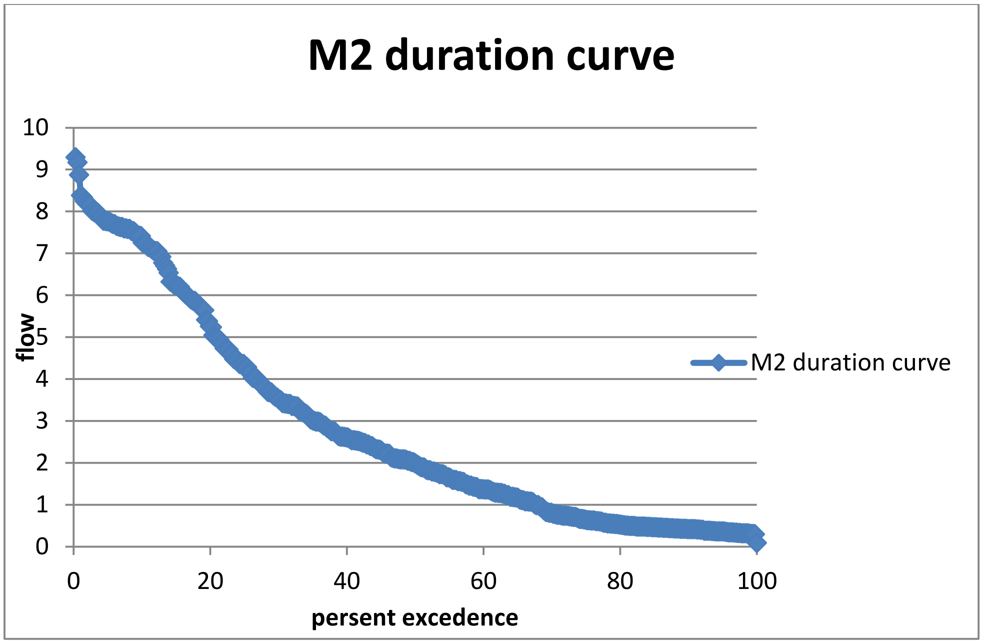
P(%)	10	20	30	40	50	60	70	80	90	100
Discharge (m^3/s)	1.721626	1.21687	0.89693	0.477552	0.391236	0.322746	0.169817	0.135	0.1135	0.0873



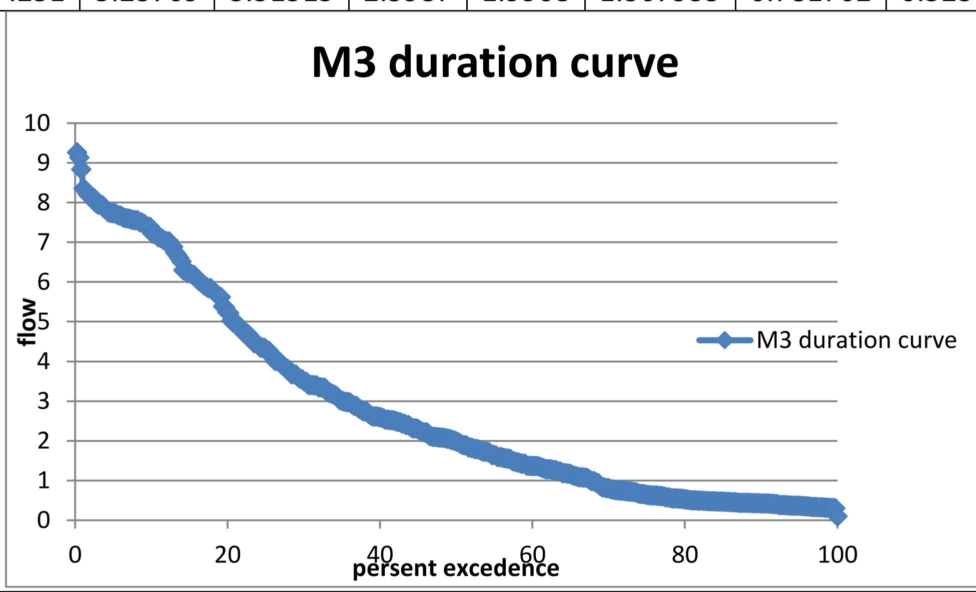
P(%)	10	20	30	40	50	60	70	80	90	100
Discharge (m^3/s)	1.7216	1.216866	0.8969	0.477552	0.391236	0.322746	0.1698	0.135103	0.113524	0.087



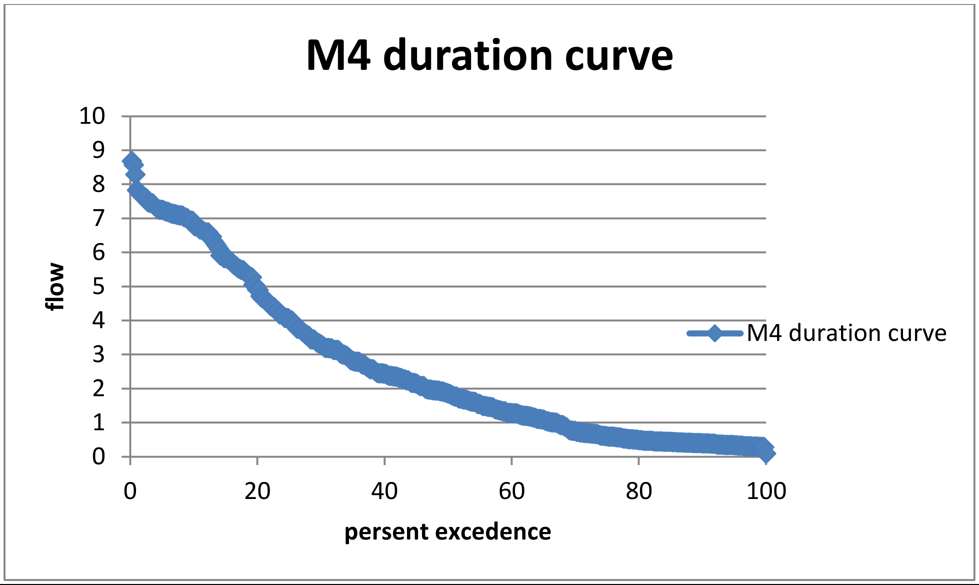
P(%)	10	20	30	40	50	60	70	80	90	100
Discharge (m^3/s)	7.3513	5.3099	3.564	2.6349	2.018463	1.386954	0.792631	0.530353	0.426376	0.310853



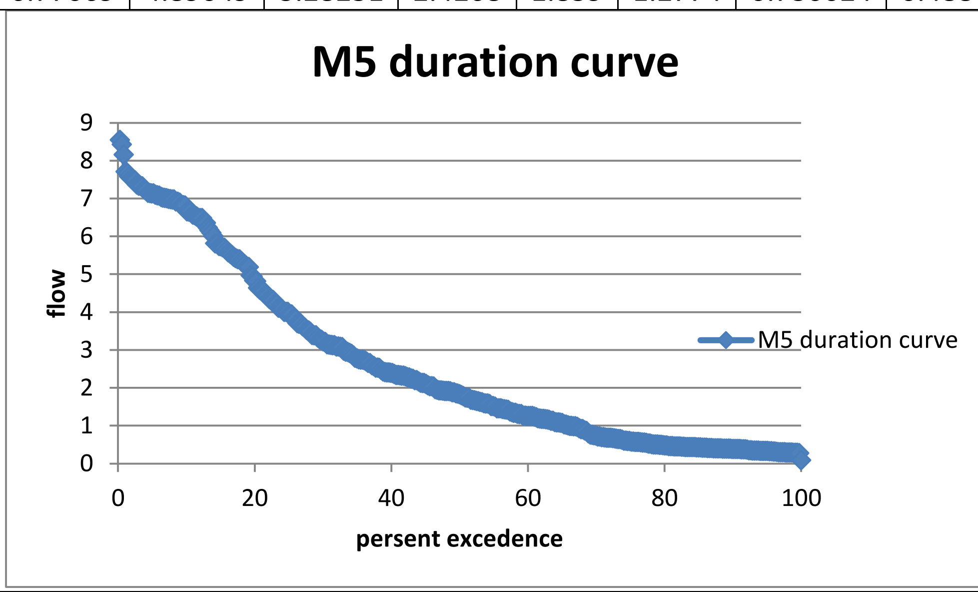
P(%)	10	20	30	40	50	60	70	80	90	100
Discharge (m^3/s)	7.251	5.23709	3.51515	2.5987	1.9908	1.367935	0.781762	0.523081	0.420529	0.295736



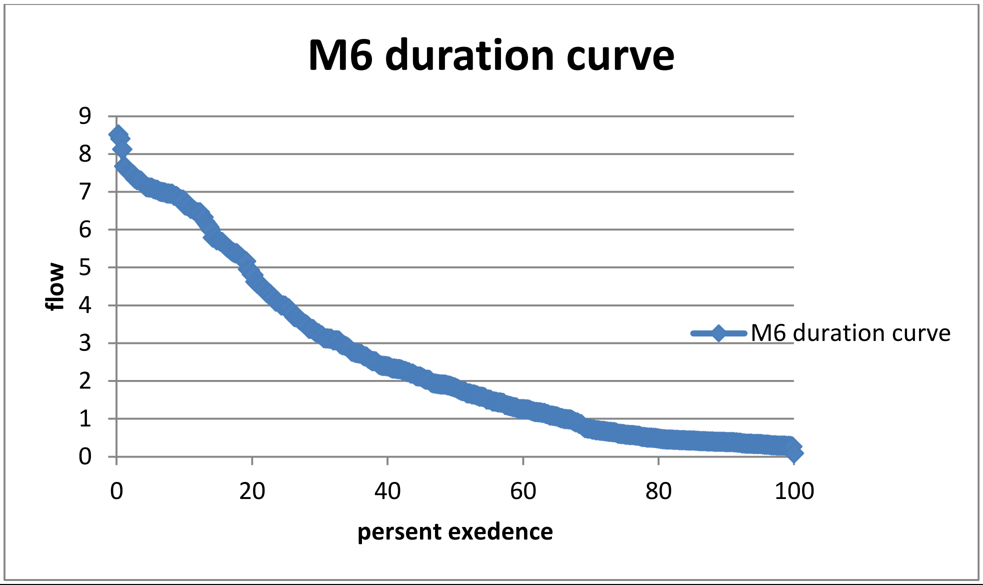
P(%)	10	20	30	40	50	60	70	80	90	100
Discharge (m^3/s)	7.2182	5.21376	3.4995	2.5872	1.981916	1.361841	0.778279	0.52075	0.418656	0.294418



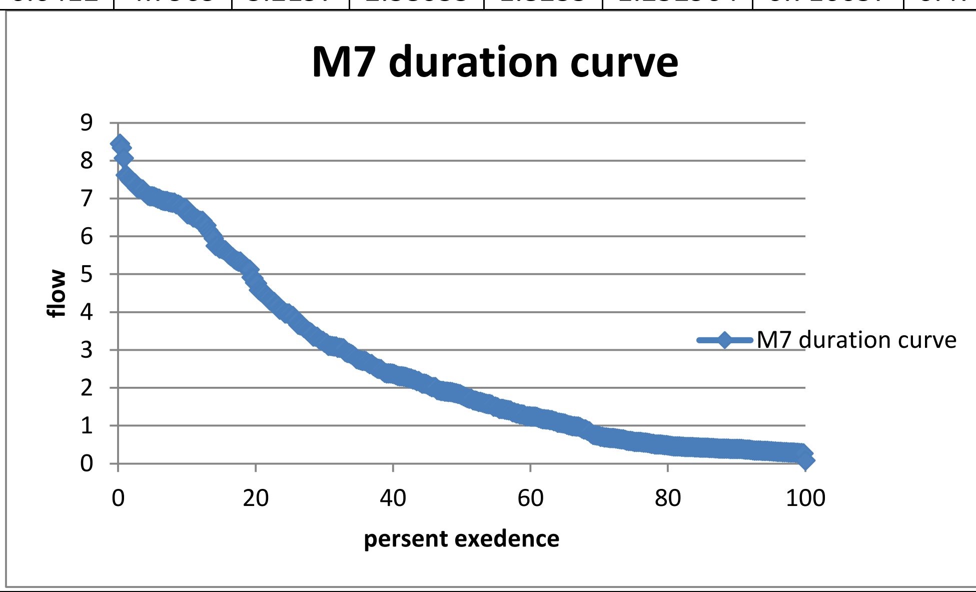
P(%)	10	20	30	40	50	60	70	80	90	100
Discharge (m^3/s)	6.77069	4.89049	3.28251	2.4268	1.859	1.2774	0.730024	0.488463	0.392698	0.276163



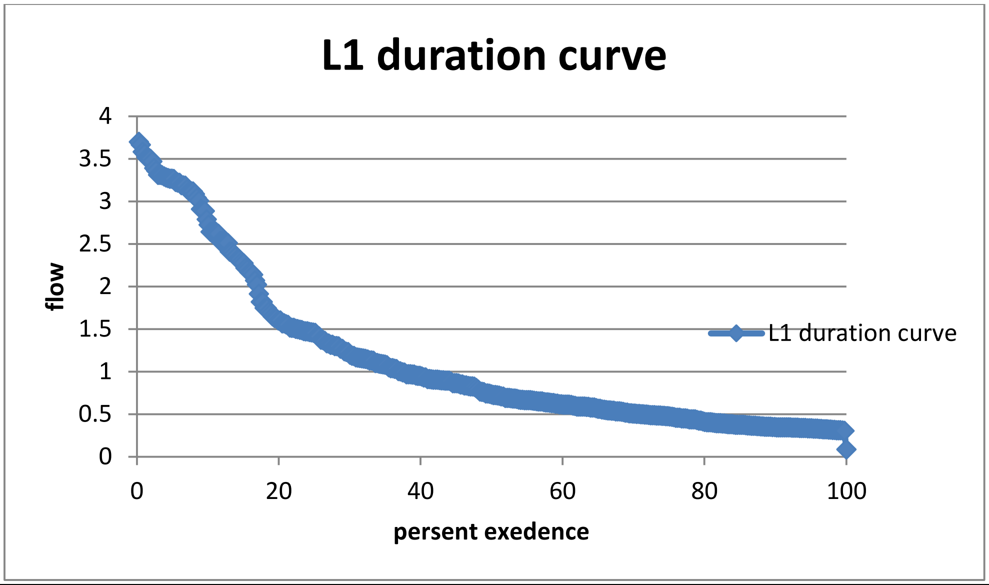
P(%)	10	20	30	40	50	60	70	80	90	100
Discharge (m^3/s)	6.6658	4.8147	3.2316	2.3891	1.830219	1.257605	0.71871	0.480892	0.386612	0.271883



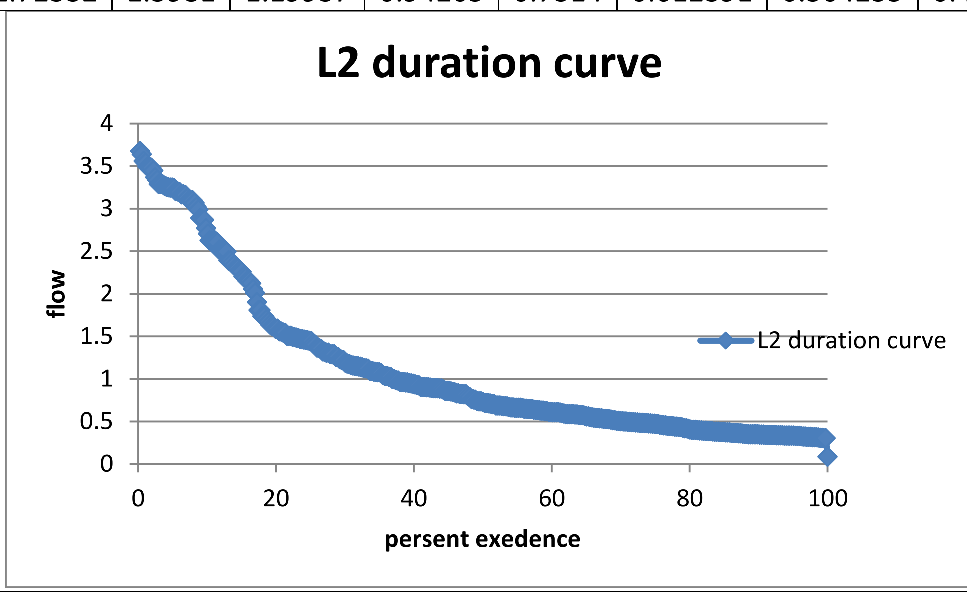
P(%)	10	20	30	40	50	60	70	80	90	100
Discharge (m^3/s)	6.6412	4.7969	3.2197	2.38033	1.8235	1.252964	0.716057	0.479117	0.385185	0.27088



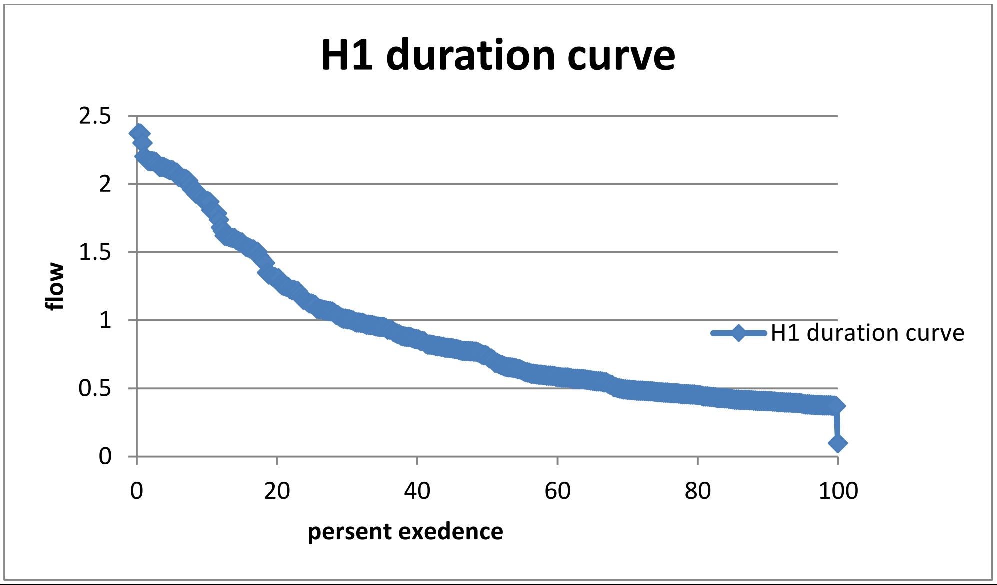
P(%)	10	20	30	40	50	60	70	80	90	100
Discharge (m^3/s)	6.58935	4.7595	3.1946	2.3618	1.80924	1.24319	0.710472	0.47538	0.382181	0.268767



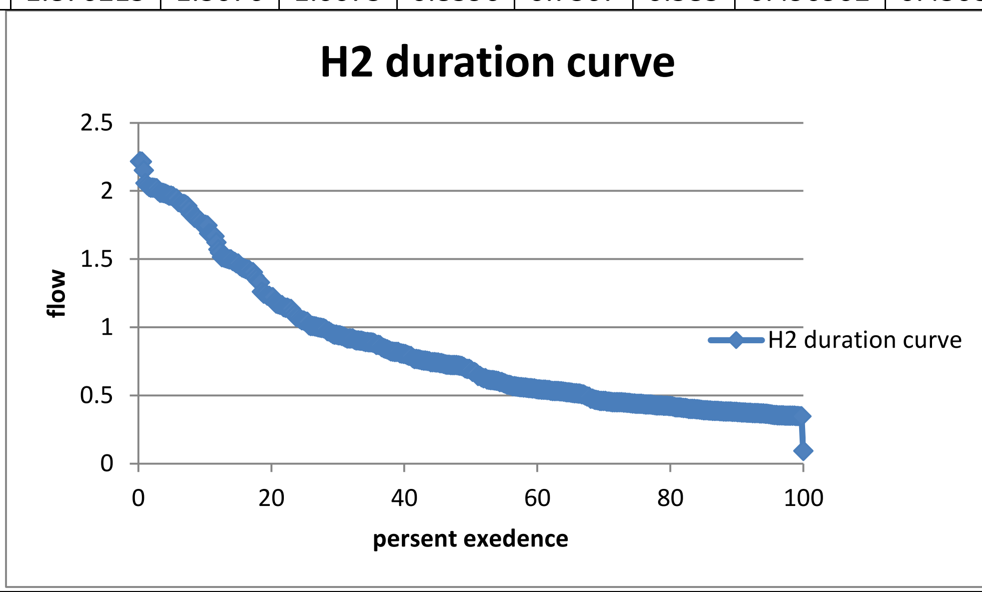
P(%)	10	20	30	40	50	60	70	80	90	100
Discharge (m^3/s)	2.72332	1.5981	1.19937	0.94205	0.7314	0.612391	0.504235	0.4065	0.345093	0.303634



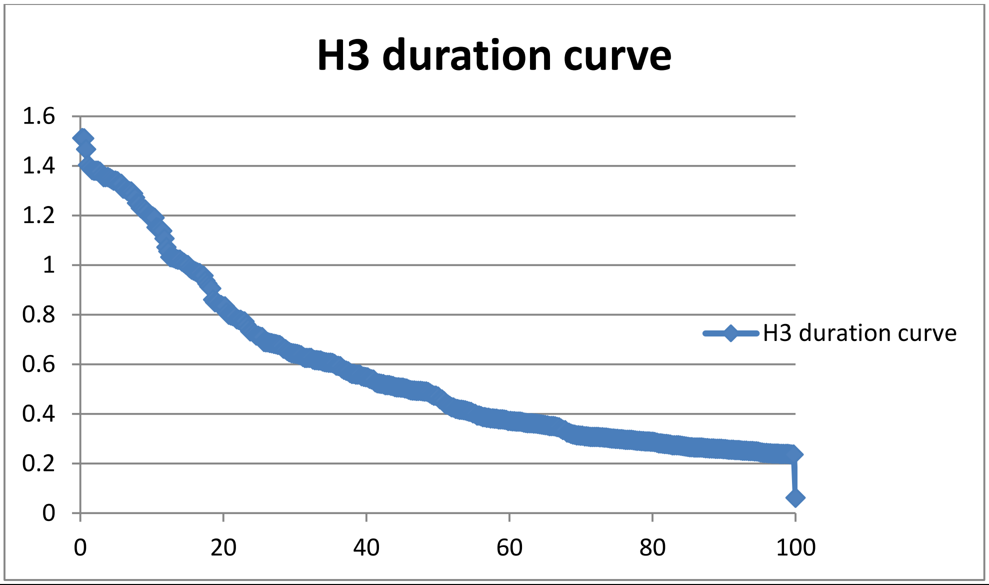
P(%)	10	20	30	40	50	60	70	80	90	100
Discharge (m^3/s)	2.70588	1.5878	1.1917	0.936	0.7267	0.608469	0.501006	0.403907	0.342883	0.301689



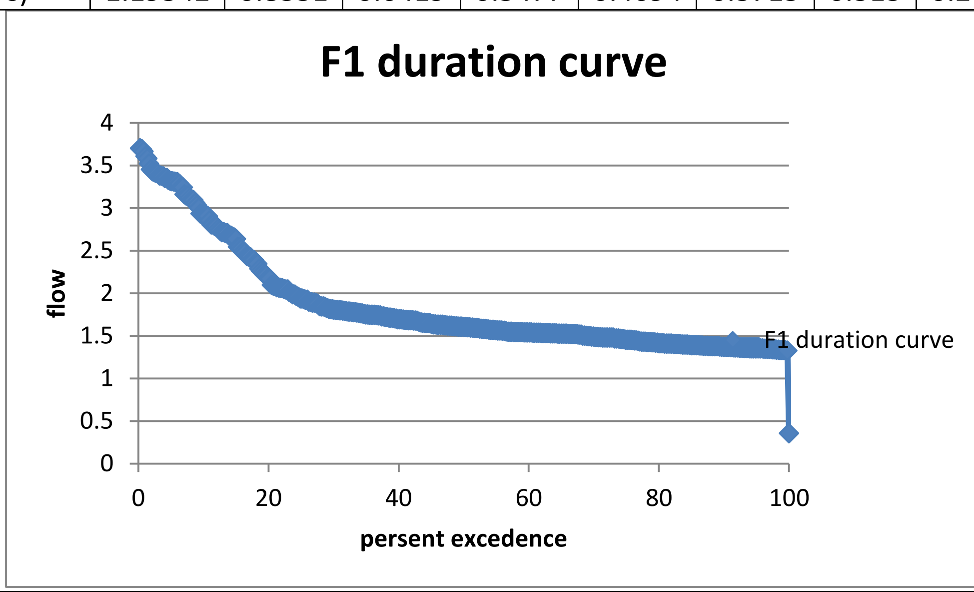
P(%)	10	20	30	40	50	60	70	80	90	100
Discharge (m^3/s)	1.876215	1.3076	1.0075	0.8596	0.7367	0.583	0.490902	0.450893	0.404903	0.370089



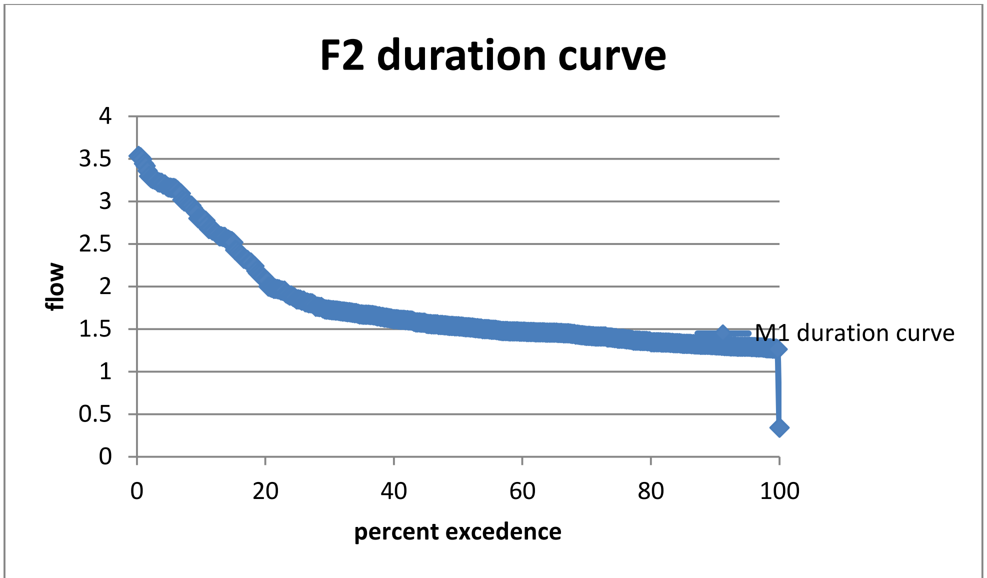
P(%)	10	20	30	40	50	60	70	80	90	100
Discharge (m^3/s)	1.7529	1.2217	0.94132	0.8031	0.6883	0.544736	0.458637	0.421258	0.378291	0.345765



P(%)	10	20	30	40	50	60	70	80	90	100
Discharge (m^3/s)	1.19542	0.8331	0.6419	0.5477	0.4694	0.3715	0.313	0.287284	0.257981	0.2358



P(%)	10	20	30	40	50	60	70	80	90	100
Discharge (m^3/s)	2.91462	2.16588	1.8076	1.6931	1.6048	1.5384	1.488486	1.411221	1.371595	1.323562



P(%)	10	20	30	40	50	60	70	80	90	100
Discharge (m^3/s)	2.78007	2.0659	1.72416	1.61498	1.5307	1.46742	1.419773	1.346075	1.308278	1.262462

Appendix C: Base flow for each river

C1: mean monthly Base flow of Maribo												
	Months											
	jan	feb	mar	apr	may	june	jul	aug	sept	nove	dec	oct
year												
1984	0.258	0.189	0.171	0.17	0.516	1.293	3.316	4.808	5.087	1.276	0.95	0.328
1985	0.2	0.146	0.118	1.377	2.506	0.455	1.984	4.161	4.961	1.546	0.996	0.348
1986	0.199	0.259	0.265	1.449	1.601	1.498	2.484	3.726	3.958	1.704	0.607	0.391
1987	0.191	0.215	0.928	1.947	1.747	1.704	1.378	7.015	6.794	4.192	1.365	0.511
1988	0.391	0.492	0.201	1.083	0.652	0.801	5.004	7.874	6.862	4.152	1.374	0.467
1989	0.118	0.105	0.258	1.674	2.289	1.414	4.153	7.901	6.862	4.152	1.374	0.509
1990	0.394	0.503	0.554	1.737	2.289	1.414	4.153	7.901	6.862	4.152	0.749	0.262
1991	0.138	0.133	0.311	1.304	0.714	0.474	2.827	9.814	8.937	1.661	0.362	0.238
1992	0.211	0.576	0.248	0.321	0.962	0.524	2.271	7.019	6.254	3.703	1.464	0.625
1993	0.704	1.586	0.341	2.146	3.624	2.56	4.103	7.525	4.88	3.717	1.117	0.24

1994	0.149	0.102	0.108	0.363	0.464	0.644	3.796	7.816	5.67	1.602	1.111	0.238
1995	0.102	0.108	0.36	2.41	0.617	0.298	0.626	4.559	3.503	1.422	0.208	0.238
1996	0.355	0.165	0.312	1.28	3.137	1.641	2.763	7.279	5.976	1.381	0.414	0.226
1997	0.196	0.053	0.044	0.619	0.536	0.487	3.755	4.418	2.254	4.511	5.04	1.373
1998	1.348	0.897	1.052	0.502	2.726	0.505	4.453	6.412	5.246	4.515	1.083	0.305
1999	0.131	0.142	0.651	0.513	0.523	0.71	3.552	5.187	3.547	6.102	0.614	0.404
2000	0.196	0.13	0.12	0.295	0.461	0.22	2.898	5.685	5.306	4.122	0.996	0.039
2001	0.12	0.206	1.197	2.126	2.073	2.806	4.895	6.852	5.802	1.198	0.407	0.465
2002	0.467	0.081	0.771	1.356	0.459	0.424	0.616	2.84	3.519	0.908	0.495	0.436
2003	0.322	0.18	0.184	0.676	0.581	0.523	3.14	7.24	4.597	1.988	0.621	0.59
2004	0.465	0.441	0.232	1.459	0.382	0.35	0.871	3.92	5.196	4.02	0.806	0.448
2005	0.288	0.209	0.408	1.03	4.147	0.922	3.403	6.794	4.243	2.425	0.758	0.366
2006	0.201	0.184	0.457	2.065	2.616	1.148	3.924	6.491	5.056	3.513	1.551	0.901
2007	0.349	0.485	0.253	1.266	1.104	1.79	3.273	6.504	5.134	2.36	1.182	0.496
2008	0.342	0.233	0.168	0.396	1.924	1.414	4.153	7.901	6.862	4.152	1.374	0.527
Average	**0.31**	**0.31**	**0.39**	**1.18**	**1.55**	**1.04**	**3.11**	**6.31**	**5.33**	**2.98**	**1.08**	**0.44**

C2: mean monthly Base flow of Furuna												
	Months											
	jan	feb	mar	apr	may	june	jul	aug	sept	nove	dec	oct
year												
1984	1.846	1.992	1.992	1.999	2.172	2.417	2.947	3.465	3.579	2.388	2.278	2.04
1985	2	1.982	1.992	2.454	2.821	2.135	2.603	3.247	3.515	2.485	2.28	2.041
1986	1.99	2.01	2.02	2.47	2.48	2.45	2.72	3.17	3.19	2.56	2.12	2.06

	4	8	8	2	2	7	7	2	7		8	2
1987	1.994	2.014	2.245	2.676	2.592	2.533	2.444	4.003	3.932	3.194	2.458	2.098
1988	2.063	2.097	2.032	2.404	2.239	2.249	3.561	4.317	3.966	3.196	2.445	2.103
1989	1.981	1.975	2.063	2.368	2.255	1.9	3.291	3.403	2.432	2.128	2.032	2.431
1990	2.012	2.761	3.721	3.926	2.268	1.926	2.43	3.752	2.745	2.056	1.895	1.883
1991	1.803	1.795	1.972	1.974	1.869	1.76	4.622	6.39	3.245	2.075	1.818	1.714
1992	1.858	1.949	1.778	1.853	1.872	1.828	2.113	9.256	4.117	2.352	1.861	1.801
1993	1.897	1.88	1.862	1.904	1.962	1.934	3.641	7.521	3.746	2.491	2.011	2.013
1994	1.897	1.879	1.862	1.931	1.962	1.934	3.641	7.521	3.746	2.491	2.011	2.013
1995	1.937	1.958	2.086	3.023	2.338	1.993	2.406	9.139	6.303	1.971	1.717	1.505
1996	1.671	1.849	2.069	2.199	2.321	3.635	3.419	6.692	2.879	2.386	2.259	2.11
1997	1.912	1.859	1.867	2.071	1.966	1.925	2.727	2.958	2.376	2.382	2.946	2.35
1998	2.652	2.391	2.451	2.149	2.473	2.096	3.692	5.567	5.115	3.927	2.69	2.091
1999	1.985	1.89	2.164	2.064	2.005	2.007	2.731	3.415	3.085	3.178	2.434	2.12
2000	1.981	1.899	1.861	2.029	2.123	1.977	2.684	4.378	3.266	3.265	2.774	2.21
2001	2.072	2.255	2.106	2.068	2.327	2.324	2.996	4.782	2.916	2.333	2.165	2.024
2002	2.093	1.881	2.116	2.099	1.971	2.064	2.196	2.508	2.403	2.179	2.058	2.09
2003	2.048	1.971	2.047	2.217	2.15	2.108	3.005	5.123	3.266	2.547	2.122	2.075
2004	2.021	2.025	1.962	2.492	2.022	2.173	2.441	3.764	2.443	2.475	2.032	1.941
2005	1.915	1.862	2	2.063	2.855	2.182	2.943	3.668	3.566	2.318	2.031	1.919
2006	1.863	1.879	1.924	2.373	2.477	2.041	2.659	4.455	3.287	2.688	2.254	1.98
2007	1.857	1.972	1.873	2.047	1.992	2.279	3.123	4.211	3.317	2.359	1.968	1.778
2008	1.73	1.69	1.65	1.77	1.81	1.91	2.40	3.92	2.66	2.14	2.24	1.92

	4	4	6	6	7	7	7	9	7	4	2	
Average	**1.96**	**1.99**	**2.07**	**2.27**	**2.21**	**2.15**	**2.94**	**4.83**	**3.40**	**2.54**	**2.20**	**2.01**

C3: mean monthly Base flow of Herero

	Months											
	jan	feb	mar	apr	may	june	jul	aug	sept	nove	dec	oct
year												
1984	0.797	0.828	0.828	0.836	0.97	1.228	1.873	2.384	2.496	1.228	1.115	0.882
1985	0.838	0.817	0.819	1.278	1.652	0.956	1.453	2.17	2.449	1.322	1.142	0.884
1986	0.831	0.858	0.869	1.297	1.342	1.295	1.601	2.041	2.115	1.381	0.982	0.908
1987	0.831	0.853	1.103	1.48	1.401	1.389	1.278	3.065	3.007	2.166	1.281	0.948
1988	0.909	0.947	0.869	1.185	1.036	1.078	2.463	3.375	3.027	2.161	1.274	0.949
1989	0.816	0.81	0.876	1.324	1.443	1.117	2.359	3.146	2.526	1.826	1.175	0.931
1990	0.157	0.638	0.745	0.804	0.137	0.019	0.13	0.958	1.019	0.144	0.004	0.002
1991	0.002	0.006	0.01	0.056	0.042	0.014	0.394	3.025	1.229	0.049	0.005	0
1992	0.004	0.087	0.019	0.043	0.095	0.09	0.694	1.497	1.131	0.457	0.932	0.333
1993	0.09	0.289	0.127	0.246	0.459	0.219	0.501	2.72	1.108	0.575	0.419	0.118
1994	0.051	0.021	0.036	0.061	0.133	0.162	0.835	2.849	1.592	0.27	0.208	0.17
1995	0.72	0.853	1.132	1.653	1.811	1.21	1.489	3.88	2.332	1.345	1.106	0.963
1996	1.228	0.638	1.064	1.469	2.063	2.612	2.09	5.078	2.667	1.7	1.005	0.717
1997	0.645	0.757	0.765	1.006	0.962	0.851	2.437	2.734	1.532	1.952	2.84	1.304
1998	1.375	1.391	1.513	1.148	1.547	0.925	1.636	4.117	3.407	3.128	1.401	0.877
1999	0.535	0.571	0.921	0.763	0.71	0.843	1.707	2.063	1.593	3.263	1.079	0.791
2000	0.735	0.872	0.808	0.827	0.968	0.841	2.686	2.758	2.186	2.554	1.43	0.918

2001	0.652	0.568	0.792	2.141	3.278	2.139	3.327	4.337	3.776	1.328	1.004	0.987
2002	0.879	0.643	0.692	1.068	0.688	0.797	0.943	2.137	1.754	1.071	0.787	0.771
2003	0.739	0.598	0.601	0.878	0.798	0.693	1.711	3.167	2.836	1.443	0.867	0.876
2004	0.705	0.624	0.434	0.869	0.585	0.739	1.55	2.96	3.602	2.007	0.898	0.735
2005	0.669	0.602	0.747	1	2.276	1.046	1.665	3.915	2.517	1.745	0.986	0.863
2006	0.765	0.824	0.891	1.827	1.458	1.14	2.942	3.994	3.325	1.487	1.314	1.054
2007	0.847	0.977	0.799	1.273	1.092	1.208	2.271	4.357	3.066	1.689	1.16	0.712
2008	0.783	0.707	0.625	0.74	1.204	1.124	1.92	3.421	2.608	1.829	1.299	0.891
Average	**0.66**	**0.67**	**0.72**	**1.01**	**1.13**	**0.95**	**1.68**	**3.05**	**2.36**	**1.52**	**1.03**	**0.74**

C4: mean monthly Base flow of Leliso												
	Months											
	jan	feb	mar	apr	may	june	jul	aug	sept	nove	dec	oct
year												
1984	0.329	0.306	0.304	0.306	0.443	0.734	1.503	2.071	2.181	0.732	0.615	0.368
1985	0.315	0.293	0.288	0.774	1.203	0.42	0.999	1.827	2.132	0.836	0.628	0.369
1986	0.31	0.336	0.343	0.797	0.856	0.815	1.18	1.66	1.749	0.893	0.481	0.396
1987	0.309	0.323	0.596	0.994	0.915	0.901	0.774	2.894	2.815	1.836	0.771	0.436
1988	0.388	0.435	0.326	0.662	0.5	0.555	2.147	3.231	2.841	1.817	0.774	0.421
1989	0.287	0.283	0.32	0.779	0.861	0.483	2.035	2.736	1.855	1.137	0.585	0.635
1990	0.234	0.227	1.084	0.645	0.352	0.077	0.274	1.115	0.688	0.363	0.104	0.059
1991	0.171	0.19	0.263	0.359	0.292	0.208	0.994	3.533	1.172	0.346	0.207	0.295
1992	0.311	0.453	0.241	0.301	0.264	0.184	0.458	6.673	1.21	0.592	0.443	0.263
1993	0.35	0.61	0.20	0.39	0.62	0.56	1.62	3.79	1.03	0.70	0.60	0.29

	9	2	6	1	1	7	8	5	4	4	8	7
1994	0.234	0.203	0.184	0.169	0.219	0.259	0.824	5.506	2.364	0.837	0.499	0.286
1995	0.118	0.122	0.185	1.412	0.629	0.306	0.631	5.983	3.766	1.043	0.695	0.608
1996	0.624	0.546	0.587	0.883	0.511	1.108	0.7	3.412	1.41	0.423	0.165	0.141
1997	0.115	0.112	0.116	0.286	0.2	0.134	1.366	2.031	0.501	1.158	1.847	0.882
1998	0.611	0.48	0.474	0.289	0.953	0.317	0.95	3.06	2.653	2.471	0.743	0.389
1999	0.286	0.198	0.318	0.243	0.242	0.234	1.146	1.749	1.463	2.04	0.581	0.359
2000	0.295	0.263	0.194	0.352	0.487	0.306	1.102	3.024	1.852	1.555	1.061	0.41
2001	0.255	0.199	0.629	0.848	0.626	0.543	1.524	3.667	0.97	0.555	0.456	0.327
2002	0.322	0.256	0.38	0.545	0.336	0.318	0.377	0.996	1.266	0.559	0.381	0.413
2003	0.375	0.289	0.283	0.465	0.387	0.342	1.159	3.031	1.592	0.981	0.588	0.57
2004	0.542	0.521	0.449	1.119	0.504	0.532	0.99	1.92	1.054	1.194	0.601	0.519
2005	0.473	0.439	0.568	0.669	1.783	0.662	1.446	3	1.9	0.863	0.583	0.5
2006	0.44	0.443	0.49	1.169	1.098	0.544	1.21	3.301	2.533	1.335	0.868	0.633
2007	0.446	0.499	0.408	0.627	0.501	0.792	2.247	3.173	2.349	0.994	0.558	0.415
2008	0.382	0.33	0.311	0.419	0.408	0.49	0.974	2.156	1.666	0.676	0.779	0.393
Average	**0.34**	**0.33**	**0.38**	**0.62**	**0.61**	**0.47**	**1.15**	**3.02**	**1.80**	**1.04**	**0.62**	**0.42**

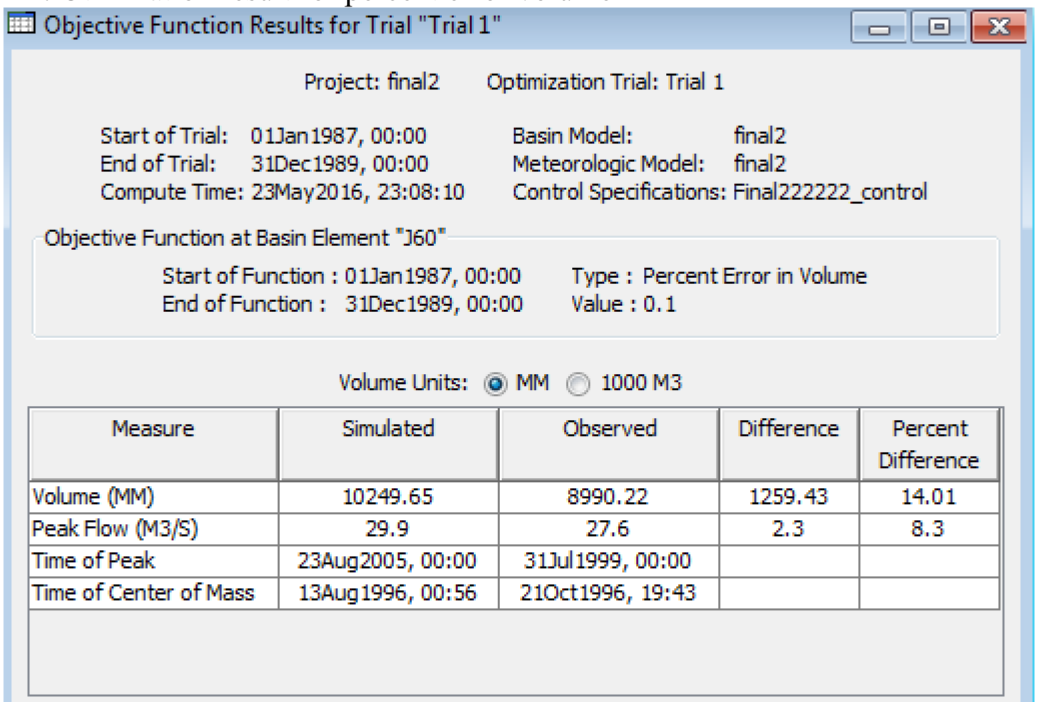
C5: mean monthly Base flow of Maribo chnaguite												
	Months											
	jan	feb	mar	apr	may	june	jul	aug	sept	nove	dec	oct
year												
1984	0.241	0.163	0.154	0.166	0.338	0.988	2.652	5.441	7.155	1.568	1.107	0.325
1985	0.27	0.14	0.15	1.29	2.84	0.42	2.47	4.33	6.54	1.97	1.10	0.37

	3	4	7	8	5	8	1	9	8	1	1	5
1986	0.204	0.197	0.229	1.708	0.808	1.233	2.102	4.71	5.063	2.142	0.722	0.325
1987	0.223	0.158	0.9	2.326	1.591	1.656	2.302	5.398	3.84	2.248	0.637	0.298
1988	0.201	0.196	0.229	0.99	0.537	0.555	4.018	6.482	5.787	4.456	0.724	0.315
1989	0.211	0.19	0.295	1.906	0.991	0.76	3.018	3.824	7.171	1.72	1.58	2.568
1990	0.643	1.083	2.308	3.168	0.958	0.305	2.903	5.256	4.853	1.943	0.434	0.337
1991	0.222	0.251	0.408	1.327	0.819	0.528	2.415	5.892	6.952	2.075	0.544	0.448
1992	0.285	0.629	0.314	0.386	0.852	0.775	3.078	6.46	6.086	2.863	1.577	1.012
1993	1.398	2.517	0.766	2.462	4.932	1.879	1.926	6.896	4.232	2.465	1.47	0.55
1994	0.32	0.081	0.254	0.457	0.626	1.767	2.74	8.43	10.12	5.11	1.791	0.862
1995	0.281	0.405	1.011	1.824	3.038	1.224	1.148	3.723	2.817	1.515	1.264	0.581
1996	0.796	0.364	0.901	1.642	3.56	1.971	3.359	5.687	2.572	0.832	0.429	0.297
1997	0.298	0.189	0.206	0.562	0.528	0.463	3.596	4.442	3.32	5.719	4.425	1.4
1998	1.644	1.427	1.142	0.611	3.32	0.569	4.205	5.3	4.965	5.832	0.919	0.325
1999	0.22	0.165	0.688	0.466	0.818	5.583	7.882	0.754	0.674	4.556	3.171	0.371
2000	0.2	0.351	0.978	1.144	1.445	1.085	3.901	5.995	6.129	4.977	2.007	0.951
2001	0.902	0.93	1.795	3.179	3.519	3.705	5.717	7.697	6.672	2.043	1.274	1.367
2002	1.322	0.872	1.405	2.077	1.189	1.203	1.426	3.541	3.866	1.678	1.223	1.18
2003	1.098	0.901	0.9	1.408	1.302	1.221	3.63	7.425	5.261	2.6	1.336	1.352
2004	1.18	1.129	0.87	1.977	1.06	1.099	1.991	5.005	6.567	4.597	1.517	1.16
2005	1.014	0.921	1.162	1.778	4.814	1.66	3.838	7.266	4.784	3.086	1.53	1.15
2006	0.996	1.033	1.3	2.894	3.055	1.856	4.691	7.146	5.857	3.818	2.244	1.647
2007	1.15	1.33	1.04	2.00	1.84	2.35	3.99	7.31	5.71	3.01	1.90	1.18

	7	1	6	8	2	7	2	6	5	1	9	5
2008	1.11 6	1.00 6	0.89 7	1.14 3	2.43	2.04 2	4.37 6	7.66 4	6.55 5	4.29 2	2.12 5	1.28 2
Averag e	**0.65 8**	**0.66 5**	**0.81 3**	**1.55 6**	**1.88 9**	**1.47 6**	**3.33 5**	**5.68 4**	**5.34 2**	**3.08 5**	**1.48 2**	**0.86 7**

Appendix D: HEC-HMS Model calibration & Validation outputs

D1: Otimization result for percent error volume



Objective Function Results for Trial "Trial 1"

Project: final2 Optimization Trial: Trial 1

Start of Trial: 01Jan1987, 00:00 Basin Model: final2
End of Trial: 31Dec1989, 00:00 Meteorologic Model: final2
Compute Time: 23May2016, 23:08:10 Control Specifications: Final222222_control

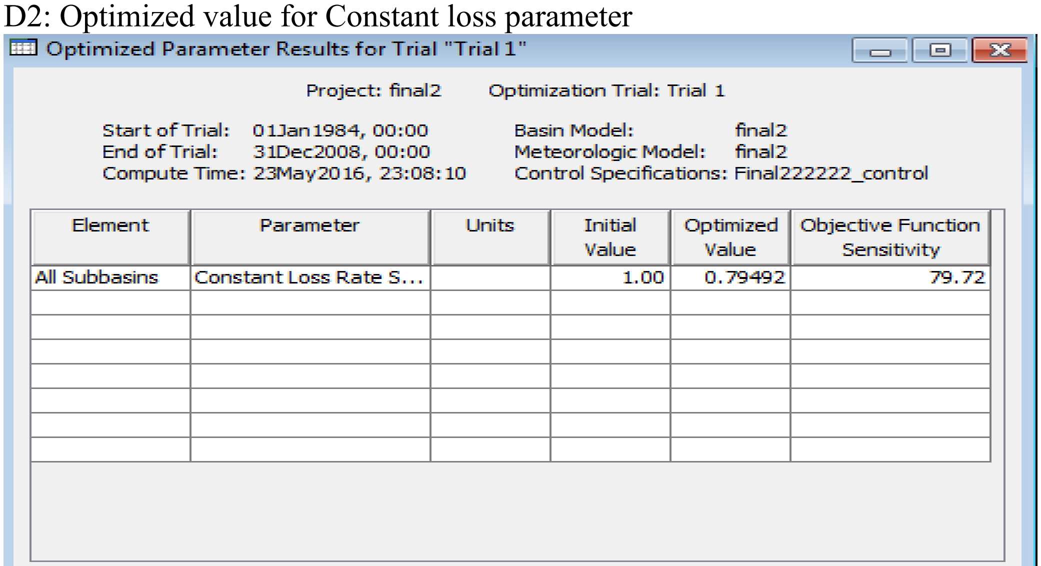
Objective Function at Basin Element "J60"

Start of Function : 01Jan1987, 00:00 Type : Percent Error in Volume
End of Function : 31Dec1989, 00:00 Value : 0.1

Volume Units: MM 1000 M3

Measure	Simulated	Observed	Difference	Percent Difference
Volume (MM)	10249.65	8990.22	1259.43	14.01
Peak Flow (M3/S)	29.9	27.6	2.3	8.3
Time of Peak	23Aug2005, 00:00	31Jul1999, 00:00		
Time of Center of Mass	13Aug1996, 00:56	21Oct1996, 19:43		

D2: Optimized value for Constant loss parameter



Optimized Parameter Results for Trial "Trial 1"

Project: final2 Optimization Trial: Trial 1

Start of Trial: 01Jan1984, 00:00 Basin Model: final2
End of Trial: 31Dec2008, 00:00 Meteorologic Model: final2
Compute Time: 23May2016, 23:08:10 Control Specifications: Final222222_control

Element	Parameter	Units	Initial Value	Optimized Value	Objective Function Sensitivity
All Subbasins	Constant Loss Rate S...		1.00	0.79492	79.72

D3: Sample Time series table at Maribo gaging station

Time-Series Results for Junction "J60"

Project: final2
Optimization Trial: Trial 1   Junction: J60

Start of Trial: 01Jan1984, 00:00   Basin Model: final2
End of Trial: 31Dec2008, 00:00   Meteorologic Model: final2
Compute Time: 23May2016, 23:08:10   Control Specifications: Final222222_control

Date	Time	Inflow from W220 (M3/S)	Inflow from W210 (M3/S)	Outflow (M3/S)	Obs Flow (M3/S)
29Jun1984	00:00	4.2	0.7	4.9	2.1
30Jun1984	00:00	3.0	0.8	3.8	2.2
01Jul1984	00:00	1.9	0.7	2.5	2.3
02Jul1984	00:00	4.3	0.5	4.8	2.2
03Jul1984	00:00	4.6	0.5	5.1	2.3
04Jul1984	00:00	4.7	0.5	5.2	2.7
05Jul1984	00:00	6.8	0.7	7.5	3.1
06Jul1984	00:00	10.9	1.4	12.3	3.0
07Jul1984	00:00	11.4	2.3	13.7	2.6
08Jul1984	00:00	9.7	2.5	12.2	2.3
09Jul1984	00:00	10.3	2.4	12.7	2.3
10Jul1984	00:00	9.3	2.4	11.7	2.2
11Jul1984	00:00	8.8	2.3	11.1	2.2
12Jul1984	00:00	8.6	2.1	10.7	2.2
13Jul1984	00:00	9.3	2.1	11.4	2.0
14Jul1984	00:00	9.0	2.1	11.2	2.1
15Jul1984	00:00	9.9	2.2	12.1	2.6
16Jul1984	00:00	9.4	2.3	11.7	4.7
17Jul1984	00:00	12.9	2.5	15.4	6.0
18Jul1984	00:00	14.2	3.2	17.4	6.6
19Jul1984	00:00	11.0	3.4	14.4	7.0
20Jul1984	00:00	9.5	2.9	12.4	7.1
21Jul1984	00:00	8.2	2.4	10.7	6.4
22Jul1984	00:00	9.9	2.2	12.1	6.2
23Jul1984	00:00	10.0	2.4	12.3	7.7

Appendix E

Typical Manning roughness coefficient

material	Coefficient
concrete	0.012
Gravel bottom with sides	
concrete	0.02
mortared stone	0.023
riprap	0.033
Natural stream channels	
Clean, straight stream	0.03
Clean,winding stream	0.04
winding with weeds and pools	0.05
with heavy brush and timber	0.1
Fllood plains	
Pasture	0.035
Field crops	0.04
Light brush and weeds	0.05
Dense brush	0.07
Dense trees	0.1