GENERATION OF SOLAR-HYDROELECTRIC ENERGY TO CONTROL THE EXPNASION OF LAKE BESEKA, ETHIOPIA

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(M.Sc in Hydraulic Engineering)

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APPROVED BY BOARD OF EXAMINERS

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1 INTRODUCTION

1.1 Background

The effect of climatic change and environmental degradation has caused the reduction of water level in Lakes, rivers and streams. For instance, Lake Chad in Chad, Abiyata, Hawassa, Ziway, and Haromaya are good examples. However, the water level of Lake Besaka has been rising for more than three decades. The surface area of the Lake has been expanding from 3 km^2 to 46.6 km^2 between 1973 and 2008 with corresponding to rise in Lake water Level (Goerner et al. 2009).

The main cause for the increase in Lake water level and its surface area is the inflow of groundwater from the western part of the watershed. The average annual increment of Lake water level was 0.2 m and hence the Lake had risen by four meters between 1976 and 1997 (Goerner et al. 2009). The expansion and flooding effect of Lake Besaka has become the cause for the loss of 57 human lives, inundation of about 35km^2 of grazing land, and displacement of 910 indigenous people. Methara sugar plantation has also been inundated and the company lost income from 161.55 ha of land (WWDSE 2011). The damages to the nearby railway line and highway caused a loss amounting to 2.6 million US$ (Tessema; Ayenew 2008).

The expansion of Lake Besaka have still continued and severely affecting the life of the surrounding community and also becoming the contaminant to Awash River. To minimize the expansion rate of the Lake, Awash Basin Authority had installed six surface pumps of 1.7m^3/sec capacity and had been pumping water from the Lake and released to Awash River (WWDSE 2011). However, the pumps were fully submerged due to high inflow and consequently the Lake continued to expand and submerge public infrastructures and becoming a contamination fear to Awash River. If Lake Besaka discharges to Awash River by itself without any control, the river is going to be contaminated forever (Wiliam Davison 2013).

Despite, many researchers have conducted studies to mitigate the expansion Lake Besaka and proposed options to maintain the water quality of Awash River, there was no clear demarcation yet put that sustainably mitigate the crisis. However, this research aimed to answer how it is possible to sustainably Control the Expansion Lake Besaka, and maintain the water quality of Awash River through generating hydroelectric power. The research focused on solar energy to pump the Lake Water and generation of hydroelectric power while releasing the pumped water from upper reservoir to Awash River with safe blending ratios.

1.2 Statement of the Problem and Justification

One of the major current problems within Lake Besaka is the rise in the Lake water level and its increase in surface area. The surface area of the Lake has been increasing from 2.5 km^2 in 1960 to 54.8km^2 in 2015 (JICA2015). The increase in water level and surface area of the Lake has been therefore submerging the surrounding public infrastructures and extortion to the water quality of Awash River. Moreover, the expansion of the Lake is instilling terror/fear in both the local/domestic and international investors who have invested near Lake Besaka. Currently, the railway and highway are totally covered by the Lake water and alternative routes were constructed (Haile A. Shishaye 2015). In addition, the rises in water level of the Lake invading sugar plantation farms, and the entire ecology of the catchment.

The expansion of the Lake and its severe water quality problems have been becoming the threat to the Awash River and downstream mega water supply and irrigation projects. The expansion extent of the Lake is very terrible and creates a great development challenge in the Awash River basin. All the beneficiaries of the basin, concerned institutions and the decision makers of the country should consider the condition seriously and adopt appropriate mitigation measures before the Lake brings irreversible damage to Awash River (Olumana and Willibald 2009).

To reduce the drastic expansion of the Lake, Awash Basin Authority installed seven surface pumps and has been draining 1.7m^3/sec water from Lake Besaka to Awash River. However, the pumps were submerged due to their low pumping capacity and high rate inflow to the Lake (OWWDSE 2014).

In spite of the fact that the Lake is alarmingly growing with its problem, there was no study result that yet appropriate to sustainably control the expansion of the Lake without severely affecting the water quality of Awash River. Thus, it is necessary to conduct researches and studies which sustainably mitigate the expansion of the Lake through protecting Awash River from intolerable contaminations.

1.3 Significance of the Study

The study is very important for the country and downstream users to tackle the expansion of Lake Besaka and the damage that it has been imposing to the surrounding public infrastructures and the severe contamination threats on Awash River. It is also vital to specify tolerable mix ratios of Lake Besaka to Awash River which compiles to WHO standards for drinking and irrigation water. The study touches the major causes for Lake Besaka expansion, toxicity and its contamination effect to the downstream of Awash River and surrounding community Mathahara town.

By identifying the major problems that the Lake imposes to the surrounding and downstream users of Awash River, the study will help to inform MoWIE, Awash Basin Authority and policy makers to implement the findings of this research. Because, results of the study put clear outline how it is possible to sustainably controls the expansion of the Lake through mixing it with Awash River and generation of hydroelectric power. The study also creates better clue for the implementers to get better understanding on the practicality of the entire system components, operation efficiencies and schedules.

Hence, the research findings would inform MoWIE, Awash Basin Authority and policy makers to rethink and pinpoint about the damages that Lake Besaka has been imposing and the future crisis it will bring to the surrounding community and Awash River if not timely controlled.

1.4 Objectives the study

General Objective

The general objective of the research is to **Control the Expansion of Lake Besaka** by pumping the Lake water to upper reservoir using solar energy and draining it to Awash River with safe blending ratios through generation of hydroelectric.

Specific Objectives

- Determining the current Lake Besaka surface area and assessing its annual inflow.
- Estimating the safe buffering ratios of Lake Besaka to Awash River.
- Assessing the annual flow of Awash River and solar intensity within Lake Besaka catchment.

1.5 Scope and Limitation of the study

The study was focused on controlling the expansion of Lake Besaka through adopting safe mixing proportions that don't violate the tolerable standards for drinking and irrigation water. It also summarized the methods of controlling the expansion of the Lake and the energy that can be obtained while releasing water from Lake Besaka to Awash River. Besides, study analyzed the capacity of hydroelectric power that can be generated, system components, project cost estimation, and the possible environmental impacts of the project.

Unavailability of organized meteorological data, Lake Besaka inflow data, and updated digitized topographic map of the Lake and previous studies data that conducted on the control Lake Besaka expansion etc. were the major limitations to this study.

1.6 Methodology

This study was conducted using secondary and primary data that obtained from literature review and field work respectively. Secondary data such as digitized topographic map of the Lake and meteorological data were obtained from Ethiopia Geological Survey and National Meteorology Agency respectively. Whereas, mean monthly flow of Awash River and Lake incremental depth records were obtained from MoWIE. The Lake annual inflow and water quality data for safe mixing of Awash and the Lake were adapted from literature review. Whereas, elevation data for upper reservoir, length of penstock, pressure line and drainage route data were collected from the field using GPS.

The activities are classified into office and field works;

Office Work

- Collection and organization of pervious works within and round study area.
- Analyzing current Lake Besaka size and reviewing the annual inflow to the Lake.
- Estimating net solar radiation of the study area and designing solar energy for pumping.
- Analyzing the water quality data of and deciding the tolerable proportions to mix water from Awash River and Lake Besaka wet and dry seasons.
- Estimating the size of upper, geological conditions and the hydroelectric power that can be produced while draining the Lake water to Awash River.
- Calculating the water application efficiency of Fantale irrigation project and its contribution to the expansion of Lake Besaka.

Field Work

- Collecting elevation and length of penstock, pressure line and drainage pipe routes.
- Observing the current damage scale of Lake Besaka and the future consequences it can bring if appropriate mitigation measures has taken.
- Taking the sample elevation data on the periphery of Lake Besaka to correlate with the produced map using GIS.
- Collecting water sample data and conduct laboratory test to correlate the tests results with data obtained from literature review.

1.6.1 Material Used

To achieve the objective this research work, the following materials were used.

1. **Digital materials**
   - **i. Hardware components**
     - Computer
     - Printer
     - Scanner
   - **ii. Software component**
     - ArcGIS 10.1

- Global mapper3.1
- Water CAD 6.5
- Surfer

iii. Auxiliary materials

- Aerial photographs
- Digitized topographic map of study area.
- Geological map of study area.
- Local geological map of mount Fantale.
- Field equipment (GPS, Meter and Digital Camera)

1.7 STRUCTURE OF THE THESIS

The Thesis was organized in six (6) chapters. After the introduction, general problem and research objectives were outlined in (chapter 1); chapter 2 states the general description of project area. Chapter 3 deals with the Literature Review on the issues that clearly state the effect of Lake Besaka expansion on public infrastructures, ecosystem, sugar plantations and the measures yet has taken to mitigate the consequences. Computation of Solar energy plant capacity for pumping the Lake water to upper reservoir and hydroelectric power that can be generated while releasing the pumped water to Awash River and Estimated capital Cost of the Project were presented in chapter 4 and 5 respectively. Chapter 6 is devoted to Environmental impact that may occur if the project under study will be implemented.

Finally the paper was summarized by conclusion and recommendation.

2 DESCRIPTION OF THE AREA

2.1 Location and Climate

The Lake Besaka is situated at the center of the Ethiopia Rift Valley about 190km east of the capital Addis Ababa. The Lake watershed lies between 39°43'-39°59' East longitude and 8°41'-9°0' north latitude. The surface area of the lake has grown dramatically within the last three and half decades and now covers about 11.67% of the total watershed area. The Lake receives the surface runoff during wet season from a watershed of estimated area 532km^2 and the lake is located at the eastern part of the basin. Two towns, Mathara and Addis Ketema are located close to the Lake.

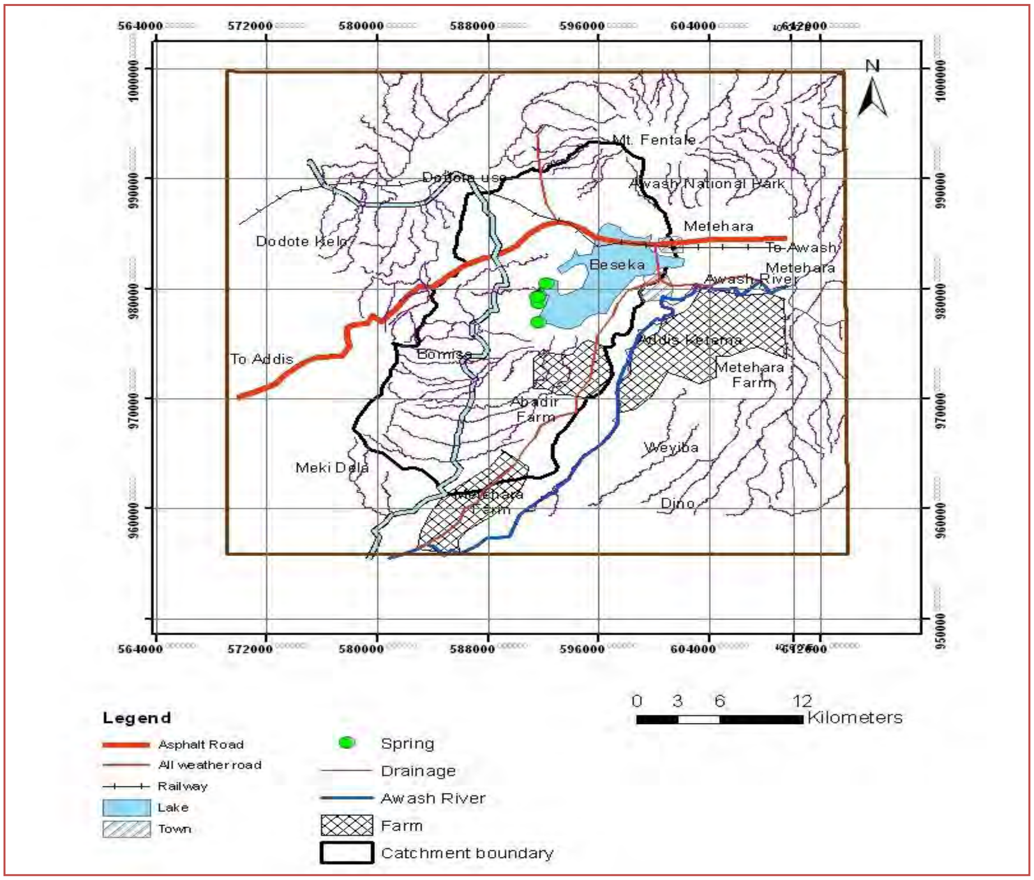


Fig.2.1a. Lake Besaka, its watershed and surrounding Infrastructures (OWWDES 2014)

The seasonal migration of the Inter Tropical Convergence Zone (ITCZ) controls rainfall patterns in most parts of the country. During the summer, the ITCZ is located in northern Ethiopia, and the region is under the influence of moisture from the Atlantic and Indian Ocean (Kebede, 2004). During the months of October to February, the ITCZ migrates to the south of Ethiopia, and most of the country is characterized by dry air. The northward movement of the ITCZ from March to April brings moisture from the Indian Ocean, which results in small spring rain. Rainfall in the western-most sector of Ethiopia is mainly in summer (July -September), while in the eastern part of the country it is in spring (March – May) and in October.

The central part of the country is characterized by a bimodal rainfall pattern, with both summer and spring rainfall. The study area is located in the center of Ethiopia, is influenced by two moisture sources, and characterized by two rainy seasons in summer (July - September) and spring (March - April). The climate is semi-arid with a mean annual temperature of 25°C and a total mean annual rainfall of 534 mm.

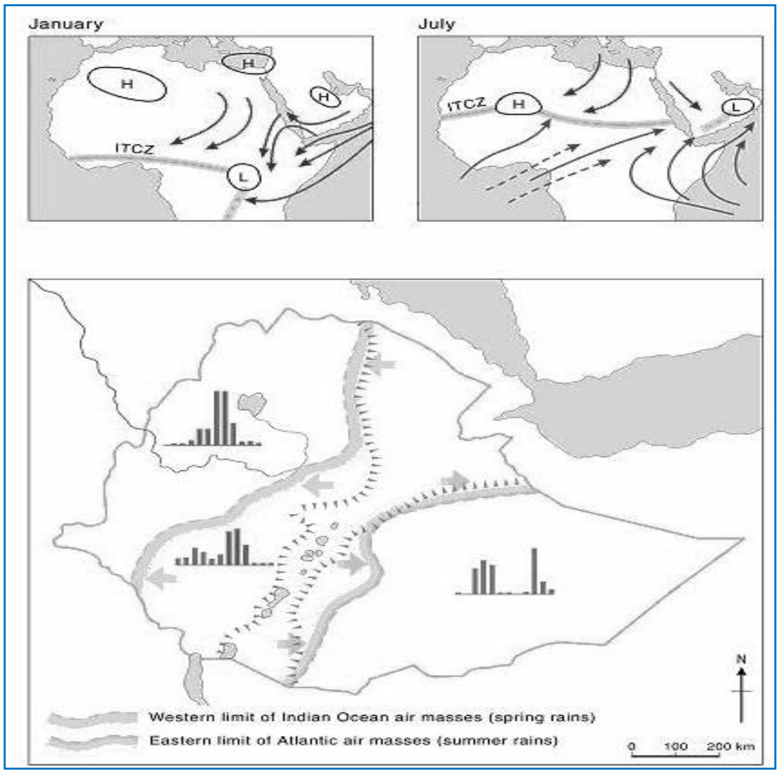


Figure 2.1b: Seasonal drifting of the ITCZ in Ethiopia (Lacaux et al, 1992)

Long-term (1966-2007) mean monthly records of temperature, relative humidity; wind speed and sunshine hours were analyzed. The study area is characterized by an average daily maximum and minimum temperature of 33°C and 17°C, respectively. The lowest temperatures are between November and January, while May and June are characterized by higher temperatures. Relative humidity is high during the main rainy season. Wind speed is at a maximum during the months June and July and at a minimum value during October. Long- term average sunshine hours in the study area are 8.4 hours, and the lowest sunshine hours being during the main rainy season. Potential Evapo-transpiration exceeds monthly rainfall in nearly all months.

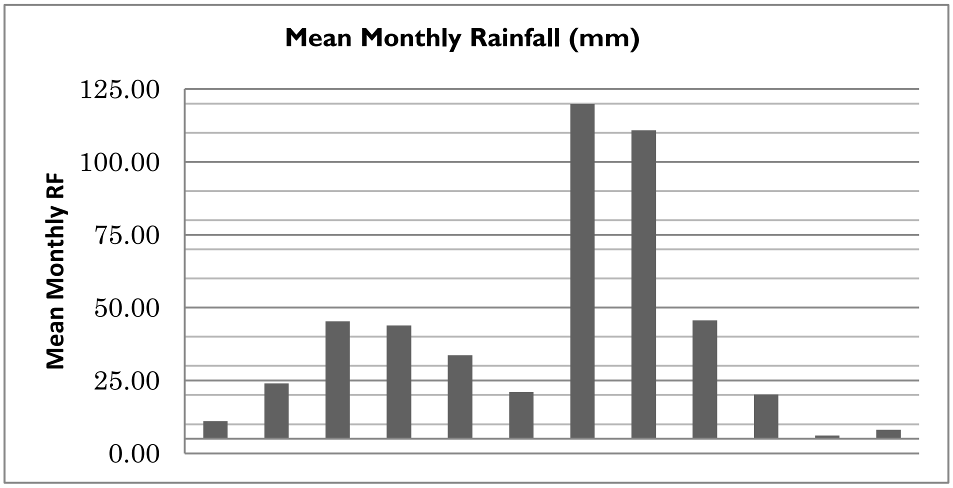


Fig.2.1c. Rainfall Seasonal Variation in the area from Jan-Dec. Left to right (1985-2014)

The mean monthly maximum temperature of the study area varies from 31°C to 36°C; while the mean monthly minimum temperature varies from 13°C to 22°C.

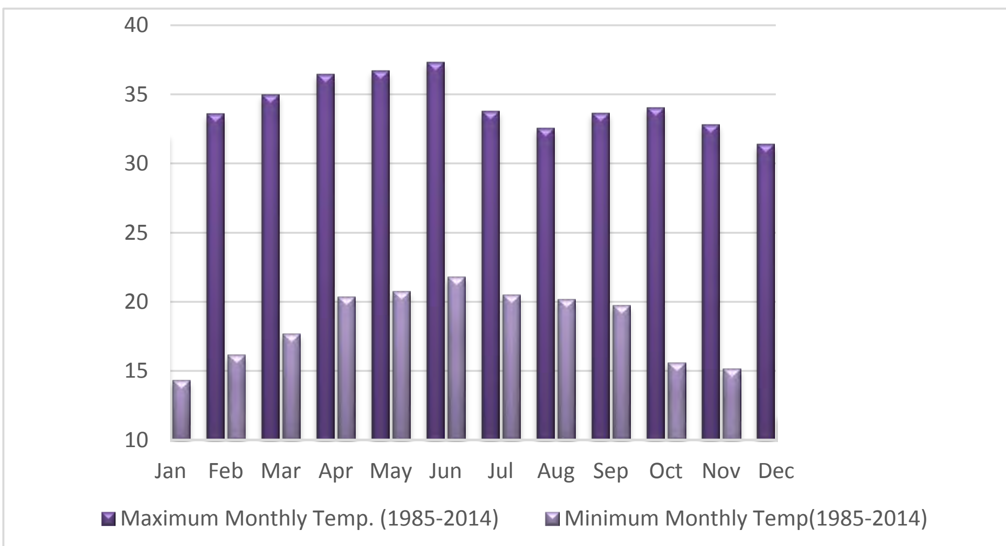


Fig.2.1d. Temperature Variation in the area (1985-2014)

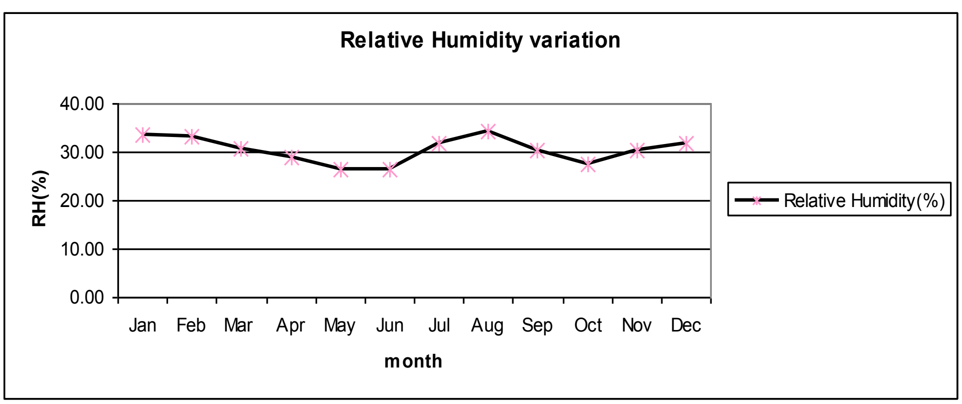


Fig.2.1e. Related Humidity Variation

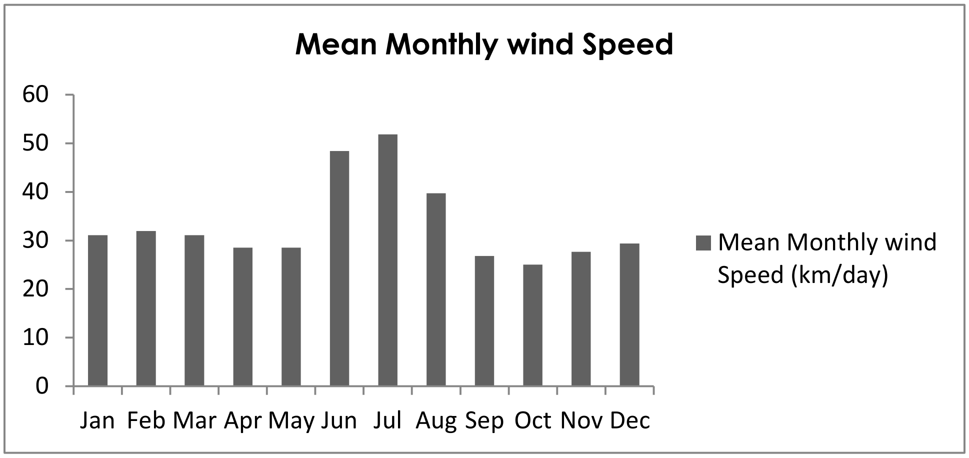


Fig.2.1f. Mean Monthly Wind Speed

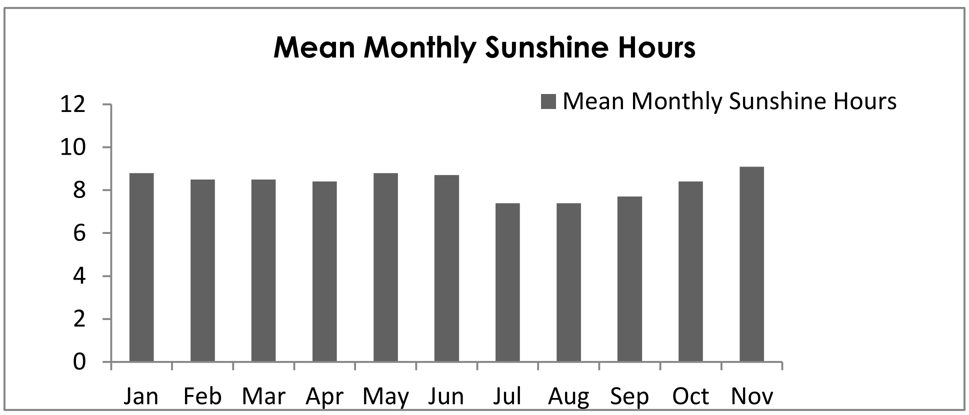


Fig.2.1g. Mean Monthly sunshine hours

The topography of the Lake Besaka area ranges from flat to undulating plains and from hills to the high Mount Fantale. Most of the watershed is characterized by flat to undulating plains with altitudes ranging from 940 to 1200m above sea level. Plains with small and high gradient are located in the northwestern and western part of the watershed. Volcanic cones are also concentrated in the western part. Hills with medium slopes are located in the south, while a high slope mountain is situated in the northern part of the watershed.

2.2 Topography

Watershed is a natural topographical and hydrological entity which collects all the rainwater falling on it to a common outlet and hence forms an ideal unit for management and sustainable development of its natural resources like water, soil, land and vegetation (Goswami, 2004). Generally, watershed is the boundary of the drainage basin. According to Schramm, 1980 watershed is classified depending on size of the catchment. The upper reaches of the main drainage system of the sub basin are almost steep and mountainous, while the lower part of the drainage system is flat with broad Valley. The altitude of the study area ranges from 950 to 1200m above mean sea Level.

2.3 Soil

According to the FAO soil classification, the soil mapping units have been identified in the study area at a scale of 1:250,000. From these soil classifications map units, broad divisions of soil types in to different families were grouped as silt clay, clay loam, and sandy loam.

Sandy loam Silt- loam soils are covering the central part of the study area occurring under an arid moisture regime. Clay loam soils cover most of the boarder part of the study area especially on the high lands of the project area. Silt clay soils cover the western boarder part of the area.

Table 2.3a. Summary for major soils of the study area (Eleni Alemayehu 2009)

Major soil groups		Drainage
Chromic Luvisols,Dystric nitosols	SiL	Somewhat excessive drained
Eutric fluvisols, Eutric, cambisols	SiL-SL	Well, moderately, very poorly, imperfectly drained
Vertic, Andosols	CL-C	Well, moderately, excessively drained
Leptosols	CL-SCL	Well, moderately drained

Remark: - S-Sandy, SCL-Sandy Clay Loam, L-Loam, Si-Silt, C-Clay

The soil map of the study area was determined based on the above classification of soil texture. This map is used to compute actual evapo-transpiration of the sub catchment.

2.4 Land-Use and Land-Cover

The land-use and land-cover (LUC) of Lake Besaka watershed pattern was mapped in different perspectives from 1999 WWDSE to the recent studies of OWWDSE based on aerial photos and field investigations. The areal extent of land use and land cover mapping units in the sub basins is highly dependent on the climatic, topography and edaphic factors. Open bushy woodland is the most dominant LUC type, accounting for 46% of the total watershed area. This LUC unit is typically found covering the shallow soil of the plain, medium to high gradient hills, volcanic cones and Mount Fantale. This unit is mainly used for grazing, firewood collection, and charcoal production, and for very limited rain fed agriculture.

Dense woodland, open grassland and open grassland with bare rock are common LUC types covering 24% of the total lake watershed area. Dense woodland is commonly found in the northwestern part and covers 8.7 % of the total lake watershed area. Population, remoteness and traditional factors attribute to the type of use and the natural vegetation as they are presently expressed in the sub basins.

In the arid and semiarid areas grasses, shrubs, and sparsely cultivated lands are common. On the extreme where the climate is arid, salt flats, exposed rock or sand surface is predominant land cover. The wide occurrence of shrubs and grass land is perhaps associated with the less population pressure and cultural orientation of the people, which is of pastoral farming system. The reclassified land use/cover map is used for the calculation of actual Evapo-traspiration from the watershed.

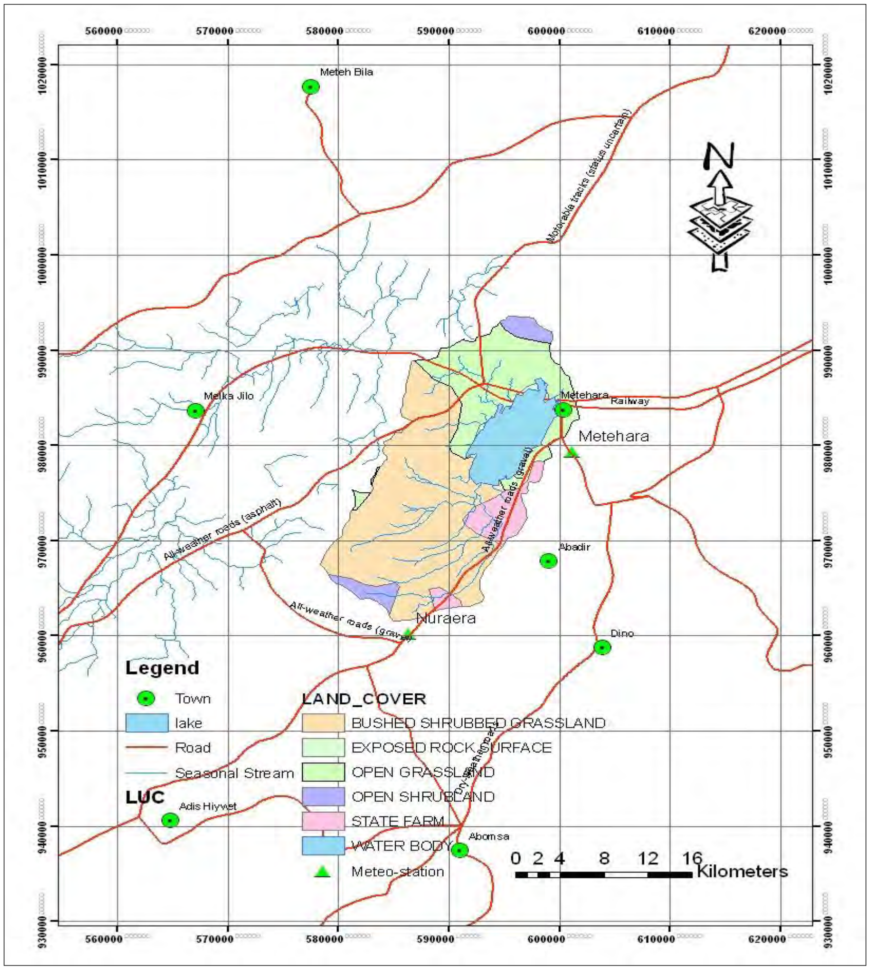


Fig.2.4b. Land-use and Land-cover of Lake Beseka Catchment (OWWDSE-2014)

2.5 Geology of the Study Area

As Lake Besaka is located in Rift Valley and active volcanic terrain, different study documents and researches conducted in Lake Besaka watershed were reviewed so as to know especially the geological condition of upper reservoir and power houses.

2.5.1 General Geology

Three important rift structures, two oceanic (Red Sea and Gulf of Aden) and one continental (East African Rift), come together to form very complex geological features. The Ethiopian rift system is part of the EARs, a complex tectonic feature with a system of down faulted, non-continuous but related troughs. The tectonic and volcanic processes of Ethiopian Rift systems are not uniformly continuous; rather they are characterized by distinct periods of increased and decreased activity (Elc Electoconsult, 1987).

2.5.2 Local Geology of Upper Reservoir

The geology of Lake Besaka area reviewed in depth from the perspectives of upper reservoir structure foundation stability, turbine-pump and power house foundation conditions. Due to tectonic volcanic nature of the area, detail literature review of area geologic conditions were considered and correlated to investigate the risks of fault and major fractures for foundation instability.

Volcanic and seismic activity and faulting are phenomena linked to the geologic processes in the axial part of the rift resulting from strain localization after the abandonment of the border faults during the Pleistocene. Acidic to basic lavas and pyroclastics characterize the volcanic products which last occurred in the 1830's. These and the presence of thermal manifestations in the area attest to its active state. Most of the diverse Lithologic units, like scoriaceous and basalt have considerable porosity to allow water storage. Secondary permeability induced by faults, open fissures and joints seems to play a determining role in the hydrology and hydrogeology of Besaka and its surroundings.

Even though interfering of various lava flows and pyorlcastic deposits from diverse sources and lacustrine sediments (some of which were accumulated uninterruptedly), makes it difficult to establish a clear volcanic stratigraphy, the lithologic units exposed in the Besaka area from the oldest to the youngest are described hereunder, the final local geological investigation of conducted in the Lake watershed indicated as the foundation for upper reservoir is safe from faults and from faults and other geological successions as shown below. Besides, some foundation improvements such compacting and lining work will be made to increase the water tightness of the upper reservoir.

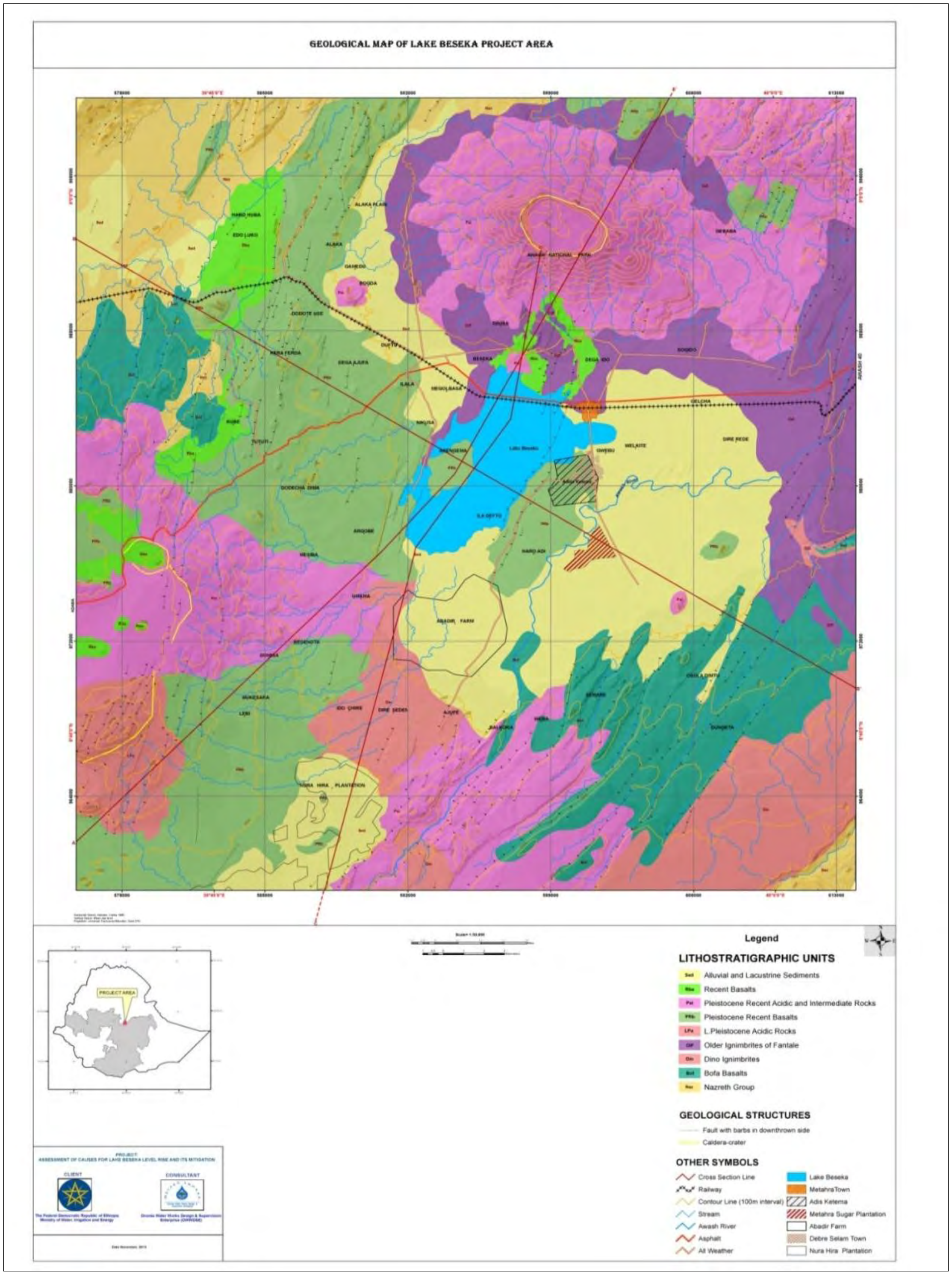


Fig. 2.5.2a. Local Geological map of Lake Besaka area (adapted from OWWDSE, 2014)

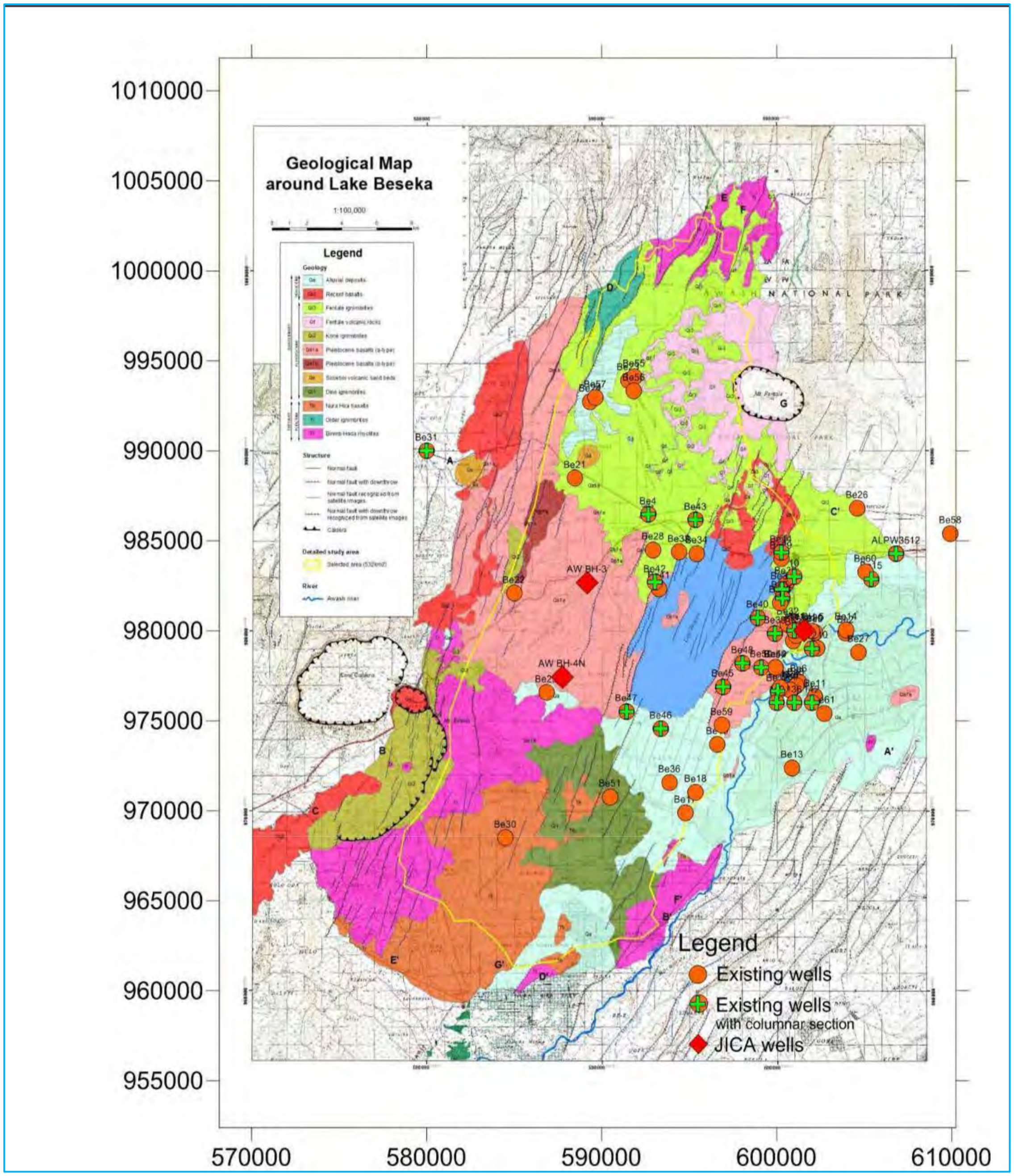


Fig. 2.5.2b. Geological map of Lake Besaka and its surrounding (JICA 2015).

3 LITERATURE REVIEW

3.1 Causes for the Expansion of Lake Besaka

The main cause for the expansion of the Lake surface area and rise in level is the increased groundwater flow from the western part of the watershed. The discharges to the Lake from hot springs constitute as the major source and it is estimated to be 51% of the total inflow to the Lake (Belay 2009). However, some investigators relate the phenomena to neotectonism (Ayenew1991; Tessema 1998). The average annual increment of the Lake was 0.2 m and the level of the Lake has risen by four meters from 1976 to 1997 (Goerner et al 2009). The expansion and flooding of the Lake became a reason for the loss of 57 human lives, inundation of about 35 km^2 of grazing land, and displacement of 910 people. Methara sugar plantation has also been inundated and the company has lost income from 161.55ha of land (WWDSE 1999).

The starting time of Lake Besaka expansion is not exactly known. However, most previous studies tend to agree that the problem has initiated in 1964 when the Methara mechanized farm around the lake was started to be irrigated for cultivation of cotton and citric fruits and latter shifted to sugarcane development (Ayenew 2007).

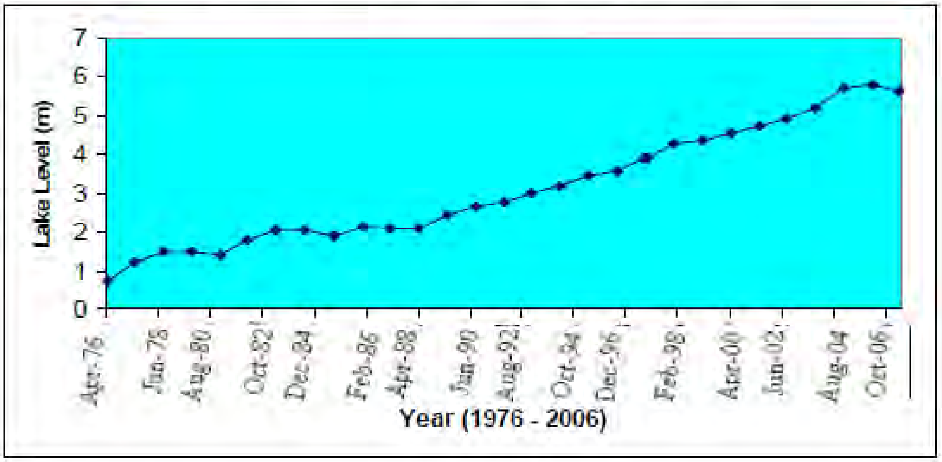
The main changes in the water balance of Lake Besaka comes from groundwater inputs, which is related to the recent increment of recharge from the irrigation fields and construction of Koka dam located at 152 km upstream. Some authors relate the expansion of the Lake to neotectonism (Ayenew, 1998; Tessema, 1998). Prior to the construction of the Koka dam Awash River could sometimes go dry between December and March. However, after the construction of the dam there has been fairly steady flow throughout the year. Hence, the regulated flow has become a source of continuous recharge to groundwater ultimately feeding the Lake. Recent estimation of the water balance shows that groundwater contributes 50% (53.8 MCM/yr) input to the Lake and 64% of the groundwater input to the Lake inflow comes from outside the catchment area i.e. the Awash River transmission loss and irrigation loss accounting 23.5 and 10.5 MCM/yr respectively (Tessema, 1998). Irrigation excess water discharged into the Lake was estimated to be in the order of 20 MCM (Halcrow, 1989). The reason for this has been poor irrigation efficiencies of Matahara sugar plantation.

The transmission loss from the Awash River and direct recharge are facilitated by the presence of modern active tensional faults. Hence, the favorable geological factors combined with the availability of water have enhanced the modern recharge. Isotopic and geological evidences have shown the occurrence of modern and sub-modern cold water and thermal water. As evidenced from isotope and hydro-chemical data and reconstruction of the piezo-metric levels groundwater flows into the Lake from the western side. The Lake level has raised by 4m during 1976-1977 as evidenced from Lake daily stage records. The hydrograph of Lake Besaka (starting in 1964) shows that the early part is gentler followed by steeper rise in recent years. The average Lake level rise is 0.20m/yr.

Table.3.1a. Expansion pattern the Lake Besaka and change in depth at different years (Abdi 2007)

Year	Apr-76	May-77	Jun-78	Jul-79	Aug-80	Sep-81	Oct-82	Nov-83	Dec-84	Jan-85	Feb-86	Mar-87	Apr-88	May-89	Jun-90	Jul-91
Lake level (m)	0.71	1.21	1.46	1.48	1.4	1.76	2.03	2.05	1.91	1.9	2.13	2.07	2.09	2.44	2.66	2.776

Aug-92	Sep-93	Oct-94	Nov-95	Dec-96	Jan-97	Feb-98	Mar-99	Apr-00	May-01	Jun-02	Jul-03	Aug-04	Sep-05	Oct-06
3	3.18	3.45	3.55	3.89	3.91	4.27	4.36	4.55	4.74	4.93	5.19	5.72	5.78	5.65



By the end of 1997 the elevation of the Lake was 952.4m above sea level (m.a.s.l). Inspection of 1:50,000 topographic map show the lowest point along its water divide is 954 m a.s.l to the northeastern side. The Lake level is therefore 1.6m below the lowest point; if the inputs to the lake continue with the same rate, it will overpass the divide by the year 2008. If inputs increase more the overflow could occur shortly. Recently the government has proposed pumping out and releasing the Lake water into the Awash River, although the ecological effect downstream is unknown (Ayenew 2007).

The inflow to the Lake Besaka is continued expanding and increasing in linearly pattern.

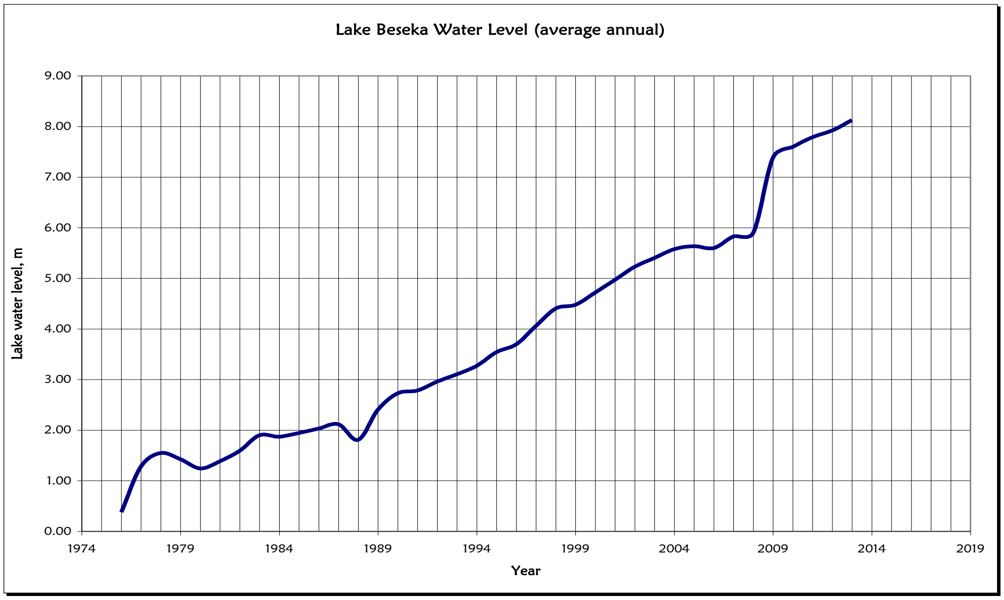


Fig. 3.1a. Lake Besaka water (1976-2013) (OWDDSE 2014)

3.2 Dependent flow system of Lake Besaka

Spatially dependent flow system analysis of the Lake indicates gravity is the dominant driving force in groundwater movement in an unconfined system. It moves downhill until in the course of its movement, it reaches the surface at a spring or through a seep along the side or bottom of a stream channel or estuary (Heath, 1983). The groundwater flow pattern is controlled by the configuration of the water table, which in turn is controlled by the prevailing climate, topography, and geology (Sophocleous, 2004).

Climatic factors, precipitation and evapotranspiration, mainly controls the rate of recharge, while topography and geology determine the spatial distribution of discharge and recharge zones (Dingman 2002). A groundwater contour map of the study area was prepared from the groundwater level data of 78 boreholes, which are located inside and around the Lake watershed (Alemayehu 2009).

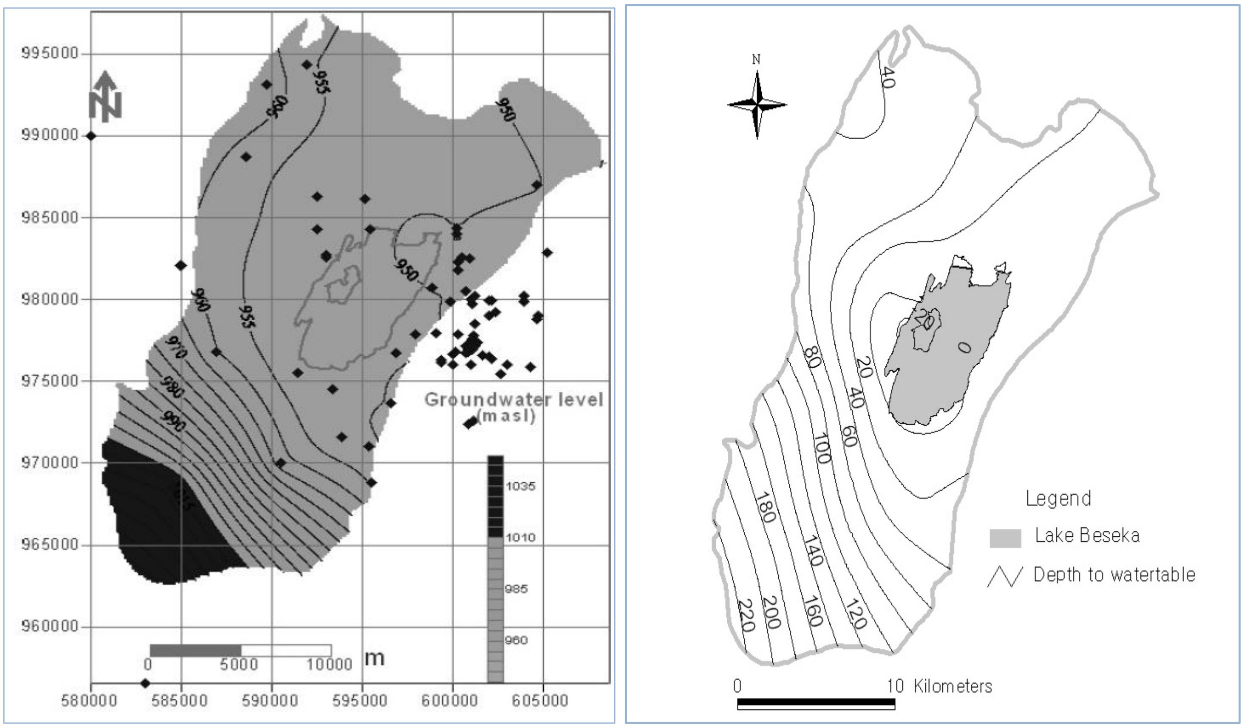


Fig.3.2a. Contour of water table and borehole locations in Lake Besaka (Eleni Alemayehu 2009)

The contour map describes the groundwater flow system where the system is in equilibrium with its fluxes, and the flow is governed by the medium properties of the aquifer. The groundwater level is higher in the topographically elevated area, in the south and northwestern part of the watershed compared to the area around Lake Besaka. And these areas can be considered as recharge zones. These zones are topographically higher areas, and are characterized by a deeper water table compared to the lower areas around the Lake.

In recharge areas, the unsaturated zone between the water table and land surface there is often a deeper than in discharge areas (Fetter, 2001). However, tectonic structures are abundant features in the study area, and can favor direct and localized rainfall recharge to the groundwater system (Alemayehu 2009).

In the study area, the groundwater flows from the south and northwestern part of the watershed toward to the Lake. The groundwater level is high in the south and northwestern part of the watershed and becomes lower toward Lake Besaka and the eastern part of the watershed. The groundwater level is at its lowest east of Lake Besaka, around the Methara town area, and this may induce groundwater outflows from the Lake. Part of the basin around Lake Besaka and east of the Lake can be considered as a discharge zone. The southwestern and western border of the lake is characterized by widespread hot springs. In discharge areas, the water table is usually at or near the surface, and such areas are usually the sites of streams, Lakes, wetlands, or springs (Dingman, 2002).

The groundwater contours of the study area are influenced by tectonic structures, as can be seen west of Lake Beseka, where hot springs are aligned along fault lines. Hot springs along tectonic lines are evident around the Tone area (southwest) and the northwestern part of the

Lake, where two of the hot springs are currently submerged. The groundwater flow direction in the study area is comparable to the surface water flow direction, and is controlled mainly by topographic, geologic and tectonic features (Alemayehu 2009).

In spatially dependent groundwater flow systems, flows are mostly dependent on the hydraulic conductivity of the aquifers. The rate of movement of groundwater from recharge areas to discharge areas depends on the hydraulic conductivity of the aquifers (Heath, 1983). Thus, assuming similar recharge values, hydraulic conductivity values can be roughly interpreted from groundwater contour maps. The northwestern and eastern parts of the Lake watershed are characterized by low hydraulic gradients in contrast to the southwestern part. These gently sloping groundwater contours around Lake Besaka area can be related to the existence of relatively high hydraulic conductive zones compared to the closely spaced contours in the southwest.

3.3 Inflow to Lake Besaka

The inflow to Lake Besaka has been contributed from different water sources that the flow gradient permits to flow the Lake as shown.

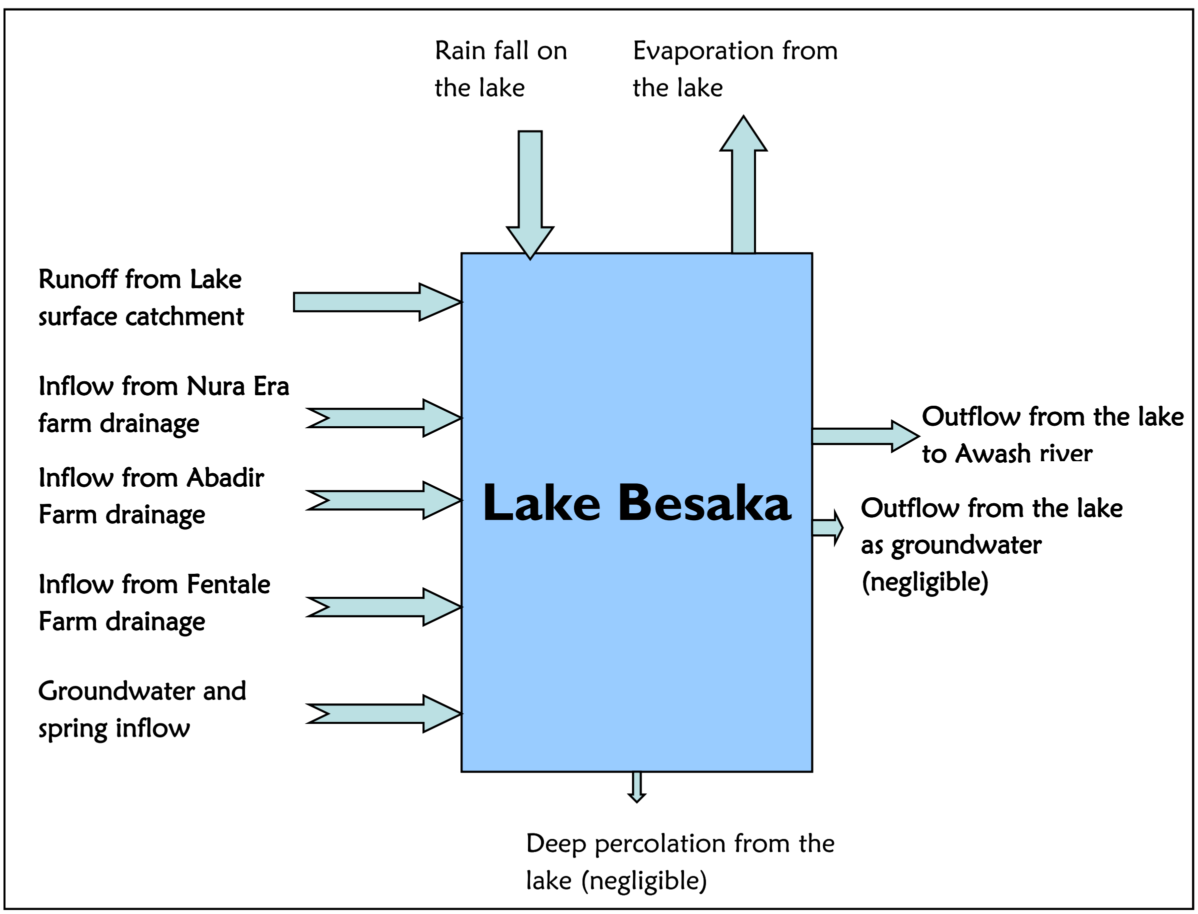


Fig.3.3. Major Inflow and outflow components of Lake Besaka (OWDDSE2014).

In 1977 and 1978 the inflow from Abadir farm was significant; the irrigation efficiency of Abadir farm was 54%, and the remaining 46%, accounts the water volume that disappear without contributing to plant growth. For annual average water intake of 102 MCM, annual loss is about 47MCM. This accounts about 35 MCM (= 47MCM x 73.3%) is a loss amount of water generated in the Lake Besaka basin, and this is the largest irrigation surplus water return flow that may flow from Abadir farm to Lake Besaka (JICA 2015). In 2007, Nura Era drainage canal that diverted to Awash River was silted up and it was flowing to Lake Besaka for almost one year until it was maintained and diverted to Awash River (Russom G.E and Engida Z.A, 2009).

The Fentale main canal as well as the irrigation farm is on porous media and most of the drainage water to Lake Besaka inflow as subsurface inflow. The average annual discharge to Lake Besaka from groundwater and springs estimated by Darcy Approach method of MoWR, 1998 was 47.3 MCM/year. This inflow amount highly agrees with the water balance computation result of the same year. A pumping station having 8 (eight) pumps (3 of them 300 l/s and 5 of them 166 l/s), with a maximum capacity 1.73 m^3/s and minimum of 0.166m^3/s were installed and operational from 2007 to 2008.

The amount of Lake Water discharged to Awash River is in 2007 – 11.5 MCM and 2008 -13.92 MCM. In 2009, the pump station is flooded by the Lake level rise and the pumps were dismantled (OWWDSE 2014).

3.4.1 Lake Besaka Water Balance

Lake Besaka has been a closed Lake until 2006 and open Lake since 2007. Lake Water balance was carried out for the different scenarios of the Lake conditions (JICA 2015, OWWDSE 2014). If the meteorological and hydrological situation such as the external conditions if stable, long-term Lake area is considered to be stable at equilibrium Lake area. Even if there is and some of the seasonal variation to be short-term scaling in floods and low rainfall, etc., thereby departing from the area in the long-term perspective is considered. A result of meteorological and hydrological analysis, annual average rainfall of Metehara point is 508 mm, the average annual amount of evaporation has been calculated to be 3.023 mm, to apply it to Lake Besaka basin (JICA 2015).

In general, the following conditions remain valid for different Values of Equilibrium Lake Area equation shown below:

$$E \cdot ELSA = R \cdot ELSA + Q$$

Where:
ELSA= Equilibrium Lake Surface Area
E=Annual amount of Evaporation
R=Annual Rainfall
Q=inflow from year of basin

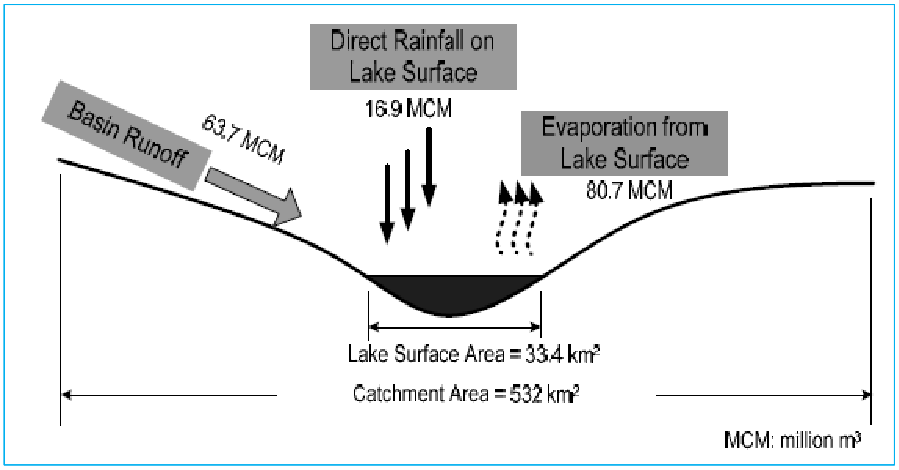


Fig. 3.4.1a. Lake Besaka water Balance when **ELSA=33.4km²**(JICA2015).

$$C = 1.090(A - ELSA)^{-0.236}$$

Where: A=Lake catchment area, 532km²

Table 3.4.1a. Lake Besaka water Balance at **ELSA=33.4km²** (JICA 2015)

Item	Volume	Calculation
Direct rainfall on the Lake Surface	16.9MCM	508mmx33.4km²
Inflow from watershed	63.7MCM	508mmx1.090(532km²-33.4km²) $^{-0.236}$ (532 km² – 33.4 km²)
Total inflow	80.7MCM	
Outflow		
Evaporation from Lake Surface	80.7MCM	3.023mmx0.8x33.4km²
Total outflow	80.7MCM	
Net balance	0	

Rainfall on Lake Surface and runoff from effective catchment follow the same trend as that of storage change. Low rainfall periods relatively manifest high Lake Water evaporation and high rainfall periods relatively experience low Lake Water evaporation. Generally, high temperature, low relative humidity and strong winds result in higher evaporation loss and reduced storage change could be observed and the change in the storage of the Lake was convergent to zero (OWDDSE 2014).

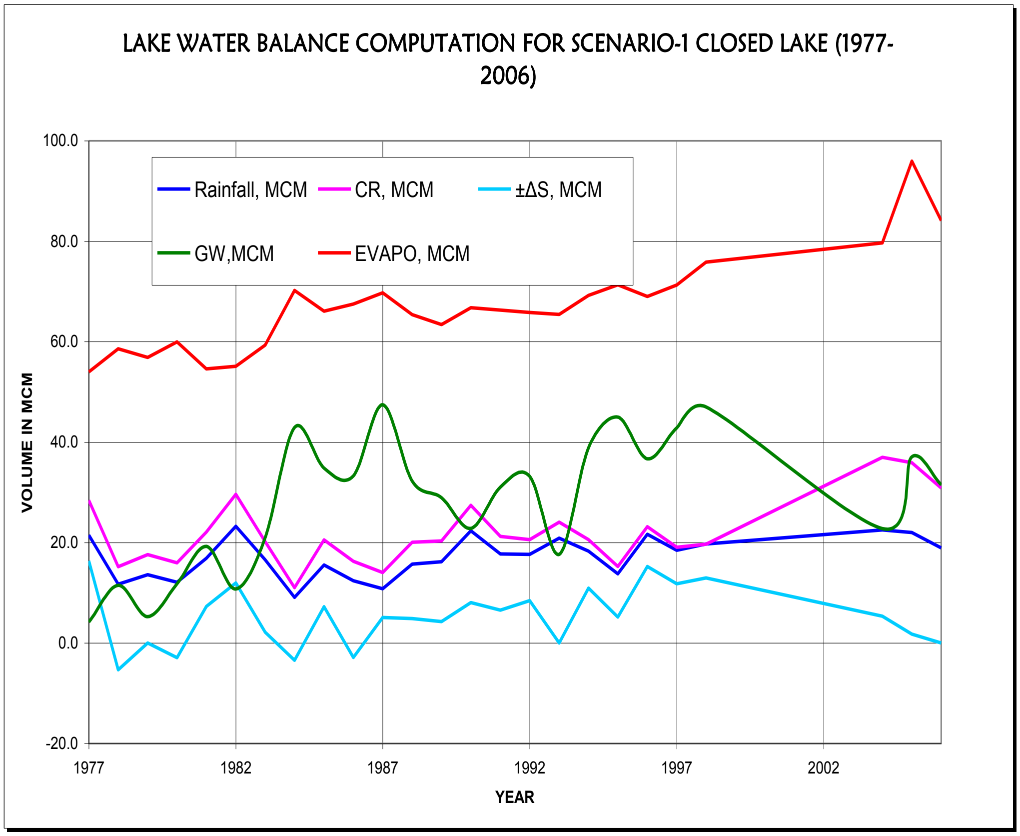


Fig.3.4.1b. Lake Besaka water Balance for Scenario I (OWWDSE 2014)

However, since 2007-2008 the Lake was pumped to Awash River and starting from 2009 the Lake started to flow by gravity through the pumping scheme to Awash River and the Lake become an open Lake. During this period additional inflow of Nura Era farm drainage water was diverted to Lake Besaka and Fentale Farm become operational and significant inflow to the lake occurred, the Lake was pumped to Awash and finally the Lake naturally started to overflow to Awash River (OWWDSE 2014).

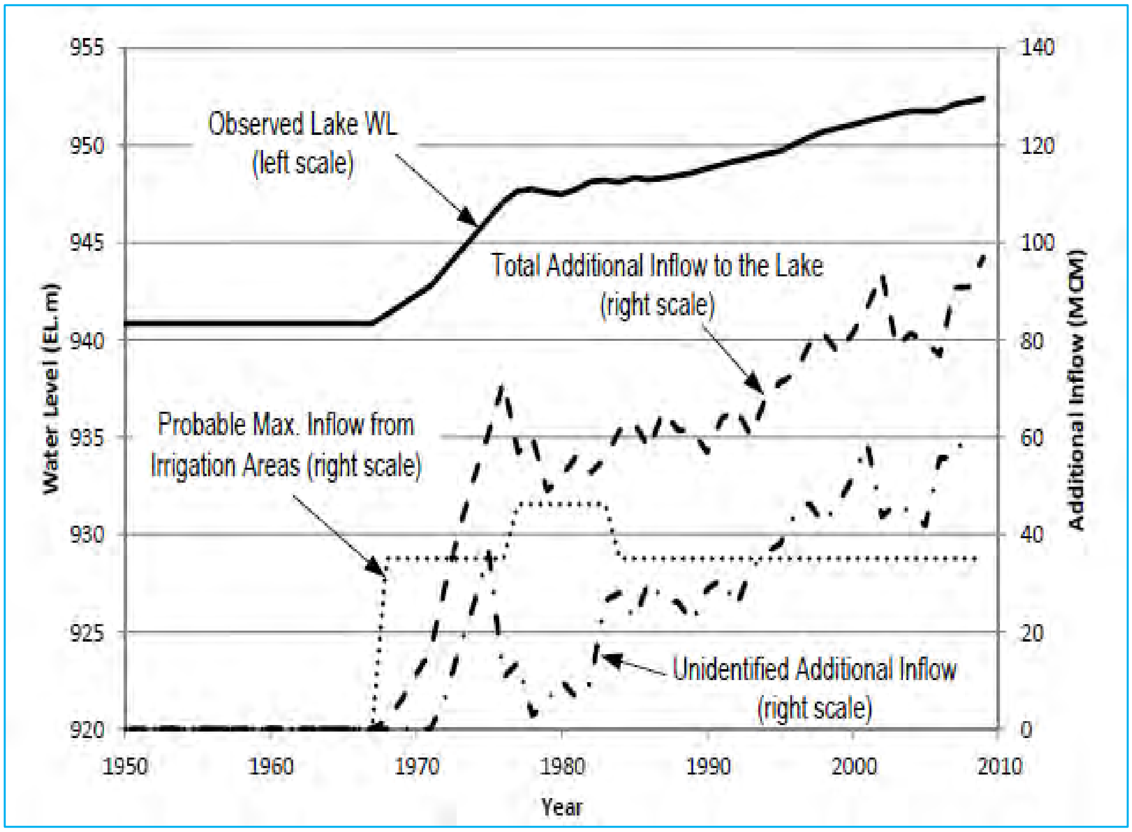


Fig. 3.4.1c. Annual water balance of Lake Besaka (1950-2010) (JICA-2015)

Since 2009, Fentale farm drainage water that inflows to Lake Besaka was very high in magnitude and caused an abrupt water level rise of Lake Besaka i.e. the estimated inflow to the Lake was 78.5 MCM in 2011 and 86.0MCM in 2012 (OWWDSE 2014).

3.4 Effects of Lake Besaka Expansion

3.4.1 Effect of the on Matahara Sugar Planation

The highly saline Lake Besaka expansion is expected to affect the groundwater dynamics of the sugarcane plantation fields. The maps of GW table depth and salinity presented in figure below were produced based on the ordinary interpolation ArcView 3.3 from the six month average monthly point measurement piezometer data. As shown in the figure, most of the GW table depth of the area is categorized as shallow (< 3m). It is under severe condition(< 1 m) at the Abadir extension (South of Lake Besaka), North, East and Awash sections of the plantation and in moderate range (13m) in the other areas (away from water bodies) according to the FAO/UNEP (1984) guidelines (Masoudi et al.,2006).

In general, the GW table depth and salinity is better correlates with the slope of the area. Lower altitudes with relatively shallower GW table depth have severe salinity than the higher altitudes. Almost all plantation fields have certain amount of GW salinity problem ranging from moderate to severe according to the FAO (1976) guidelines.

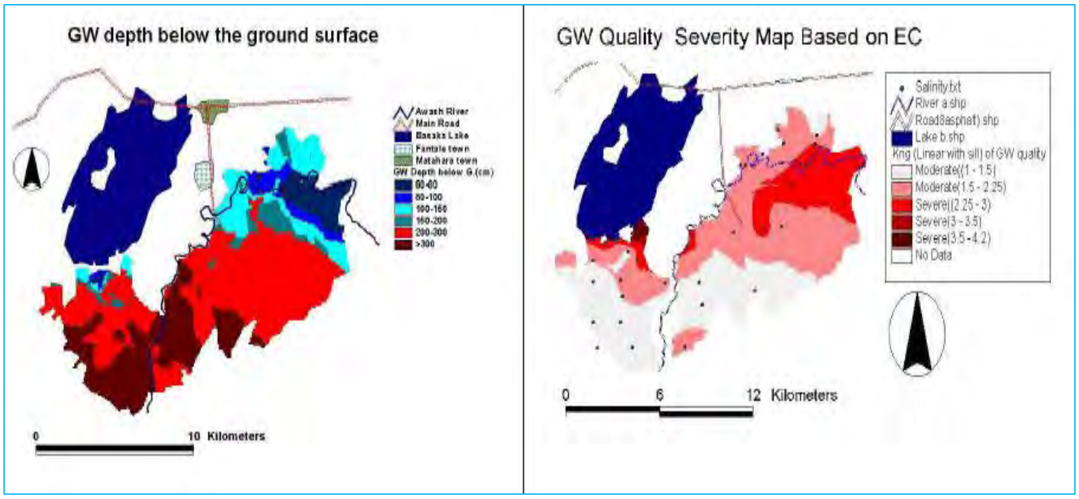


Fig 3.4.1a. GW table depth and GW salinity of the farm (Olumana and Losikandl 2009)

The GW table depth less than 3m is expected to contribute to the crop Evapotranspiration (Kahlown et al., 1998; Kahlown et al., 2005), which is maximum when the depth is less than 1.0 m. For sugarcane crop, the GW contribution increases as function of increment in GW table depth (Kahlown et al., 2005). The saline GW(EC >4 dS/m) will result in a decrease of sugarcane yield when the GW table depth is less than 2m below the ground (Kahlown and Azam, 2002).This is also true in the Matahara sugarcane plantation, the GW table depth below the soil surface & groundwater electrical conductivity (EC) showed a strong negative correlation (r = 0.83).

That means the contribution of salinity to the crop root zone is significant as the GW table depth become shallower. The expansion of the lake is interfering with the production & productivity of the sugar plantation. The sugarcane growth is observed to be stunted and out of production. Significant cultivated fields are abandoning in Abadir ext areas because of salinity and the GW table is coming to the surface. The Lake has inundated the total command area of 381.5ha (OWWDSE 2014).

Fig. 3.4.1b. Sugar plantation affected Lake Contamination

3.4.2 Effect of Lake Besaka Expansion

Since 1967 the majority of Lake Besaka flooding in the surveyed locations has been slow but sure in origin (Besaka overtopping their banks), but localized Besaka Lake flooding (irrigation drainage combined with surface flow in streets) has also occurred, most notably in the end of 2001. Metahara town with more than 30,000 residents, at risk as some part of the town already inundate. There were 419 built up area and individual dwelling houses inundated with water (OWWDSE 2014).

Fig.3.4.2a. Impact of Lake Besaka on Matahara water supply (WWDSE2014)

Besaka Lake spill also risks Oromia National Regional Government efforts to expand and assure quality of education in Matahara town by Submerging Gudina Tumsa High school that serve the town and the environs community.

Fig.3.4.2b. Submerged Public school(OWDDSE2014)

The expansion of the Lake water had dreadful treats for private investments also. The resorts found at western part of the lake under construction which was reported to be property of Saudi American investors is inundated by the Lake water.

Fig.3.4.2c. Submerged public investment (OWWDSE2014).

The health, social and economic impacts of major Besaka Lake are devastating, affecting people's physical and mental health and creation of stagnant water here and there which is supposed by health professionals in the districts host for breading mosquitoes that transmitted malaria.

Fig.3.4.2d. Submerged public health center (OWWDSE 2014)

Though the railway and the road have been elevated recently, it is still in jeopardy by inundation. The roads built were invaded by Lake Besaka water volume augmentation there by becoming inaccessible or they were cut off that forced to redirect the road running from Addis Ababa to Djibouti, which led to addition cost and blockade for trafficking as the root is road that transport import and export items from Djibouti.

Fig.3.4.2e. Submerged railway access (OWWDSE2014)

Power transmissions are under treat due to enlargement of lake water. The electric poles are inundated by lake water increase and enquire huge amount of cost to find alternate for power transmissions.

Fig.3.4.2f. impact of Lake Besaka on Electric Poles (OWWDSE2014)

3.4.3 Recommendable EC Value for Drinking Water

Electrical conductivity is widely used for monitoring the mixing of fresh and saline water, for separating stream hydrographs and for geophysical mapping of contaminated groundwater (Hayashi 2004). Distilled water should typically have an EC of less than 0.3µS/cm. The EC of drinking water should be no more than 2500µS/cm (European Commission 1998); water with a higher TDS may have water quality problems and be unpleasant to drink. Electrical conductivity is proportional to the sum of cautions and anions, and roughly equivalent to total dissolved solids (TDS) in water (EPA 2005).

$$TDS(mgl^{-1}) = EC(\mu Scm^{-1})x0.67$$

Table 3.4.3a. Recommended or Mandatory Limit values of EC for Drinking water (EPA 2005)

EU Directive or National [Ministerial] Regulations		*Units of Analysis*	*G Value*	*I/PV Value*	*Note(s)*
Surface Water Regulations [1989]	A1 waters	µS/cm	n/a	1,000	[1]
	A2 waters	µS/cm	n/a	1,000	[1]
	A3 waters	µS/cm	n/a	1,000	[1]
Drinking Water Directive [98/83/EC]		µS/cm	n/a	2,500	[1,2]

3.4.4 Recommended EC Value for Irrigation water

The EC values as high as 2000µS/cm may be acceptable for irrigation water (EPA2005). Irrigation water quality parameters are commonly selected considering their impact on crop production, livestock health and human health. The effect on crop production is evaluated by considering Salinity (total dissolved solids or electrical conductivity), sodicity (residual sodium carbonate, sodium adsorption ratio, or adjusted sodium adsorption ratio) and toxicity due to specifications that affect sensitive crops. Salinity is important as it affects crop water availability and hence the growth. Sodicity affects soil structure and hence the rate of infiltration thus affecting crop growth. Since irrigation water is also commonly consumed by livestock and thus affects its health. There is possibility of uptake of contaminants from irrigation water by crops thus affecting human health (UNDP 2007).

Table.3.4.4a. Recommended Values of EC& TDS for Irrigation (UNDP 2007).

Water Quality Guideline			**Conditions of Use**
Water Quality Parameter		**Guideline Value**	
Salinity	TDS or EC	2000 mg/L or 3.0 dS/m	Coarse textured soils
SAR		10	
RSC		2.5 me/L	
Salinity	TDS or EC	1500 mg/L or 2.3 dS/m	Medium textured soils.
SAR		8	
RSC		2.3 me/L	
Salinity	TDS or EC	1000 mg/L or 1.5 dS/m	Fine textured soils
SAR		8	
RSC		1.25 me/L	

Irrigation water whether derived from springs, diverted from streams, or pumped from wells, contain appreciable quantities of chemical substances in solution that may reduce crop yield and deteriorate soil fertility. In addition to the dissolve salts, which has been the major problem for centuries, irrigation water always carry substances derived from its natural environment (FAO 2003).

3.5 Water Quality of Lake Besaka

3.5.1 The EC Value of the Lake

The electrical conductivity (EC) across the surface of the Lake water varies between 4.79 ± 0.04 mS/cm and 7.06 ± 0.01 mS/cm. The mean EC value is 6.54 ± 0.87mS/cm. In other words, it is approximately between the range of 4790µS/cm and 7060µS/cm with a mean value of 6520µS/cm (Abubekar 2007). However, the inflow of irrigation water to Lake Beseka, the average value of the Lake Water EC was lowered to 5030µS/cm (Haile Arefayne 2015). Although variation in the EC values of unpolluted natural waters depends on the underlying geology, it generally ranges between 30-400µS/cm (Susan and Joy, 1997). It has been noticed by different authorities that there is a natural pollution concern in the Lakes of the MER such as Lake Besaka (Tamiru, 2000; Berhanu, 1996; Zinabu and Pearce, 2003) driven by high-temperature-water to rock interaction from the underlying geology (Darling *et al.,* 1996 and MoWR, 1999).

Table. 3.5.1a. Electrical conductivity of Lake Besaka water (UNDP 2007)

Parameters	SS1	SS2	SS3	SS4	SS5	SS6	SS7
Field temp.(°C)	28.5	33.1	31.0	25.0	29.6	41.4	37.0
EC*(mS/cm)	6.96	7.06	6.91	6.66	6.88	1.71	4.79
EC **(mS/cm)	6.35	5.63	5.87	6.66	6.09	1.01	3.35

EC* is EC of surface water at field temperatures, and EC** is calculated EC at 25°C.

The spatial variation in EC across the surface of the Lake appears to be affected by the spatial variation in surface temperature (Abubekar2007).

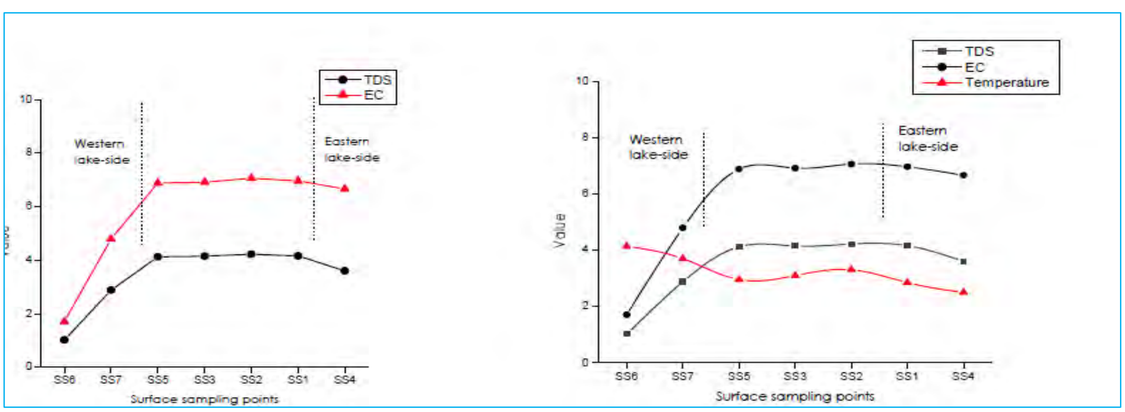


Fig..3.5.1a. Impact of surafce temprature on EC value of Lake Beseka(Abubekar 2007)

On the other hand, keeping other existing influential factors, the impact of variability in surface temperature on the spatial variability of EC values can be uncovered by using the relation (Abubekar 2007). Temperature is one of the most important factors; among others such as TDS, mobility and valence; which can affect electrical conductivity (APHA, 1998).

$$EC_{28.5} = EC_T\ (1 - 0.025\Delta T)$$

Where:

$EC_{28.5}$ = Electrical conductivity at 28.5°c
EC_T = Electrical conductivity at temperature T
ΔT = Change in temperature

3.5.2 TDS Values of the Lake Water

The total amount of dissolved chemical species (TDS) over the surface of the Lake water varies between 2.88 ± 0.06 g/l and 4.22 ± 0.00 g/l with a mean value of 3.86 ± 0.53 g/l (Abubekar 2007). This is nearly between 2,880 mg/l and 4,220 mg/l with a mean value of 3,855 mg/l. Since natural waters with TDS values between 1,500 mg/l and 5,000 mg/l are classified as brackish (Susan and Joy, 1997); Lake Besaka's water can also be classified as a brackish water type. In other words, the Lake water is between the classes of natural waters known as fresh-waters and saline-waters whose TDS values are less than 1500 mg/l, and greater than 5000 mg/l, respectively (Susan and Joy, 1997).

FAO (1998) classified water resources with TDS values greater than 2,000 mg/l under 'severe' degree of restriction for the use of irrigation based agriculture. Lake Besaka's mean TDS value (3,857 mg/l) is well above the FAO guideline limit and thus cannot be recommended for irrigation purpose. Besides, especially since the Lake water has been reported to be sodium-bicarbonate type (Bedilu, 2005; MoWR, 1999); the application of the water for irrigation purpose may cause sodicity problem to agricultural soils. On the other hand, the pumping out of the Lake water into the Awash River as it is being practiced by the Ministry of Water Resources to regulate the ever expanding volume of the Lake may also have possible negative implications on one of the most vital rivers of the country (Abubekar 2007).

3.6 Fresh Water Quality of Awash River

The fresh water chemical composition of Awash River is almost similar with that of stream water except few chemical compositions whose sources are from fertilizers (Elias and Brook 2016).

Table.3.6a. Chemistry of Awash River (Olmana & Josef 2009)

Type of water	PH	EC	Na^+	$Ca2^+$	HCO_3^{2-}	Cl^-	SAR
Awash River	7.73	0.38	1.58	1.52	2.70	0.50	*1.6*

3.7 Acceptable blending Ratios of Lake Besaka water to Awash River

3.7.1 Acceptable Ratios for Irrigation

The results have shown, the EC value of blended (Besaka to Awash) irrigation water of the study area ranges from 1.606 to 2.97dS/m. Maximum EC of Blended irrigation water obtained from treatment (T7, 50%) was 2.97dS/m and all blended irrigation treatments were increased with increasing mixed ratio as shown in Table 3.10.1a(Kebede et.al 2016).

Table3.7.1a. Samples mix ratios of Lake Besaka water to Awash for irrigation (Kebede et al. 2016)

Treatment (T)	Awash water (%)	Beseka Lake water (%)
Treatment 1	92	8
Treatment 2	90	10
Treatment 3	85	15
Treatment 4	80	20
Treatment 5	75	25
Treatment 6	70	30
Treatment 7	50	50
Treatment 8 (control)	100	0

Table.3.7.1b. Resulted EC & PH values for different mix ratio (Kebede et. al 2016)

Irrigation Treatment	Mixing ratio (%) (Awash:beseka)	pH	EC (dS/m)
T1	92:08	8.50	0.95
T2	90:10	8.70	0.96
T3	85:15	8.60	1.20
T4	80:20	8.70	1.19
T5	75:25	8.90	1.34
T6	70:30	8.60	1.30
T7	50:50	8.60	1.60
T8	100:00	8.70	0.96

It is easily presumable from the result shown in Table 3.10.1b that in terms of EC value, all the mixed irrigation water treatments were suitable for irrigation purpose. According to FAO (1985) standard, EC value of treatment (T8, 100%) falls within ‘permissible’ irrigation water quality classification standard. In terms of the ‘degree of restriction on use’, EC values of all treatments Table 3.10.1b are categorized under ‘slight to moderate’ according to UCCC (1974) (Kebede et.al 2016).

Table 3.7.1c. Crop yield parameter result for samples test indicated on Table 3.7.1b (Kebede et.al 2016)

Irrigation treatment	Mixed ratio (%)	Stand count	Average plant height (cm)	Boll number	Yield (Q/ha)
T1	92:08	26.6667[a]	69.333[a]	11.0000[ba]	39.143[b]
T2	90:10	24.3333[b]	69.333[a]	11.6667[a]	36.450[c]
T3	85:15	26.0000[ba]	69.333[a]	11.0000[ba]	34.497[dc]
T4	80:20	25.3333[ba]	63.333[a]	10.0000[bc]	33.403[d]
T5	75:25	25.0000[ba]	59.000[a]	8.6667[dc]	29.657[e]
T6	70:30	26.0000[ba]	74.333[a]	8.6667[dc]	28.233[e]
T7	50:50	26.3333[a]	72.333[a]	8.0000[d]	25.243[f]
T8	100:00	25.3333[ba]	71.000[a]	11.3333[ba]	41.393[a]
Mean	-	25.666	67.708	33.502	25.666
LSD(0.05)	-	1.827	17.759	1.351	2.218
CV (%)	-	4.066	14.977	7.683	3.781

3.8 Water quality standards for Drinking and Irrigation

3.8.1 Drinking Water

Water contains a variety of chemical, physical and biological substances that are either dissolved or suspended in it. The quality of drinking-water is a powerful environmental determinant of health. Therefore assurance of drinking-water safety is a foundation for the prevention and control of waterborne diseases and control the effect of different chemicals and physical parameters. The main aim of establishing water treatment plants is to make the water safe for drinking that meets international and/or national water quality standards.

Chemical contamination of water sources may be due to certain industries and agricultural practices, municipal solid waste, urban runoff or from natural sources. When toxic chemicals are present in drinking water, there is the potential that they may cause either acute or chronic health effects. After exposure of chemicals in drinking water for extended years rather than months they become of health concern (WHO, 2006). Chronic health effects are more common than acute effects because the levels of chemicals in drinking water are seldom high enough to cause acute health effects. There are many evidences that chemical contaminants created adverse human health problems in urban watersheds (EPA, 2005)

4 ESTIMATION HYDROELECTRIC POWER & SOLAR ENERGY

4.1 Pumping Water from Lake Besaka

The quantity of water to be pumped from Lake Besaka and drained to Awash River was estimated on the basis of annual inflow to the Lake and tolerable blending ratios to be released respectively. The studies that conducted by JICA in 2015 & OWWDSE 2014 indicated the average net annual inflow to the Lake is 92MCM / (2.92m^3/se).

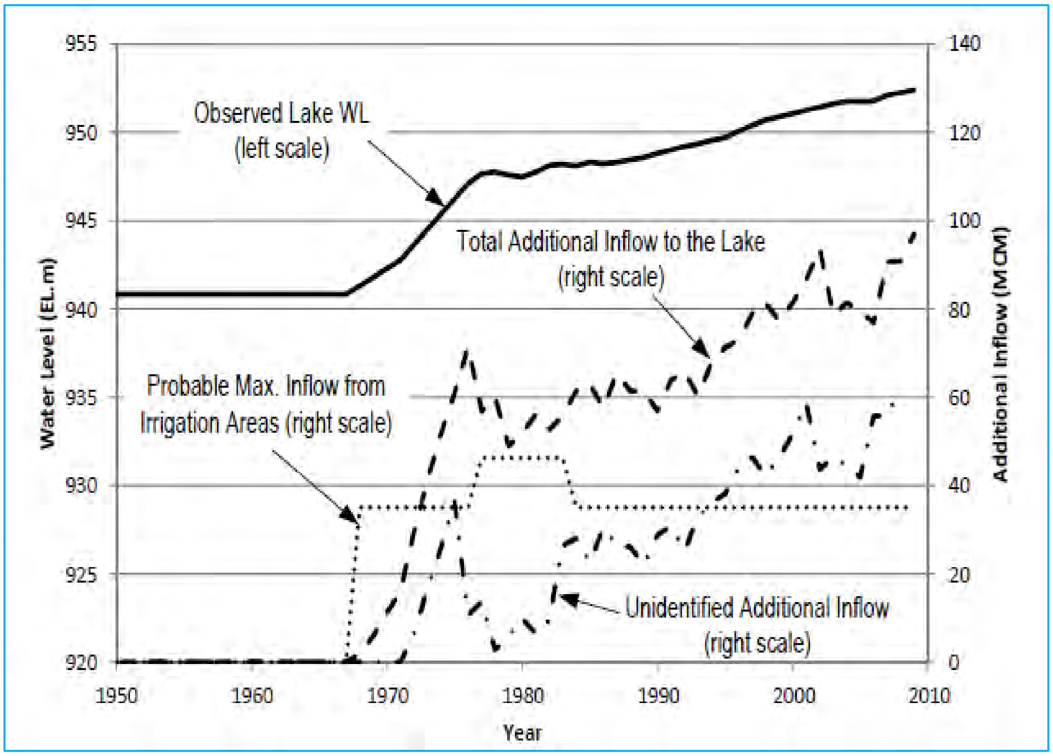


Fig.4.1a. Annual inflow to Lake Besaka and Lake water Level

The volume of water aimed to be pumped from Lake Besaka was primarily estimated based on:

- The volume of upper reservoir
- The rate net inflow to the Lake
- The optimum blending ratios of the two water bodies in all seasons.
- Pumping, operation and maintenance cost of the plant

4.1.1 Mixing water from Lake Besaka with Awash River

The estimation of safe blending ratio of water from Lake Besaka to Awash River is depending on the annual water budget of the Awash River and the recommended chemical composition in the mixes. The buffering ratios have estimated in the manner that the mixes are tolerable for drinking and irrigation at downstream.

Hence, in order to estimate the tolerable buffering ratios in all months of the year and release water from Lake Besaka to Awash River, the annual flow pattern of the river was considered.

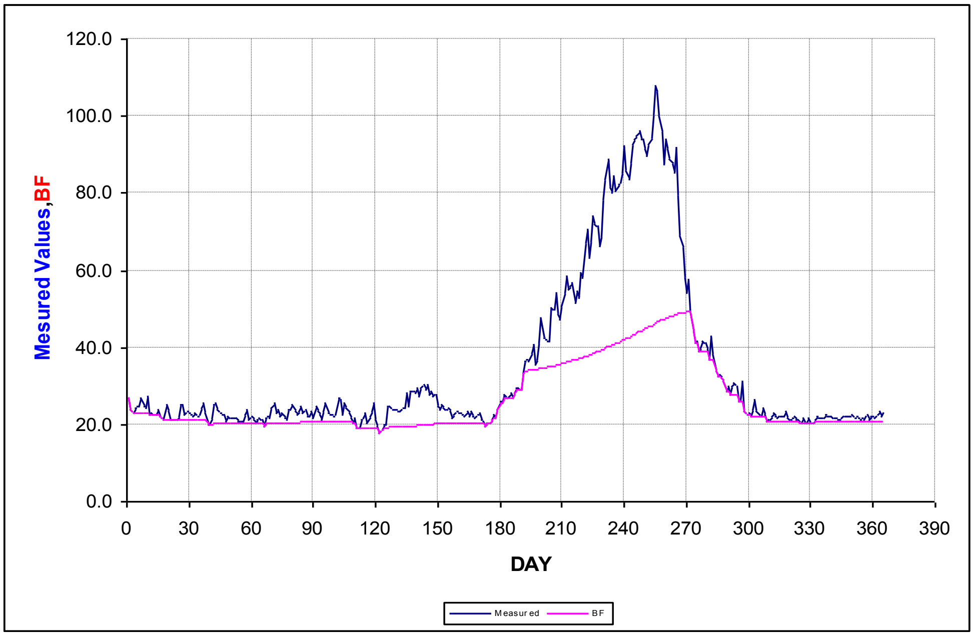


Fig.4.1.1a Annual flow in Awash River at Matahara. JICA 2015

From the above hydrograph, the monthly average flow of Awash River was calculated and tabulated as follows.

Table. 4.1.1a. Average monthly flow of Awash River as estimated from fig.4.1.1a.

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Awash River Flow(m^3/se)	23	22	24	20	26	27	52	86	60	24	21	22

In the previous chapter, it was indicated that the tolerable buffering ratio of water from Lake Besaka to Awash for irrigation and water supply is **(8%): (92%).** As it was clearly indicated in finding of the researches mentioned in Chapter 3, this mix ratio is tolerable for irrigation and drinking. Taking these mixes into account, the maximum rate flow that can be released from Lake Besaka to Awash River for each month was estimated for all flow condition of the river throughout the year.

For instance, the flow of Awash River in the month May=26m^3/sec. The tolerable flow rate that we can release from Lake Besaka to Awash River in the month of May calculated as:

$$\frac{\text{flow from Lake Besaka}}{\text{flow from Awash River}} = \frac{8\%}{92\%}$$

$$\text{Maximum flow from Lake Besaka in May} = \text{flow of Awash in May}\left(\frac{8\%}{92\%}\right) = 26\text{x}\frac{8}{92} = \mathbf{2.26}\,\frac{\mathbf{m^3}}{\mathbf{sec}}$$

Table.4.1.1b. Annual flow variation of Awash River and maximum flow that can be released from Lake Besaka

Month	Jan	Feb	Mar	**Apr**	May	Jun	Jul	**Aug**	Sep	Oct	Nov	Dec
Awash River flow (m^3/se)	23	22	24	**20**	26	27	52	**86**	60	24	21	22
Release from Besaka (m^3/sec)	2	1.91	2.09	**1.74**	2.26	2.35	4.52	**7.48**	5.22	2.09	1.83	1.92

From the table, the maximum and minimum flow possible to be released from Lake Besaka to Awash River in dry and wet seasons are **7.48 m^3/sec** and **1.74m^3/sec** respectively. Hence, the peak discharge for the design of hydraulic and appurtenance structure is 7.48m^3/sec.

The allowable turbine working hours per day for Francis type hydropower turbine is 20hrs (US Energy and power market 2010). However, 18hrs working time was considered in this study for a safety matter. Hence, 70.06MCM volume of water can released to Awash River annually.

For instance, the volume water that can be released from Lake Besaka to Awash River for the month of May is:

$$V_{may}= (2.26m^3/sec)x31x18x60x60MCM=4.54MCM$$

Table.4.1.1c. Volume of water that can be released
For the estimated monthly flow of Awash River

s/n	Month	No of days in the month	Tolerable flow in (m^3/se)	Volume water that can be released
1	Jan	31	2	2*31*18*60*60=4.02MCM
2	Feb	28	1.91	3.47MCM
3	Mar	31	2.09	4.20MCM
4	Apr	30	1.74	3.38MCM
5	May	31	2.26	4.54MCM
6	Jun	30	2.35	4.57MCM
7	Jul	31	4.52	9.08MCM
8	Aug	31	7.48	15.03MCM
9	Sep	30	5.22	10.15MCM
10	Oct	31	2.09	4.20MCM
11	Nov	30	1.83	3.56MCM
12	Dec	31	1.92	3.86MCM
Maximum annual volume of Water that can be released from Besaka to Awash				**70.06MCM**

4.1.2 Pumping Energy

Solar energy was preferred for pumping energy due to its environmentally friend and low running cost. The energy was designed to pump water from the Lake and lift up water to the upper reservoir capacity of 50MC. The capacity of solar energy for pumping was estimated based on the monthly sunshine hour's data obtained from Ethiopia Meteorology Agency. Hence, the minimum average monthly sunshine hour of the area is 8.55hrs. But, 8.30hrs is considered in design for safety matter.

4.1.3 Design Discharge

To meet the aim of the study, and fully Control the Expansion of Lake Besaka, the volume of water which is greater or equal to the annual inflow to the Lake should be pumped. Keeping other factors constant, the pumping of 92MCM water annually can ensure zero expansion of Lake Besaka.

However, the tolerable mix ratio of Besaka to Awash only allow the release of 70.06MCM/year water from the Lake to Awash River. Whereas, the net annual inflow to Lake Besaka is 92MCM and the change in volume [$\Delta V = (92-70.06) = 21.9$MCM] which is equivalent to the evaporation loss from upper reservoir of 6.8km^2 surface area as per the estimation of JICA 2015.

Therefore, the design discharge for pumping is estimated on the basis of maximum volume of inflow to the Lake annually i.e. 92MCM and the minimum average sunshine duration at Lake Besaka catchment. The minimum average sunshine duration in the catchment is 8.5hrs. However, 8.30hrs considered for safety conditions in designing solar energy plant capacity.

Hence, the maximum discharge that can be pumped for the maximum inflow of the Lake is calculated as:

$$Q_d = 92\times10^6 m^3 / (8.3\times60\times60\times365)\ sec$$

$$Q_d = 8.44 m^3/sec$$

4.1.4 Design Pressure Line/Penstock

The water conveying pipe from the Lake to the upper reservoir is primarily serving as pressure Line. However, it serves as penstock when water may release back to the Lake and the plant works as pumped storage hydropower. The design of the waterway for a pumped storage hydropower project is an important part of the overall project design as it has the potential to greatly impact the overall efficiency of the facility and the performance of the pump/turbine units. Based on the hydraulics of the system, surge tank is required to protect the waterway and pump/turbine units from water hammer if the ratio of gross head to length of the penstock is greater than 5. However, the effect of water hammer is considered to compute the thickness of the penstock.

The estimated design discharge to be pumped from the Lake is **8.44m^3/sec**. The ground elevation difference between pump-turbine inlet and upper reservoir as the data collected from topographic map and primary elevation data by GPS is H=350m and the length is L=4200m.

The hydraulic design of pressure line/penstock for pumping should consider the following points (Karlos Martins, 2013).

- The design discharge of the pressure line/penstock is the design discharge for pumping.
- Length of the pressure line/penstock, L_p
- Design head for the pressure line/ penstock, H_{max}, which is equal to the gross head of the system. The friction loss is generally less than (9-12)% of the gross head.
- Ultimate tensile strength of the penstock material.
- Economic diameter of the penstock (in mm).
- Flow velocity in the penstock for the chosen diameter. This value should be in the following ranges: (2-3) m/s for low head, (3-4)m/s for medium head and 4-5 m/s for high head schemes. (head (H): low H < 50 m; medium 50 ≤ H ≤ 250, high H > 250).
- The minimum thickness, t_{min}, is automatically calculated according to the material of the penstock.
- The safety factor, *SF*. If the value is less than 4 then increase the effective thickness, t_{eff}, until it the *SF* is at least 4.

The economic pressure line/penstock size selection is depending on installation, material as well as whether used above ground or buried.

Warnick et.al (1984), developed empirical formula for the estimation optimum penstock diameter (D_e) for small hydro projects in terms of rated discharge:

$$\mathbf{D_e = 0.72Q^{0.5}}$$

Where;

D_e = Diameter of the penstock (m)

Q= Flow in the penstock (m^3/s)

$$D_e = 0.72(8.44)^{0.5}$$
$$= 2.1\text{m}$$

The friction loss in the system is calculated for the indicated data as:

$$V_l = 1.274Q/D_e^2$$
$$= 2.15\text{m/sec}$$
$$R_e = V*D_e/(\vartheta)$$

Roughness of steel

$\varepsilon = 0.045\text{mm}$

The average surface temperature of Lake Besaka is 28.3°c and its sanity content is 10.7dS/m. Besides, the kinetic viscosity (ϑ) the Lake water is $1.4593\times10^{-6} m^2/sec$ (H.Ezzat Khalifa 1981).

The friction loss in the penstock (h_f) is calculated for different values of penstock diameter iteratively using moody diagram till (h_f/H_g) 100% = (9-12) %

Reynolds number

$Re=V*D_e/\vartheta$

$=2.15*2.1/1.4593x10^{-6}$

$=3.1x10^6$

Relative roughness of steel

$=\varepsilon/De$

=0.045mm/2100mm

=0.000021

From Moody diagram, friction coefficient (f) =0.0142

$$h_f = f\frac{L_p}{D_e}\frac{V^2}{2g}$$

Where:

h_f = Head loss (m)

f = Friction factor

L = Length of penstock (m)

D_e = Diameter of penstock (m)

V = Water velocity in penstock (m/s)

g = Gravity (m/s^2)

h_f = 6.69m

Where:

H_g = gross head

= (ground elevation + friction loss + minor friction loss

Minor friction loss= 0.8% net head (H_n) (Karlos Martins, 2013).

H_d= (350-6.69)m

= 343.31m

Minor loss= 0.008*343.31m

=2.75m

Gross head= (6.69+2.75+343.31)m

=352.75m

(h_f/H_g) 100% = (6.69/352.75)100%=1.9%.

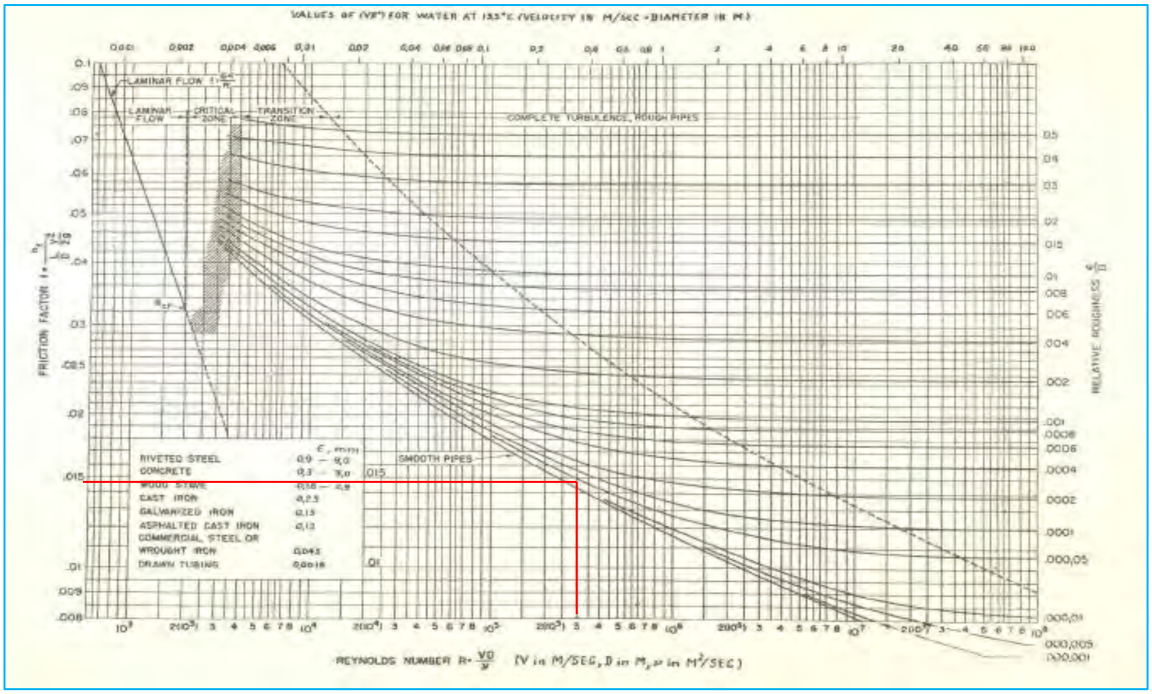


Fig. 4.1.4a. Moody diagram

Hence, further iteration is needed till the percentage of (h_f/H_g) fall between (9-12) %:

Table.4.1.4a. Penstock friction loss and diameter iteration

Iteration	Diameter (m)	friction loss (h_f), m	flow velocity (m/s)	%h_f
1	2.1	6.69	2.15	1.90
2	2.0	10.99	2.69	3.02
3	1.9	14.21	2.98	3.87
4	1.8	18.61	3.32	5.01
5	1.7	24.74	3.72	6.56
6	1.6	33.51	4.2	8.68
7	1.5	44.76	4.7	11.27

The last iteration results satisfy the %h_f and flow velocity requirements as it is high head hydropower scheme. The constraints in deciding the thickness of the penstock are:

- The costs of the penstock and
- Strength to withstand pressure

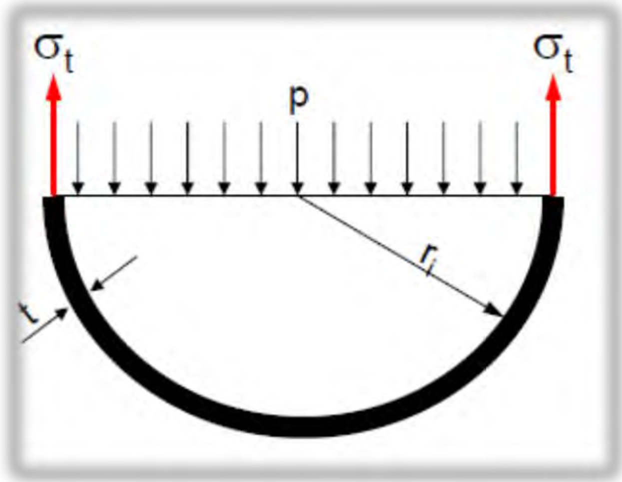


Fig. 4.1.4b. Internal fluid pressure on penstock

Therefore, the follow results are used for penstock thickness estimation

- D_p=1.5m
- The material used is Mild steel (rolled welded steel pipe)
- Penstock length (Lp) = 4200 m
- Flow of water (Qp) = 8.44m^3/sec
- High gross (H gross) = 397.2 m
- Head Loss (H loss)= 44.76 m
- Minor friction loss= 2.44m
- Net Head = 302.80m
- Water velocity in the penstock (V) = 4.7 m / s
- Efficiency of penstock (pipe eff) = H_n/ H_g * 100% = 76.23%
- K_w (bulk modulus of water) = 2.1*10^9 N/m^2
- σ_t (Tensile strength of steel penstock) = 400 * 10^6 N/m^2
- E (modulus of elasticity) = 206 * 10^9 N/m^2
- Kinetic viscosity of Lake water=1.4593x$10^{-6}$$m^2$/sec
- Working pressure (P work) of the penstock (Mild steel)=1.5 * Gross Head (Hg)

Working pressure (P) = 1.5 * 397.2 = 595.80 m
= 1.5 * (397.2 / 595.80) = 1.0 kgf/cm^2 (Bars)
= 100,000 N/m^2 (Pascal)
= 0.0001 kN/mm^2

The formula for a thin tube (if D_p / t > 20

Where:

t = thickness of penstock in mm

e_s = extra thickness for corrosion (1-3) mm

Working pressure (P) = 0.0001 kN/mm^2
Dp = diameter of 1500 mm penstock
σ_t (Tensile strength) = 400* 10^6 N/m^2
= (400 * 10^6 N/m^2) * (kN/1000N) * (m^2/1000.000 mm^2)
= 0.400 kN/mm^2

- **Minimum penstock thickness (t_{min})**

$$t_{min}(mm) = (P * D_p) / (2\ \sigma_t) + e_s.$$

Taken extra thickness for corrosion (es) = 3 mm
= (0.0001kN/mm^2x 1500mm) / (2 x 0.400kN/mm^2) + 3mm
= 3.19 mm
The impact of pipe handling in transportation, laying, deformation, etc., it is necessary to add more rapidly the penstock thickness (in the wills of 3 mm).
So thick of penstock (t) is = (3+3.19) = **6.19mm**.

Effect of Water Hammer
In the design of penstock also must take into account the effects of water and control the speed lacing.

If the ratio of gross head to penstock length **(H_g / L_p)> 5**, the surge tank is required (C.C.Warnick 1984)

However, H_g / L_p =397.2/4200
= 0.095 < 5
From the above relationship, surge tank is not required but the effects of water hammer still be considered for safety of pump-turbine units and penstock.

The thickness of the penstock (t) = 6.19 mm is to be used as initial estimation value

At wills:
% Closure of the valve flow (Z) = 50%
With the closing time (Tclose) = 4 seconds (fast enough)
Corrosion allowed thickness (e_s) = 3 mm
Overall safety factor (SF) = 4

The speed of water waves in the penstock:

$$C_{wave} = [(10^{(-3)} *K_w) / (1 + (K_w* D_p / E* t)]^{(0.5)}$$

Where:
K_w = bulk modulus of Lake water 2.1x10 ^{9} N/m^2
E_p = modulus of elasticity of penstock material 206 * 10^9 (N/m^2)
D_p = Penstock diameter 1500 mm
t = initial estimated penstock wall thickness is 6.19 mm
L_p = length of penstock, 4200 m

By entering values:

$$C_{wave} = [(10^{-3} * 2.1*10^9) / (1 + (2.1*10^9 \times 1500 / (206 * 10^9 * 6))]^{0.5}$$
$$= 769.12 \text{ m / s}$$

Critical closing time of the penstock (T_c)

The time it takes the pressure wave to return again to the valve after the sudden closure, known as the critical time(T_{cri});

$$T_{cri} = 2 L / C_{wave}$$
$$= (2 \times 4200) \text{ m} / 769.12 \text{ m / s.}$$
$$= 10.92 \text{ seconds}$$

T_{Close} (4 sec) < T_{cri} (10.92 sec).

$$H_{surge} = C_{wave} * V * z / (980)$$
$$= 769.12 * 4.7 * 50 / (980)$$
$$= 184.43\text{m}$$

$$H_{total} = (H_{surge} + H_g)$$
$$= (184.43+397.2)\text{m}$$
$$= 581.63\text{m}$$

= 581.63 m (greater than the pressure of work: 581.63m > P_{work} 579.06 m)

$(H_{surge} + H_g) > P_{work}$

Hence, the required thickness of the penstock (t_p) is calculated as:

$$t_p = (H_{total} * D_p * SF / 83700) + e_s$$
$$= (581.63 \text{ m} * 1500 \text{ mm} * 4) / 83700 + (3 \text{ mm})$$

= 44.69 mm

$t_p > t_{min}$

Hence, the initially estimated penstock thickness (t=6.19mm) is not adequate to withstand the internal pressure.

4.1.5 Design of Pump-Turbine

The approach for planning, designing and selecting pump-turbine is similar to that of conventional hydropower system turbines. However, the approach involves different experience curves and must accounts for the fact that it is not possible for the reversible pump/turbine to operate at peak efficiency as a turbine and pump (Warnick 1984).

The selection of pump-turbine sizes involves determining a suitable specific speed (N_{st}) for the turbine and pump (N_{sp}). From this characteristic, turbine or pump constant selection is made. The design speed of pump-turbine must be a synchronous a speed. Once the design speed is determined, an actual n_{st} and n_{sp} can be determined based on the desired rate power output, P_d, and rated discharge, Q_d, and rated head H_n.

The specific speed for turbine action of pump-turbine is given by the equation;

$$N_{st} = \frac{N\sqrt{P_d}}{H_n^{\frac{5}{4}}}$$

Where;

N_{st} = specific speed for turbine (rpm)
N= rotational speed pump, rpm
H_d = pump-turbine design head(m)
P_d = Turbine full gate capacity(kW)

The solar power required to lift up water from the Lake to upper reservior is 43.58MW as calculated the topic of pumping energy estimation

Hence, the synchoronus speed of the pump is calculated as:

$$N_{st} = \frac{N\sqrt{P_d}}{H_n^{\frac{5}{4}}}; \qquad 109 = \frac{N\sqrt{43{,}580}}{(397.2)^{\frac{5}{4}}}; \qquad \mathbf{N = 927rpm}$$

Hence, the primary objective of pump-turbine for this specific project is mainly to pump water from Lake Beseka to the upper reservoir that located at a total head of 397.2m and can used as turbine when the flooding effect of the Lake reduced to the desired level.

For the estimated value of synchronous speed of pump-turbine, the number of pole is calculated as:

$$N = \frac{120f}{N_p}$$

Where:

f= frequency (f=50Hz)
N=synchronous speed of pump- turbine
N_p = Number of poles

$$N = \frac{120f}{N_p}; \qquad 927 = \frac{120x50}{N_p}; \quad \mathbf{N_p = 6}$$

The generator with six poles is required needed to be installed for the operation of pump-turbine.

The equation for determining the specific speed for pump/turbine both turbine operation and for the pumping operation is;

$$N_{st}=1700xH_n^{-0.481}$$

$$H_n = (H_g\text{-}h_f\text{-}h_{min})$$
$$= (350\text{-}44.76\text{-}2.44) = 302.80m$$

Hence,

N_{st}=1700x $(302.80)^{-0.481}$

=109rpm

The specific speed (N_{sp}) of pump-turbine can be estimated from the following curve for the calculated value of S_{st}.

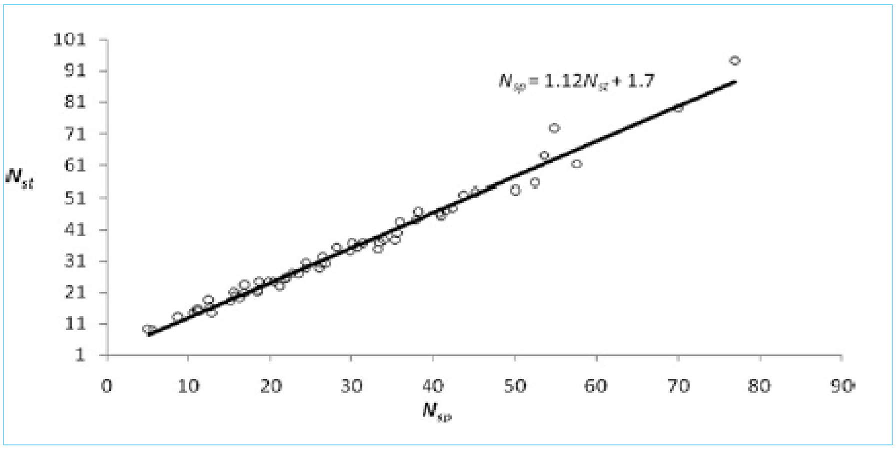


Fig. Turbine verse pump specific speed for pump-turbine (Yang et al 2012)

From the linearly represented graph, the turbine specific speed of the pump is;

$$\mathbf{N_{sp} = 1.12\ N_{st}+1.7}$$

N_{sp} = 1.12x109 +1.7

=124rpm

The curves are utilized to develop various preliminary dimensions for with the known value of N_{sp}, the peripheral velocity (ϕ_1) can be read from the curve below:

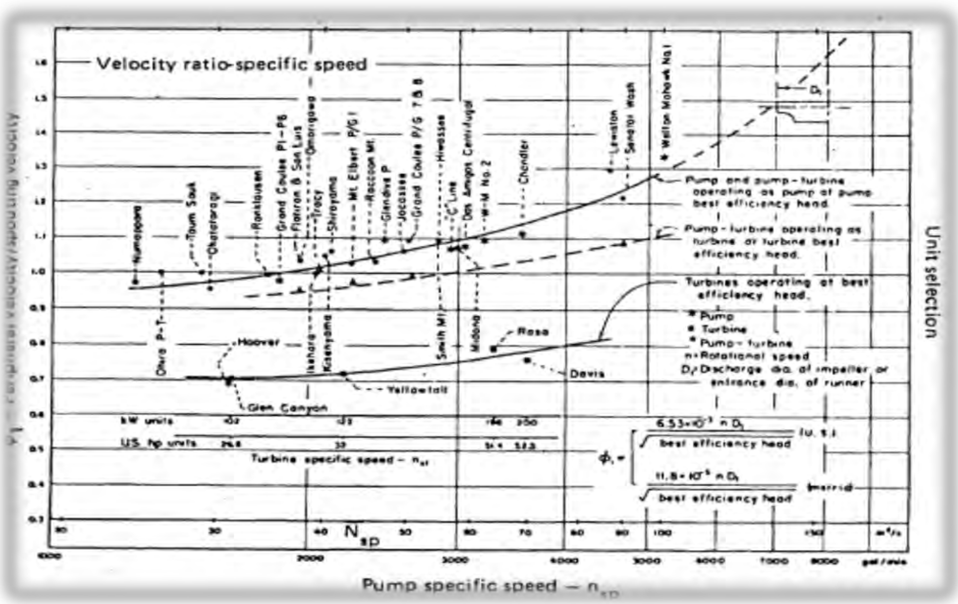


Fig.4.1.5a. experience curve that relates specific speed of the pump, N_{sp} to speed ratio. (Stelzer & Walters 1977)

For the value of $\mathbf{N_{sp}}$ =124rpm, the peripheral velocity can be read from the curve and $\phi_1 = 1.35$

$$\phi_1 = \frac{(11.8)(10^{-3})\text{xNxD}_1}{\sqrt{\text{Best efficiency head}}}$$

Where:

ϕ_1= The peripheral velocity

D_1= Impeller diameter

N= Synchronous speed of the pump-turbine

$$\phi_1 = \frac{(11.8)(10^{-3})\text{xNxD}_1}{\sqrt{\text{Best efficiency head}}}; \qquad 1.35 = \frac{(11.8)(10^{-3})\text{x927xD}_1}{\sqrt{397.20}}; \quad \mathbf{D1 = 2.46m}$$

The impeller throat diameter, $\mathbf{D_2}$ for N_{sp} value, **124rpm** can be read from the following curve. D_1/D_2=**1.15** and calculating for $\mathbf{D_2}$**=2.14m**

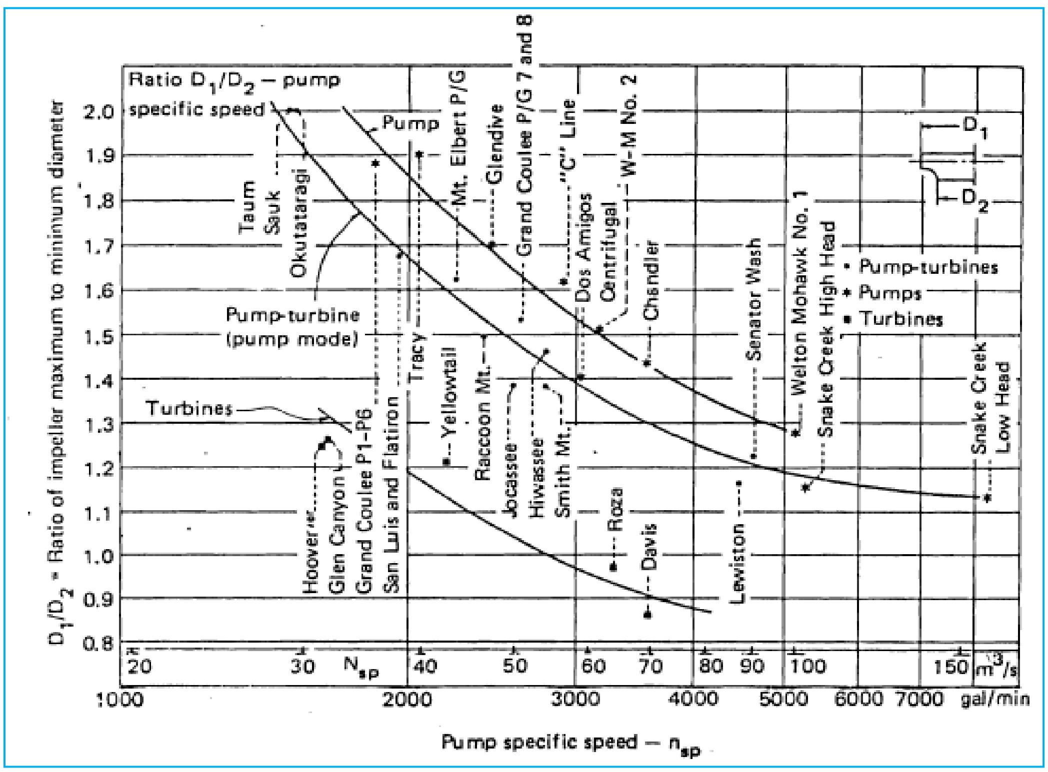


Fig.4.1.5b. curve for Impeller diameter ratio vs. specific speed (C.C.Warnick, 1984)

The wicket gate height, M, can be read from the wicket height to diameter ratio curve shown below.

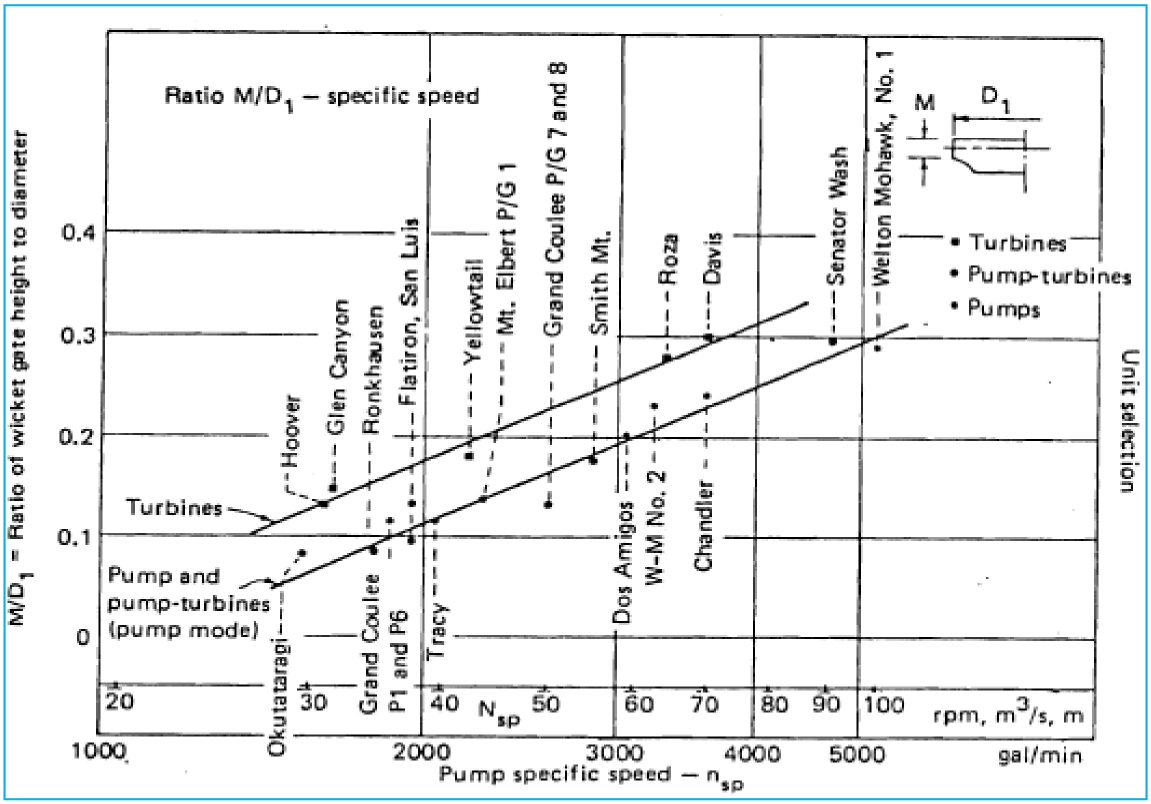


Fig.4.1.5c. Curve for wicket gate ratio to diameter and specific speed (C.C.Warnick 1984)

For specific pump speed, N_{sp}=124, the ratio of M/D_1 from the curve is **0.31**

Hence, the Wicket gate height is **(M) =0.76m**

The radial velocity of water entering the impeller can be calculated by the equation;

$$C_r = \frac{V_r}{\sqrt{2gH_p}} = \frac{0.0718Q_d}{MD_1\sqrt{H_p}}$$

Where;

C_r= ratio of radial velocity of water entering pump to the spouting velocity

The radial velocity V_r is;

$$V_r = \frac{0.0718Q_d\sqrt{2g}}{MD_1} = \mathbf{4.5m/sec}$$

Radial velocity can be calculated as;

$$C_r = \frac{V_r}{\sqrt{2gH_p}}, \qquad C_r = \frac{4.5}{\sqrt{2x9.81x397.2}} = 0.051$$

The Important point in preliminary planning for pump/turbine installations is the problem of determining the setting elevation for pump/turbines. Experiences has dedicated that the units be set at considerable submergence below the minimum elevation of the lower reservoir water elevation, the tail water for the turbine operation of pump/turbine.

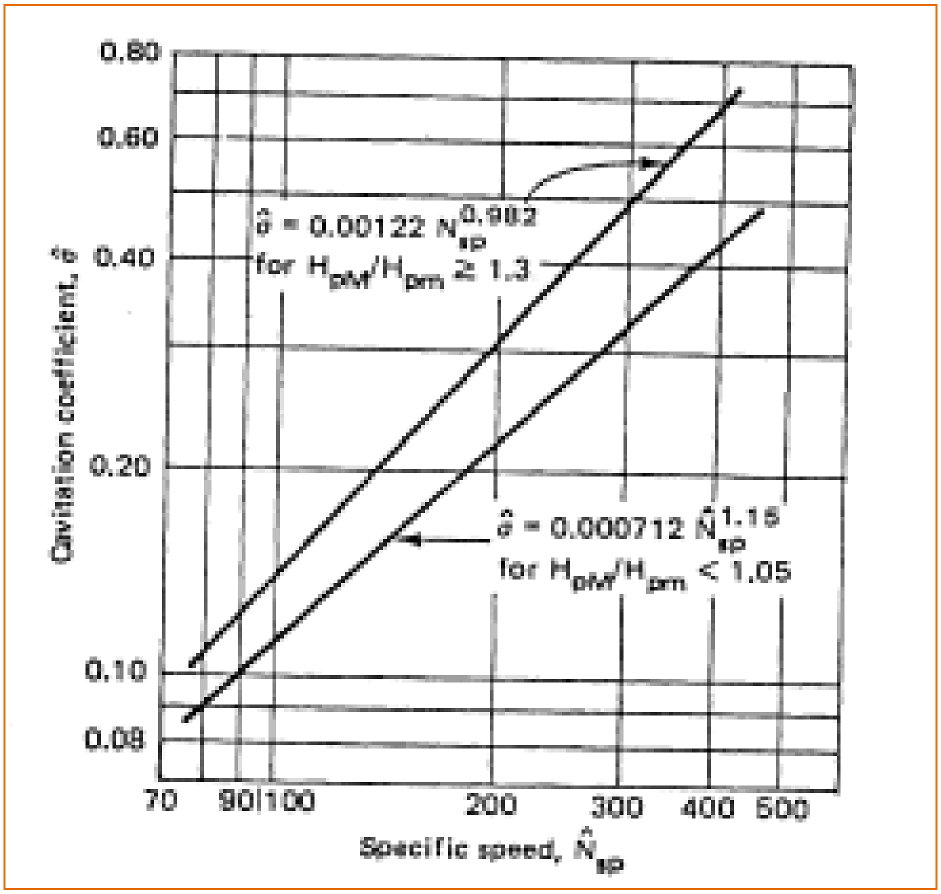


Fig. 4.1.2.5e. Cavitations coefficient vs. specific speed (Warnick 1984)

The ratio of gross pumping head (H_p=397.2m) to net head (H_n=302.80) ≥1.3 and taking the specific pump speed N_{sp}=124, the value of $\boldsymbol{\sigma}$, can be read from the upper curve;

$$\boldsymbol{\sigma} = 0.17$$

The barometric pressure at 1010m above sea level, is, H_b= 10.1m,

$$\frac{\mathrm{H_b} - \mathrm{H_s}}{\mathrm{Hp}} = \boldsymbol{\sigma}$$

$$\frac{10.1 - \mathrm{H_s}}{397.2\mathrm{m}} = 0.17, \qquad \mathrm{H_s} = -\mathbf{57.43m}$$

The most critical to which pump turbine will be exposed and represents the value that will dedicate maximum submergence requirements for the pump/turbine setting.

Hence the outlet to the pump turbine should be at;

1010m-57.43m= 952.6m

The calculated pump/turbine outlet position is appropriately designed when compared with the current water level in the lake (952.6m) and the level to which the maximum pumping reduce water from Lake Beseka (952.6m) to control its expansion and the Lake water level can be safely reduced to 954 above sea level.

4.1.5.1 Pump-Turbine type Selection

The selection of pump-turbine is similar for turbine selection for conventional hydropower. This research identified as pumping head ($H_{p=}$397.2m) and the design discharge of (Q_p=8.44m^3/sec) which falls in the categories of high head and low flow rate. Single- stage reversible Francis type Pump-Turbines with adjustable wick gates appear to offer the most economically attractive option for heads between 400 to 1000m.

4.1.5.2 Powerhouse for Pump-Turbine System

The powerhouse of hydropower plants usually consists of the superstructure and the substructure. The superstructure provides protective housing for generator and control equipment and crane. The substructure or foundation of the powerhouse consists of the steel and concrete components necessary to form draft tube, support the turbine stay ring and generator, and encase the spiral case. A control room is also included in the powerhouse to isolate the control systems from generator noise and to provide a clean and comfortable environment for operators. The powerhouse for this particular project was designed in the manner that the draft tube of pump-turbine should fully suck water from the Lake so that water level to be reduced to 952m above mean sea level.

4.1.5.3 Dimension of Pump/Powerhouse

Length: The length of the powerhouse depends on the number of units. However, the number of units for this particular project is one.

The design of pump-turbine for single stage Francis type turbine of high speed;

$$L_{ph} = 4D1 + 2.5m$$

Diameter of impeller is (D1) =2.462m

$$L_{ph} = 4 \times 2.46 + 2.5m$$
$$= 12.34m$$

Width: The width of powerhouse equal to the center to center spacing of the units plus the clearance space from the wall.

4.2 Estimation of Pumping Energy

4.2.1 Hourly Solar Radiation

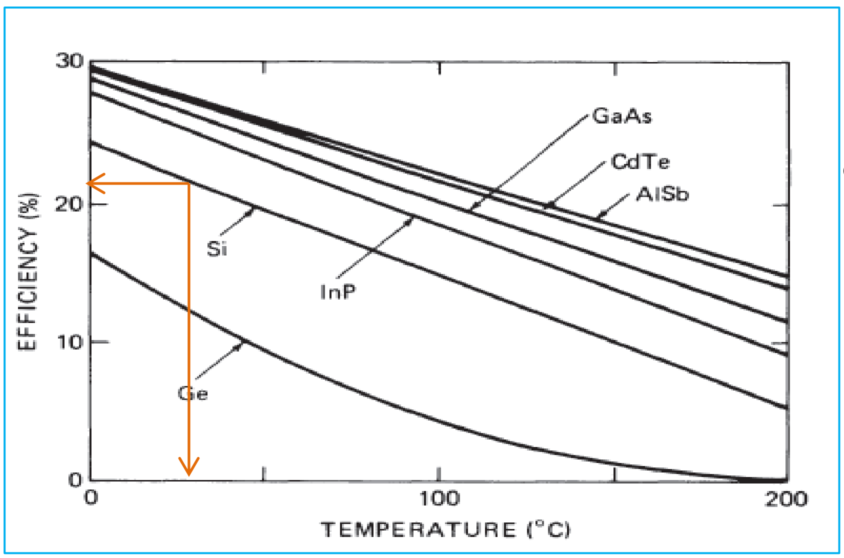
The essential parameter to estimate the capacity of solar energy output is the input solar radiation in watt/m^2. However, this parameter is not yet recorded at any of Meteorology stations found in the country as information from Ethiopia, National Meteorology Agency. For the estimation of solar plant capacity for pumping, solar radiation intensity was directly adapted from The Journal of Hydrology 245 (2001) that conducted on Lake Ziway.

The data was directly taken due to the similarities in sunshine hour's duration per day, temperature and geographical conditions.

Input solar radiation is how much amount of solar radiation is coming from sun and output solar radiation means how much of solar radiation we can utilize to generate electricity. The solar energy conversion efficiencies of solar panels depend on solar cells internal construction, dimension, active area of the panel, specific material properties, photovoltaic junction properties, anti-reflective coatings, surface texture, and cell operating temperature. The efficiency of solar cell is $(\eta) = \frac{P_{out}}{P_{in}}$.

A cell operates at its maximum efficiency η_{max}, when its maximum power output capability is utilized by an optimized load at a particular illumination intensity and cell operating temperature. The cell's operating efficiency η_{op} is the efficiency at which the cell is actually being utilized (H.S. Rauschenbach 1980).

The calculated efficiency of several solar cell types as a function of temperature is represented graphically as shown below.



Where:
Si= Silicon solar cell
GaAs=Gallium Arsenide cell
CdTe=Cadmium Telluride cell

Fig.4.2.1a Efficiency curve of different solar cells verse at different Temperature (H.S. Rauschenbach 1980)

Solar cell which is made of silicon was chosen for this study due its wider ranges availability of the market. The average temperature of Lake Besaka catchment is 28.3°c. Hence, keeping other factors constant, the efficiency of the solar cell at this temperature as read from the efficiency curve is 22%.

4.2.2 Plant Capacity and Collector Size

4.2.2.1 Estimation of Solar Energy for pumping

The solar energy plant capacity can be estimated from the basis of the net convertible solar radiation by the solar cell or the total sun radiation intensity multiplied by the efficiency of the solar cell to covert solar energy to electricity. The solar plant capacity to pump maximum flow of (Q=8.44m^3/sec) water from Lake Besaka to the upper reservoir of head (H=397.20m) is equal to the power.

$$P = \frac{\gamma QH}{\eta} = \frac{9.81x1100x8.44x397.2}{0.83} = \mathbf{43.58MW}$$

4.2.2.2 Estimation of Collector Size

The required solar collector area depends on the solar insolation level of a particular region and the efficiency of solar cell. A region with poor insolation level will need a larger collector area than one with high insolation levels.

Table. 4.2.2.2a. Solar Input and Output radiation for solar Energy production

Months of the year	**Input and output Solar Radiation Converted by solar module efficiency of 22%**	
	Total input Solar Radiation W/m^2	**Total output Solar Radiation W/m^2**
Jan	476.8	104.90
Feb	485.2	106.74
Mar	483.1	106.30
Apr	488.9	107.60
May	502.8	110.62
Jun	472.2	103.88
Jul	404.10	88.90
Aug	425.2	93.54
Sep	437.4	96.30
Oct	505	111.10
Nov	509.7	**112.13**
Dec	483.9	106.46

The minimum solar radiation output of the year is in the month of July. I.e. 88.90w/m^2 and the area of the solar collector for the can be estimated as:

Power required for pumping =Solar Plant Capacity (MW)
= Area of collector (m^2) * minimum solar radiation output (W/m^2)
Area of Collector needed = (Power Required for pumping)/(minimum radiation output)
= 43.58x10^{6}w/ (88.9w/m^2)
= 490,213m^2

The collector size can be reduced if high efficiency solar cells adopted. Besides, further analysis can be made to use the solar power when there is no pumping.

4.3 Releasing Pumped water to Awash River

The volume of water from Lake Besaka to be released to Awash River was determined based on the acceptable mixing ratios safe for drinking water and irrigation. The boundary conditions are the average monthly flow of Awash River and the tolerable mix ratios of Lake Besaka to Awash River. Different mix ratios were compared and reviewed so as to maintain the water quality standards of Awash River for drinking and irrigation for downstream users. The electric conductivity values were used for comparison to distinguish the level of tolerable and high toxic level.

The EC values less than 2000μS/cm is recommendable for irrigation and drinking water (EPA 2005, FAO 1987, WHO 2003 and UNDP 2007). Besides, Kebede et.al 2016 analyzed and evaluated the change in EC values and its impacts on the irrigation yields by mixing eight samples from Lake Besaka water and Awash River as shown on Table 3.10.1a. The analysis indicated mixing, 92% water from Awash River and 8% from Lake Besaka has resulted mixed EC value of 960μS/cm. Moreover, the experiment indicated 4% yield reduction due to the mixing the specified proportions. Hence, the EC value of (960μS/cm) was recommended for irrigation and drinking water (EPA 2005, FAO 1987, WHO 2003 and UNDP 2007).

Therefore, maximum tolerable mix ratios and monthly river flow of Awash River were considered to estimate the maximum and minimum water to be released from Lake Besaka to Awash River in dry and wet seasons. As it was mentioned in pervious chapter, the average flow of Awash River in the month of May is 26m^3/sec. The tolerable flow rate that we can release from Lake Besaka to Awash River in the month of May calculated as:

$$\frac{\text{flow from Lake Besaka}}{\text{flow from Awash River}} = \frac{8\%}{92\%}$$

$$\text{Maximum flow from Lake Besaka in May} = \text{flow of Awash in May}\left(\frac{8\%}{92\%}\right) = 26\text{x}\frac{8}{92} = \mathbf{2.26}\,\frac{\mathbf{m^3}}{\mathbf{sec}}$$

Table.4.3a. Annual flow variation of Awash River and maximum flow that can be released from Lake Besaka

Month	Jan	Feb	Mar	**Apr**	May	Jun	Jul	**Aug**	Sep	Oct	Nov	Dec
Awash River flow (m^3/se)	23	22	24	**20**	26	27	52	**86**	60	24	21	22
Release from Besaka (m^3/sec)	2	1.91	2.09	**1.74**	2.26	2.35	4.52	**7.48**	5.22	2.09	1.83	1.92

From the above table, the maximum and minimum flow possible to release from Lake Besaka to Awash River in dry and wet seasons are 7.48 m^3/sec and 1.74m^3/sec respectively. Hence, the maximum flow rate expected to be released from upper reservoir to Awash River in wet season is 7.48m^3/sec. The design of hydroelectric power penstock and other components of the plant considered this peak discharge.

4.4 Computation of Hydropower peak Plant Capacity and Components

Hydroelectric power is one of the components that this project was intended to address for the purpose of managing the flooding effect of Lake Besaka in releasing the pumped water of the Lake from the upper reservoir that located on the peak of mount Fentale to Awash River under a controlled gate system.

The upper reservoir is used as a forebay and as the water released in the penstock hits the turbine that located at 5300m and generates hydroelectric power. To minimize the head loss in the system, powerhouse for the system was designed to be located at elevation where the ground start gentle slope. Accordingly, wide receiver circular concrete channel was designed to convey water after hitting turbine to Awash River under regulated gate system.

However, the control gate is designed to be provided at upper reservoir to manage the rate of flow to the penstock based on the reason variation of flows in Awash River.

4.4.1 Upper Reservoir

The upper reservoir is designed to store the pumped water from Lake Besaka at the peak of mount Fantale. The average at an average bed elevation of the reservoir is 1416m a.m.s.l and it was formed as dry crater by volcano. The natural storage has the total surface area of 6.8km^2 and average depth of 8.8m at its mid-section.

The method used to estimate the size of the upper different from the normal hydropower or water supply dam reservoir sizing using net inflow mass curve and elevation area capacity curve.

- Area of lower base, $A_1 = \pi a^2$
- Area of upper Zone, $A_2 = \pi b^2$
- Area of Zone, $A_Z = 2\pi rh$
- Total area of the segmented Sphere ($A_T) = \pi(a^2 + b^2 + 2hR)$
- Total Volume of segmented Sphere ($V_T) = \frac{1}{6}\pi h(3a^2 + 3b^2 + h^2)$

The data GIS and global mapper indicated the proposed reservoir has:

- ✓ Base radius (a) =345m
- ✓ Average segment height (h)=8.8m
- ✓ Radius of the segment(b)=1,867m

Therefore, the total estimated storage capacity of the upper reservoir is;

$$V_T = \frac{1}{6}3.14 * 8.8[(3 * (345)^2 + 3(1,867)^2 + (8.8)^2]m^3 = 50MCM$$

The proposed reservoir is located on the peak of the mountain and the probability to be stilled up through its service age is very insignificant. Besides, the inflow to the reservoir is constant which 8.44m^3/sec.

The storage capacity of upper reservoir will not be affected and reduced by siltation. The effect of evaporation loss from the reservoir in the year is almost equivalent to the depth of perception over the surface of water in the reservoir (JICA 2015). Hence, annual evaporation loss from upper reservoir of $6.8km^2$ surface area is:

$$V_{ep}=3.023x6.8x10^6m^3$$
$$=20.9MCM/year$$

This value is almost equivalent to the change in volume of maximum annual inflow to Lake Besaka and maximum volume of drainage to Awash River. Hence, the summation of maximum volume of drainage i.e 70.06MCM and volume in evaporation loss i.e 20.9MCM is equivalent to the annual net inflow to Lake Besaka.

4.4.2 Trash rack

Trashrack is the structure used to prevent the entry of debris and floating materials. For small hydro plants the trash rack overall size is determined based on an approach velocity of 0.75 m/s to 1.0m/s. Trash racks may be designed in panels that can be lowered into place in grooves provided in the intake walls. The trashracks should to sloped at (0-20) from the vertical (4V:1H) to avoid the accumulation of bed loads on the racks. The spacing between bars is determined as a function of the spacing between turbine runner blades (Warnick 1984).

The objectives of good design are:

- To prevent entry of floating debris.
- To avoid formation of air entraining vortices.
- To minimize hydraulic losses

Generally, designs are based on excluding particles size greater than medium gravel size from (2 cm to 4 cm). A clear opening of 3.0 cm is recommended for design.

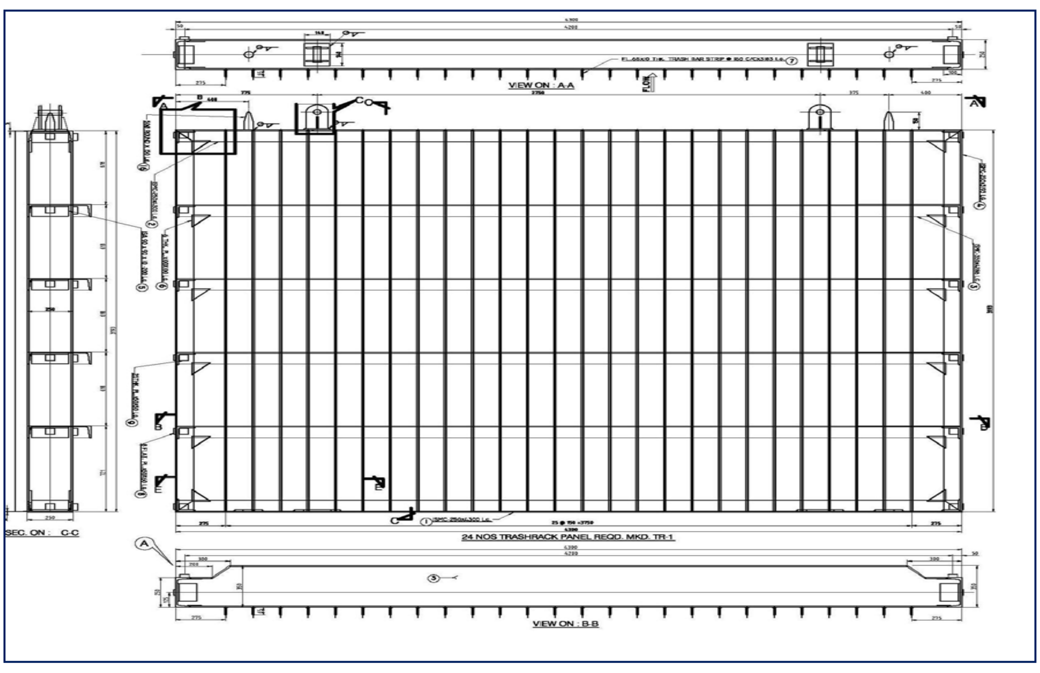


Fig.4.4.2a. Trashrack front view (C.C. Warnick 1984)

4.4.3 Penstock Gate

The gate of the penstock is designed in the manner it is strong enough to resist the hydrostatic pressure of water in the upper reservoir and to regulate the flow rate to be released to the penstock. Generally, the gates of the penstock should fulfill the following requirements;

- In closed position, the gate must be completely water tight with full pressure acting from upstream side and sealing must be reliable against maximum water level in the upper reservoir.
- The sealing of the wheel assemblies should prevent entry of water to the wheel bearings to ensure trouble free operation.
- The gate groove covers shall be design for crowd load of $500kg/m^2$.
- The following loads shall be considered:

i. Full hydro-static load on upstream side of the gate with water level at highest level of forebay/upper reservoir.

ii. The total hydro-static and hydro dynamic forces, frictional & wave loads when the gate is raised or lowered with the upstream water level at highest level of forebay. However, in addition to all the above mentioned requirements, the gate structure to be installed and fixed should have the function of regulating and controlling the maximum follow that the penstock and circular concrete canal convey it to Awash River. The gate structure should have graduated reading so that the flow quantity can be calculated and adjusted.

4.4.4 Penstock Intake

First of all, the direction of approach velocity should be axial with respect the intake if at all possible. For Hydraulic Design of Small Hydro Plants, the interesting point to be considered is significant risk of vortex problems.

For normal approach flow the submergence can be determined from the following formulae:

$$S = 0.725VD^{0.5}$$

Where:

S = submergence to the roof of the gate section (m)

D = diameter of penstock and height of gate (m)

V = velocity at gate for design flow (m/sec)

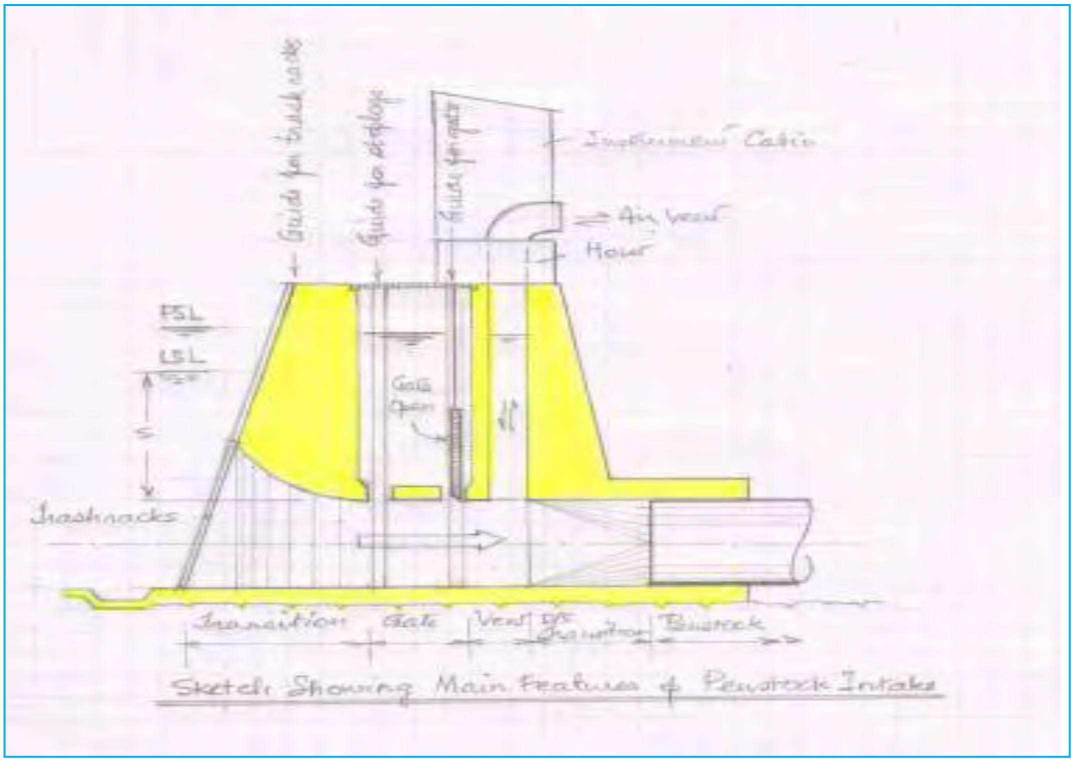


Fig.4.4.4a. Elevation of Trashrack, penstock intake, and gate structure (AHEC/MNRE/SHP, 2011)

4.4.5 Estimation of Design Head

After the excavation of earth material from the bed of the reservoir, compacting and packing clay material, the average bed elevation for the upper reservoir estimated to **1416m** a.m.s.l. The power house location was proposed at a counter and GPS elevation of **965m**. The change in ground elevation is **451m**.

4.4.5.1 Hydraulic Design of Penstock

The hydraulic design data required for conventional hydropower penstock are:

- The peak discharge of the penstock (Q_p=7.48m^3/sec).
- Length of the penstock, *(*L_p=5300m)
- Design head for the penstock, H_p, which is equal to the gross head of the scheme.
- Ultimate tensile strength of the penstock material.
- Diameter of the penstock (in mm).
- Flow velocity in the penstock for the chosen diameter. This value should be in the following ranges: 2-3 m/s for low head, 3-4 m/s for medium head and 4-5 m/s for high head schemes (head (H): low H < 50 m; medium 50 ≤ H ≤ 250, high H > 250).
- The minimum thickness, t_{min}, is automatically calculated according to the material of the penstock.

- The effective thickness, t_{eff}, is assumed to be equal to the minimum thickness.
- Thickness to allow for corrosion, salinity and alkalinity nature of the water.
- The safety factor, *SF*. If the value is less than 3.5 then increase the effective thickness, t_{eff}, until it the *SF* is at least 3.5.

The economical penstock size selection is depending on installation, material as well as whether used above ground or buried. Gordon and Penman (1979) give a very simple equation for determining penstock diameter for small hydropower installations:

$$D_p = 0.72Q^{0.5}$$

Where;

D_P = Diameter of the penstock (m)

Q= Peak estimated discharge (Qp =**7.48m³/s**)

$D_P = 0.72(7.48)^{0.5}$

=1.97m

The friction loss in the system is calculated for the indicated data as:

$$V_l = 1.274Q/D_e^2$$
$$=2.455/\text{sec}$$

$R_e = V*D_e/(\vartheta)$

Roughness of steel penstock

$\varepsilon = 0.045\text{mm}$

The average surface temperature of Lake Besaka is 28.3°c and its sanity content is 10.7dS/m. Besides, the kinetic viscosity (ϑ) the Lake water is $1.4593\text{x}10^{-6}\text{m}^2/\text{sec}$ (H.Ezzat Khalifa 1981). The friction loss in the penstock (h_f) is calculated for different values of penstock diameter iteratively using moody diagram till (h_f/H_g) 100% = (9-12) %

Reynolds number

$Re = V*D_e/\vartheta$

$=2.455*1.97/1.4593\text{x}10^{-6}$

$=3.314\text{x}10^{6}$

Relative roughness of steel

$= \varepsilon/De$

=0.045mm/1970mm

=0.000023

From Moody diagram, friction coefficient (f) =0.0140, using Darcy formula.

$$h_f = f\frac{L_p}{D_e}\frac{V^2}{2g}$$

Where:

h_f = Head loss (m)
f = Friction factor
L_p = Length of penstock (L_p=5300m)
D_p = Diameter of penstock (m)
V = Water velocity in penstock (m/s)
g = Gravity (m/s^2)

h_f = 11.57m

Where:

H_g = gross head
= (ground elevation + friction loss + minor friction loss)

Minor friction loss= 0.8% net head (H_n) (Karlos Martins, 2013).

H_d= (451-11.57) m
= 439.43m

Minor loss= 0.008*439.43m
=3.52m

Gross head= (11.57+3.52+439.43)m
=454.52m

(h_f/H_g) 100% = (11.57/454.52)100%=2.55%.

The iteration will continue till the ratio between (9-12) % and flow velocity in the penstock fall between (4-5)m/se as the head of the plant is greater than 250m.

Table.4.4.5.1a. Penstock friction loss iteration

Iteration	Diameter (m)	friction loss (h_f), m	flow velocity (m/s)	%(h_f/H_g)
1	1.97	11.57	2.45	2.55
2	1.9	13.19	2.64	2.90
3	1.8	18.18	2.94	4.0
4	1.7	24.19	3.30	5.32
5	1.6	32.75	3.72	7.2
6	**1.5**	**45.23**	**4.24**	**9.95**

4.1.1.1. Design of Penstock Thickness

Therefore, the follow results are used for penstock thickness estimation

- D_p=1.5m
- The material used is Mild steel (rolled welded steel pipe)
- Penstock length (Lp) = 5300 m
- Flow of water (Qp) = 7.48m^3/sec
- High gross (H gross) = 454.52 m
- Head Loss (h_f)= 45.23 m

- Minor friction loss= 3.25m
- Net Head = 402.52m
- Water velocity in the penstock (V) = 4.24m / s
- Efficiency of penstock (pipe eff) = H_n/ H_g * 100% = 88.56%
- K_w (bulk modulus of water) = 2.1*10^9 N/m^2
- σ_t (Tensile strength of steel penstock) = 400 * 10^6 N/m^2
- E (modulus of elasticity) = 206 * 10^9 N/m^2
- Kinetic viscosity of Lake water=1.4593x$10^{-6}$$m^2$/sec
- Working head (H_w)of the penstock (Mild steel) =1.5 * Gross Head (Hg)
  = 1.5 * 454.52)m
  =681.78m

Working pressure of the penstock= 1.5 * (H_g/ H_w)
= 1.0 kgf/cm^2 (Bars)
= 100,000 N/m^2 (Pascal)
= 0.0001 kN/mm^2

For thin tube ; D_p / t_p > 20

Where:

t = thickness of penstock in mm

e_s = extra thickness for corrosion (1-3) mm

Working pressure (P) = 0.0001 kN/mm^2

Dp = diameter of 1500 mm penstock

σ_t (Tensile strength) = 400* 10^6 N/m^2

= (400 * 10^6 N/m^2) * (kN/1000N) * (m^2/1000.000 mm^2)

= 0.400 kN/mm^2

- **Minimum penstock thickness (t_{min})**

t_{min}(mm) = (P * D_p) / (2 σ_t) + e_s.

Taken extra thickness for corrosion (es) = 3 mm

= (0.0001kN/mm^2x 1500mm) / (2 x 0.400kN/mm^2) + 3mm

= 3.19 mm

The impact of pipe handling in transportation, laying, deformation, etc., it is necessary to add more rapidly the penstock thickness (in the wills of 3 mm). So thick of penstock (t) is = (3+3.19)= **6.19mm.**

Effect of Water Hammer

In the design of penstock also must take into account the effects of water and control the speed lacing.

If the ratio of gross head to penstock length (H_g / L_p)> 5, the surge tank is required (C.C.Warnick 1984)

Hence, H_g / L_p =454.52/5300

= 0.086 < 5

From the above relationship, surge tank is not required but the effects of water hammer still be considered for safety of pump-turbine units and penstock.

At wills:

% Closure of the valve flow (Z) = 50%

with the closing time (T_{close}) = 4 seconds (fast enough)

Corrosion allowed thickness (e_s) = 3 mm

Overall safety factor (SF) = 4

The speed of water waves in the penstock:

$$C_{wave} = [(10^{(-3)} * K_w) / (1 + (K_w * D_p / E * t)]^{(0.5)}$$

Where:

K_w = bulk modulus of Lake water 2.1×10^{9} N/m^2

E_p = modulus of elasticity of penstock material 206 * 10^9 (N/m^2)

D_p = Penstock diameter 1500 mm

t = wall thickness 6.19 mm

L_p = length of penstock, 5300 m

By entering values:

$$C_{wave} = [(10^{-3} * 2.1*10^9) / (1 + (2.1*10^9 \times 1500 / (206 * 10^9 * 6)]^{0.5}$$
$$= 769.12 \text{ m / s}$$

Critical closing time of the penstock (T_c)

The time it takes the pressure wave to return again to the valve after the sudden closure, known as the critical time(T_{cri});

$$T_{cri} = 2 L / C_{wave}$$
$$= (2 \times 5300) \text{ m} / 769.12 \text{ m / s}.$$
$$= 13.78 \text{ seconds}$$

T_{Close} (4 sec) < T_{cri} (13.78 sec).

$$H_{surge} = C_{wave} * V * z / (980)$$

$$= 769.12 * 4.24 * 50 / (980)$$
$$= 166.38\text{m}$$

$$H_{total} = (H_{surge} + H_g)$$
$$= (166.38+454.52)\text{m}$$
$$= 620.90\text{m}$$

= 620.90 m (Less than working head: 620.90m <(H_w=681.78)

$(H_{surge} + H_g) < H_w$

Hence, the required thickness of the penstock (t_p) is calculated as:

$$t_p = (H_{total} * D_p * SF / 83700) + e_s$$
$$= (620.90\text{m} * 1500 \text{ mm} * 4) / 83700 + (3 \text{ mm})$$

= 47.51 mm

$t_p > t_{min}$

Hence, the initially estimated penstock thickness (t=6.19mm) is not adequate to withstand the internal pressure of the penstock.

4.4.6 Estimation of Peak Hydroelectric Power

The peak plant hydroelectric power generating capacity is calculated taking the maximum flow that can be released from the upper reservoir to Awash River during the wet season of the year and the net head between the intake of the penstock and outlet of the turbine.
Hence:

$$P_{Max} = \frac{\gamma Q H_n}{\eta} W$$

Where;

P_d= Plant power capacity (MW)
Q= Design Discharge (m^3/sec)
H_n =Net head at nozzle outlet (m)
η = Turbine efficiency ($\eta = 0.89$)

$$P_{max} = \frac{9.81x1100x7.48x402.52}{0.89} W = \mathbf{36.52MW}$$

Whereas, the minimum plant capacity is calculated using the minimum flow rate that is possible to be released from Lake Besaka to Awash River i.e. 1.74m^3/sec.

$$P_{min} = \frac{\gamma Q H_n}{\eta} W = \frac{9.81x1100x1.74x402.52}{0.89} W = \mathbf{8.49MW}$$

4.4.6.1 Selection & Design of Hydraulic Turbine

The potential energy in water is converted into mechanical energy in the turbine, by one of two fundamental and basically different mechanisms:

The water pressure can apply a force on the face of the runner blades, which decreases as it proceeds through the turbine. Turbines that operate in this way are called reaction turbines. The turbine casing, with the runner fully immersed in water, must be strong enough to withstand the operating pressure. Francis and Kaplan turbines belong to this category.

The water pressure is converted into kinetic energy before entering the runner. The kinetic energy is in the form of a high-speed jet that strikes the buckets, mounted on the periphery of the runner. Turbines that operate at high head and small discharge is Pelton turbine.

The selections criteria for hydraulic turbines depend on discharge, net head, geographical terrain, environmental requirement and labor cost. The hydraulic turbine selection criterion for this research is mainly focused on the range of discharge and net head. The net head for the conventional hydropower part is 402.52m and peak discharge is 7.48m^3/sec.

The turbine application chart below is indicating either Pelton type turbine or Francis turbine are recommended.

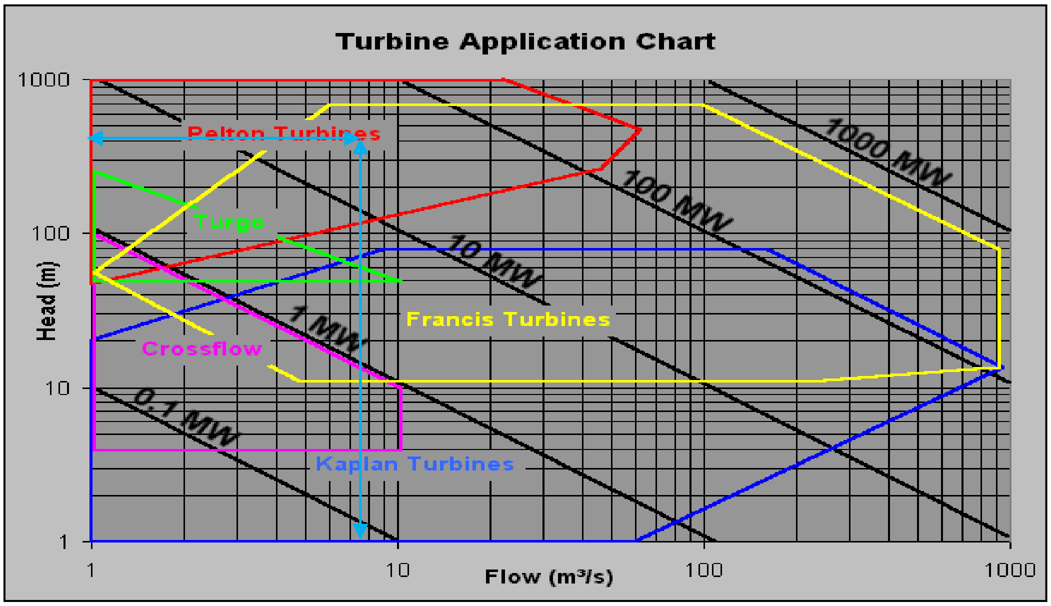


Fig.4.4.6.1a. Turbine selection chart based on Design head & Flow m³/sec of the plant (C.C.Warnick 1984)

However, Pelton type turbine is specifically best and selected for its good efficiency at highest head and low discharge. The efficiency of a Pelton is good from 30% to 100% of the maximum discharge for a one-jet turbine and from 10% to 100% for a multi-jet one.

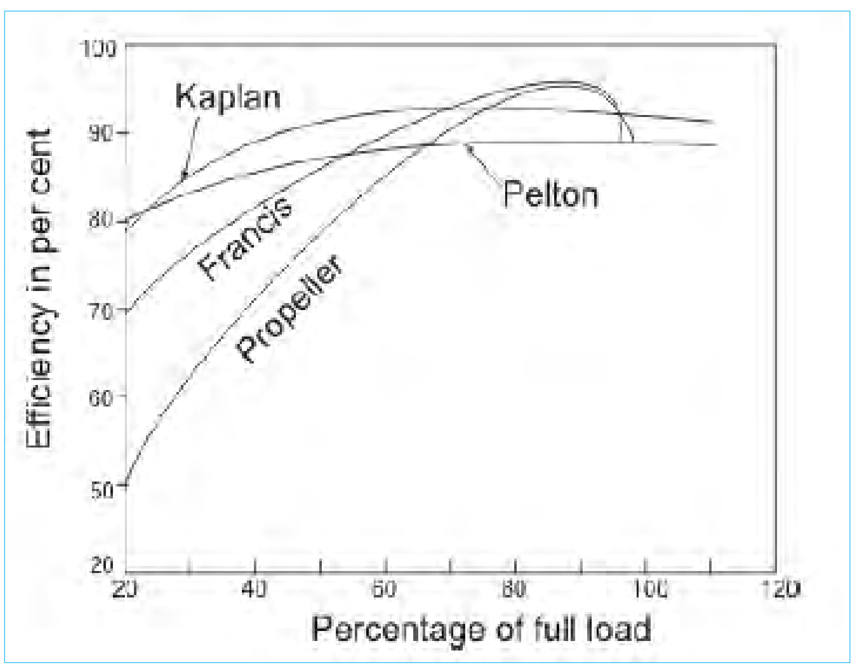


Fig.4.4.6.1b. Efficiencies curves for different turbines.

In general turbine manufacturers denote the specific speed of their turbines. A large number of statistical studies on a large number of schemes have established a correlation of the specific speed and the net head for each type of turbine.

Table. 4.4.6.1a. Best efficiencies of Pelton turbine of different nozzles (Bilal Abdallah 2012)

Pelton n nozzles	0.90
Pelton 1 nozzle	0.89

The specific speed of Pelton turbine is given by:

$$\mathrm{Nst} = 85.49\,\frac{\sqrt{n_j}}{{H_n}^{0.243}}$$

Where:

Q=Discharge (m^3/sec)

H_n= head at best turbine efficiency (m)

n_j=number of turbine nozzles (jets)

Nst= specific speed of the turbine

Table.4.4.6.1b. Possible specific speed ranges for turbines (Q.H.Nagpurwala 2011)

Type	N_s	
Pelton wheel (impulse)	12 – 60	very large head
Francis turbine (radial-flow)	60 – 500	large head
Kaplan turbine (axial-flow)	280 – 800	small head

Since, the turbine is aimed to work for small discharge, two nozzles are recommended:
Hence,

$$\mathrm{Ns} = 85.49\,\frac{\sqrt{\mathrm{n_j}}}{{\mathrm{H_n}}^{0.243}} = 28$$

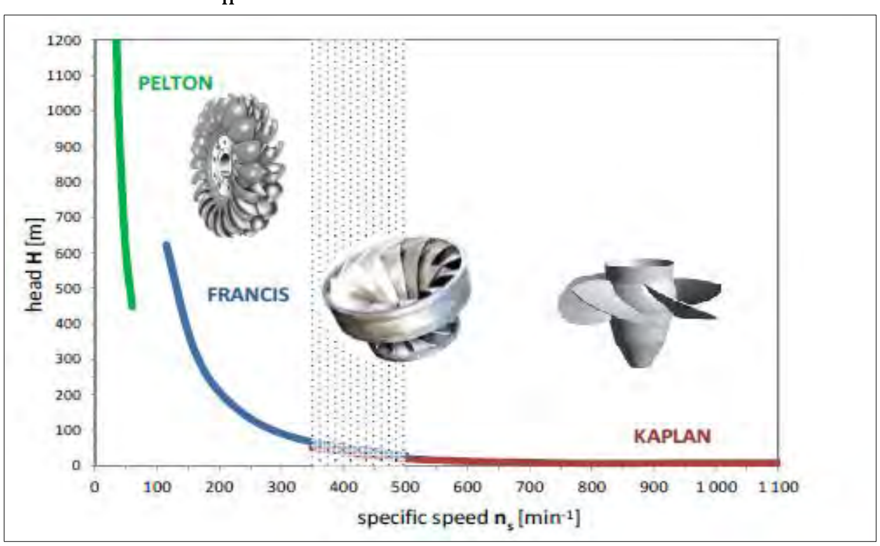


Fig.4.4.6.1a. Specific speed of different turbines (Dinesh Kumar 2013)

The rotational speed of the pelton wheel is calculated using;

$$N_s = N\frac{\sqrt{P}}{H^{\frac{5}{4}}}$$

Where:

P= the electrical power (KW)

N= synchronous speed of Pelton turbine (rpm)

H= Net head (m)

N_s= specific speed of the turbine

$$N_s = N\frac{\sqrt{P}}{H^{\frac{5}{4}}}; \qquad 28 = N\frac{\sqrt{36{,}520}}{(402.52)^{\frac{5}{4}}}; \qquad \mathbf{N = 264rpm}$$

For pelton turbine that having synchronous speed of (N=264) rpm and specific speed value of (N_s=28); the number of poles calculated as:

$$N = \frac{120f}{N_p}$$

Where;

N= number of rotation

f=frequency (50Hz)

N_p=Number of poles

Hence,

$$N = \frac{120f}{N_p}; \qquad 264rpm = \frac{120x50Hz}{N_p}; \qquad N_p = 23$$

For two nozzles of pelton turbine, the diameter describing the bucket, bucket width and nozzle diameter is given as shown below;

$$D_1 = 0.68\frac{\sqrt{H_n}}{N}$$

$$D_2 = 1.68\sqrt{\frac{Q}{n_{jet}}.\frac{1}{\sqrt{H_n}}}$$

$$D_e = 1.178\sqrt{\frac{Q}{n_{jet}}.\frac{1}{\sqrt{gH_n}}}$$

Where;

N = rotational speed (turns/s=t/s)

n_{jet} =Number of jet

D_1 =the diameter of the circle describing the bucket centre line (m)

B_2 = is the bucket width mainly depending on the discharge and number of nozzles (m)

D_e= is the nozzle diameter (m)

$$D_1 = 0.68\frac{\sqrt{402.52}}{4.4} = 3.1\text{m}$$

$$B_2 = 1.68\sqrt{\frac{7.48}{2}.\frac{1}{\sqrt{402.52}}} = 0.73\text{m}$$

$$D_e = 1.178\sqrt{\frac{7.48}{2}.\frac{1}{\sqrt{9.81\text{x}402.52}}} = 0.29\text{m}$$

D_1/B_2 =3.1/0.73 = 4.25

As a general rule, the ratio (**D_1/B_2**) must always be greater than 2.7 hence it is OK. If this is not the case, then a new calculation with a lower rational speed or more nozzles has to be carried out.

Then the runner diameter (D_r), at maximum efficiency of pelton turbine is calculated as;

$$D_r = 38.6\frac{\sqrt{H_n}}{N}; \qquad D_r = 38.6\frac{\sqrt{402.52}}{264} = \mathbf{2.93m}$$

The minimum clearance between the nozzle and the bucket is;

X_{nb} = 0.625xDr

= 0.625x2.93m

= 1.83m

The velocity of the water jet (V_j) through the nozzle in m/s, can be calculated using;

$$V_r = C_n\sqrt{2gH_n}$$

Where:

C_n= Nozzle (jet) discharge coefficient (≅ 0.98)

$$V_r = C_n\sqrt{2gH_n}; \qquad V_r = 0.98\sqrt{2\text{x}9.81\text{x}402.52} = \mathbf{87.1m/se}$$

The jet diameter D_j is calculated as;

$$D_j = \sqrt{\frac{4Q_t}{\pi V_j n_j}}; \qquad D_j = \sqrt{\frac{4x7.48}{\pi x87.1x2}} = \mathbf{0.23m}$$

The bucket axial width of the turbine is (B_w) =3.4D_j=0.78m

The bucket radial length (B_l) is :

B_l=3D_j= 3x0.23m=0.69m

The number of the bucket in each runner is determined so that no water particle was lost while minimizing the risk of detrimental interaction between the outflow water particle and the adjacent bucket.

It is calculated as;

$$n_b = 15 + \frac{D_r}{2D_j}$$

Hence;

$$n_b = 15 + \frac{2.93}{2x0.23} = \mathbf{21}$$

Number of Pelton turbine buckets should be greater than 17 and hence, it is OK.

4.4.7 Powerhouse

Power house is the main area of the hydropower station**.** It is primarily an electro-mechanical field and civil structures are planned and designed to provide proper housing of electro-mechanical equipment. The function various components of power house for which the design should make adequate arrangements are given under;

- **Spiral case and wicket gates;**
  It is used to direct and control the water entering the turbine runner. The spiral case is a steel-lined conduit connected to the penstock. Wicket gates are adjustable vanes that surround the turbine runner entrances and they control the area available for water to enter the turbine. Wicket gates settings are controlled by the governor.
- **Turbine**
  Converts potential energy of water in mechanical energy which is intern drives the generator.
- **Generator**
  Converts to the mechanical power produced by the turbine into electrical power. The two major components of the generator are the rotor and stator.

The rotor is the rotating assembly, which is attached by connecting the shaft to the turbine and the stator is fixed portion of the generator.

- **Governors**

  Regulates the speed and output of the turbine-generator units by controlling the wicket gates to adjust water flow through the turbine.
- **Draft tube**

  Conveys the water from form the discharge side of the turbine to the tailrace. It is designed to minimize the exit losses (C.C.Warnick 1984).

4.4.7.1 Design of PH

Like the design of any civil structure, the design of power house should consider the following points in addition to live load, dead load, wind load and seismic design load.

- Safety against the structural failure due to over stressing.
- Safety against overturning.
- Safety against sliding
- Safety against uplifting pressure.

4.4.7.2 Design of PH Dimensions

The design of power house dimension is mainly depends on the number of units and the components proposed to be included in the power house.

i. **Length:**

The power house length is mainly depends on the number of units and can be found by the following formulae.

$$L_{ph}= (5D_r+2.5)m$$

Where:

$L_{ph}=$ Length of power house (m)

D= diameter of outer (m)

$L_{ph}= (5*2.93+2.5)m$, $D_r = 2.93m$

$=$ **17.15m**

For high speed specific turbine;

B = 4Dr + 2.5m for high specific speed turbines

= (4*2.93 +2.5)m=**14.22m**

ii. **Width;**

Equal to the center to center spacing of the units plus the clearance space from the wall

iii. **Height;**

Fixed by the head room requirements of the crane operation.

4.4.7.3 Cavitation

When hydrodynamic pressure in the liquid flow falls below the vapor pressure of the liquid, there is the formation of the vapor phase. This phenomenon induces the formation of small bubbles that are carried out the low pressure region by the flow and collapse in the region of higher pressure. The formation of these bubbles and their subsequent collapse gives rises to what is called capitation.
Cavitation may be avoided by suitably designing, installing and operating the turbine in such a way that the pressures within the unit are above the vapor pressure of the water. The cavitation characteristic of a hydraulic machine is defined as the cavitation coefficient or plant sigma (σ), given by;

$$\sigma = (Ha - Hv - Ys)/H$$

Where;

Ha – Hv = Hb, is the barometric pressure head (at sea level and 28.3°C, Hb = 10.1 m), and H is the effective head on the runner.

4.4.7.4 Collection Basin

The collection basin was proposed immediately after tailrace to collect the water coming out of the draft tube and guide water to the circular concrete canal which is to be laid for 6km length over a gentle bed slope from the powerhouse to Awash River. The collection basin should be designed in the manner that it fully accommodates the water coming from the turbine and directly guides the flow to the circular canal inlet. It also help full to reduce the flow speed so that the cavitations will be avoided.

4.5 Hydraulic Design of Drainage Pipe

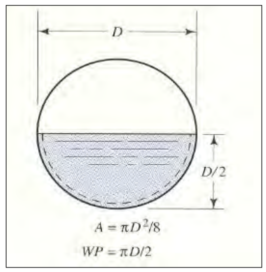
As the topographic nature of the area Lake Besaka area is indicating, there is a gentle slope and plane except for the elevation difference between the peak of Mount Fantale and Lake Besaka which has high change in elevation. The powerhouse of the conventional hydropower part was located at the foot of mount Fantale than locating the powerhouse at near Awash River which is 11.5km from the upper reservoir.
In shortening the length of the penstock and locate the power house at the foot of the mount Fantale, it is primarily advantageous to reduce the cost of penstock and energy by friction. The distance between the proposed location of the powerhouse and the point at which the water from the upper reservoir effluent to Awash River is 6km with a maximum change in ground elevation is less than 10m.
Circular reinforced concrete canal is proposed as a drainage canal for water passage from the powerhouse to Awash River. In designing the canal, half section flow was recommended to make canal efficient section. The method of determining the dimensions of the canal Manning's formula is adopted.

The velocity of the flow in the canal is given by;

$$V = \frac{1}{n} * R^{\frac{2}{3}} S^{\frac{1}{2}}$$

Where;

R= the wetted perimeter
n= the Manning's coefficient
s= the slope of the canal



The wetted perimeter(R) = $D/2$ and the wetted area of the canal

A=$\frac{\pi D^2}{8}$

The flow in the canal is given as:

$$Q = AV = \frac{1}{n}\left[\frac{\pi D^2}{8}\right] . [\frac{D}{2}]^{\frac{2}{3}} S^{\frac{1}{2}}$$

$$Q = \frac{1}{n}\left[\frac{D^{\frac{8}{3}} S^{\frac{1}{2}}}{4}\right]$$

The bed slope of the entire **7000m** canal is 0.002, **S=0.002** and the manning's coefficient for concrete pipe is **n=0.013** and the design flow is **7.48m^3/s**.
Therefore, solving for the wetted perimeter of the canal, D= m and adopting **D=2.25m**

The flow velocity in the canal is calculated using manning's formula as:

$$V = \frac{1}{n}[(R^{\frac{2}{3}})(S^{\frac{1}{2}})]; \qquad V = \frac{1}{0.013}(1.125^{\frac{2}{3}})(0.002^{\frac{1}{2}}) = \mathbf{3.72 m/sec}$$

For the maintenance purpose, concrete manhole covers at 500m interval should be provided. At the canal and high way crossing point, the canal should passing in the form of culvert with supporting reinforced rectangular bridge structure. And finally, the propose site for mixing with Awash River is the rock place where the scouring effect will not damage both the river terrain and the canal.

4.5.1 Layout of the Penstock and Drainage Canal

The layout for the penstock was selected along the route with minimum length, and cost for anchoring the pipe. The shortest route with minimum gulley and hills was selected so as to reduce the cost of penstock both for pressure line and gravity flows. The penstock length for pressure line is 4200m whereas for the gravity part is 5300m.

The drainage canal is made from reinforced concrete pipe of diameter 2.25m. As the objective is to drain canal is Lake Besaka water to Awash River, there are communities residents, public infrastructures, sugar plantation pipe line and sugar factory building will be faced while choosing the shortest path.

However, the drainage canal route was delineated and shifted in the manner that the line will minimally affect these structures. This is because; the cost of reinforced concrete production per meter is much cheaper than the cost of compensation for residential building on one square meter.

Consequently, the project water conveyance systems and other components are delineated for optimum length and cost as shown.

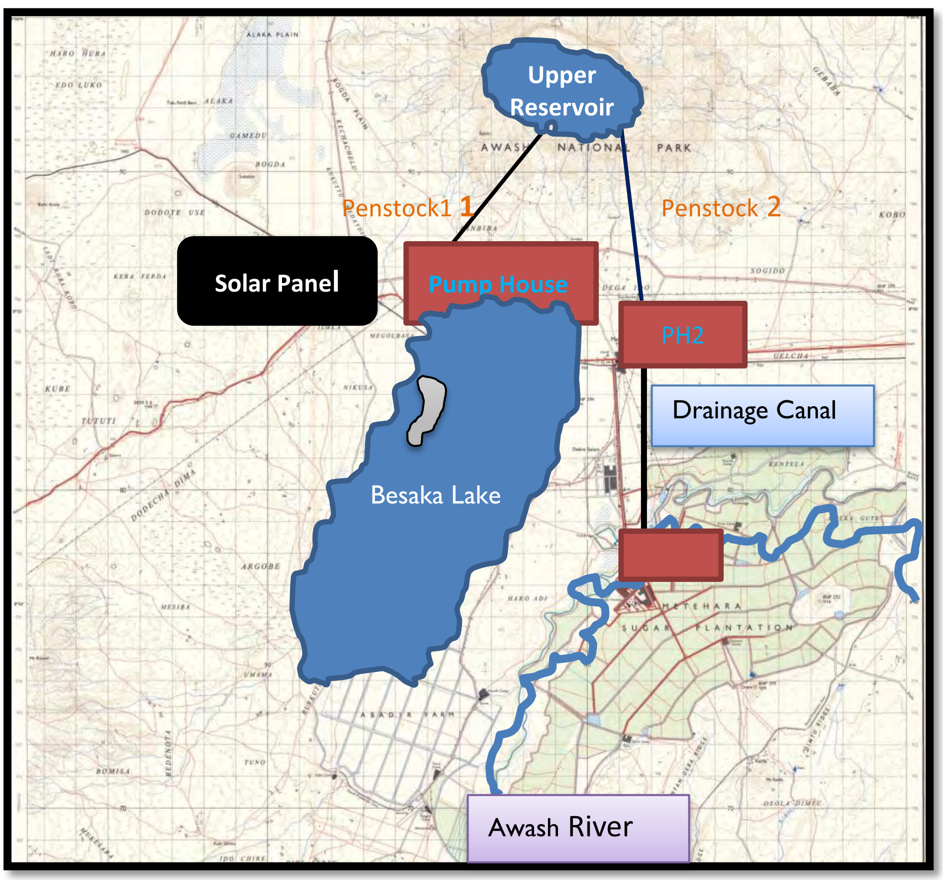


Fig. 4.5a. Research Project Layout

4.6 Pumping and Draining Schedules

The pumping of the Lake Water to the upper reservoir and releasing it to Awash River needs well scheduled operational time so that the contamination effect of the Lake will be lowered and maintained as per the water quality standards stated in this paper.

The operation (pumping & draining) schedules were prepared in order not to put the water quality of Awash River in danger both during dry and wet seasons of the year. The operation schedules were outlined in the manner that the maximum flow (7.48m^3/sec) to be released to Awash River during the season when the average month flow in the river is 86m^3/sec. Besides, the minimum flow of 1.74m^3/sec will be released from upper reservoir to Awash River during dry season of the year.

Therefore, the pumping the Lake water and draining it from upper reservoir to Awash River is scheduled with identified tolerable mixing ratios as shown below to definitely control the EC value of the Awash River at a value less than 960µS/cm at dry and wet season of the year.
This operation schedule is very important for the efficient use of upper reservoir to store pumped water and regulate the flow while releasing it to Awash River through the generation hydroelectric power.
Pump
The annual volume of water to be pumped from Lake Besaka as per the above schedule is MCM and volume of water that will be drained to Awash River annually is 92MCM. The amount of evaporation estimated based on the ELSA equation of JICA 2015 and it is equal to 1.7MCM.

4.6.1 Pumping Schedule

The paper findings indicated as 8.44m^3/sec water should be pumped from Lake Besaka to upper reservoir for 8.5hrs per day in all seasons. Hence, the volume of water that pumped from the Lake annual as indicate in the table below is greater that the Lake inflow volume i.e 92.2MCM. Besides, the pumping hours can be increased from 8.5hrs minimum sunshine in the month of July to 10.3hr in November. Though, the minimum pumping volume of Lake Water per year is shown in the table below and the pumping can be started in any one of the month.

Table. 4.6.1a. Pumping volume in each months of the year.

Month	Jan	Feb	Mar	Apr	May	June	Jul	Aug	Sep	Oct	Nov	Dec	**Total Volume**
Pumping volume in MCM	8	7.23	8	7.75	8	7.75	8	8	7.75	8	7.75	8	**94.23MCM**

4.6.2 Drainage Schedules

The pumped Lake water is released from upper reservoir to Awash River based on the seasonal variation of flow in the Awash River. Besides, evacuation of water begins when half upper reservoir volume is filled with water till reservoir dead storage and dynamic level become stable .This is advantageous to form uniform flow in the areas of penstock gate.

Hence, releasing water from the upper reservoir is scheduled to start in the first week of April till the reservoir filled up to 25MCM.

Month	Jan	Feb	Mar	Apr	May	June	Jul	Aug	Sep	Oct	Nov	Dec
Pumping volume in MCM	8	7.23	8	**7.75**	**8**	**7.75**	**8**	**8**	**7.75**	**8**	**7.75**	**8**
Release to Awash River in MCM	**4.02**	**3.5**	**4.2**	**3.4**	**4.5**	**4.6**	**9.1**	**15**	**10.2**	**4.2**	**3.6**	**3.9**

The combined pumping and releasing of the Lake water is based on the table indicated above and shaded overlapping. The maximum volume of water that can be released to Awash River per year is estimated based on the seasonal variation, water budget of Awash River and tolerable mixing ration.

5 CAPITAL COST OF THE PROJECT AND ITS BENEFITS

The cost estimation of the project and its benefits were considered from different angles for the cost solar energy and hydroelectric power. It is definite that, the growing effect of Lake Besaka is not only affecting the surrounding community and public infrastructure facilities but also the downstream mega irrigation and water supply projects as well.

And this is the reason why different researchers from in and out of the country have been conducting studies to identify the sources of Lake Besaka inflow and the remedial measures to tackle the growing effect of the Lake. And this research is aimed to find sustainable solution to control the expansion of the Lake. And it is indirectly to generate hydropower as the water released to Awash River in regulated manner.

As it was mentioned above, the implementation of this project has extended benefits in protecting the downstream water supply and irrigation projects. Besides, it will ensure the sustainable controlling of Lake Besaka expansion for the protection of public and government infrastructures in the vicinity of the Lake.

As Ethiopia is implementing this multi-purpose project, it will be an exceptional project in:

- The potential of water bodies all over the world are reducing due to climatic changes, however, Lake Besaka is alarmingly growing and invading the surrounding.
- The sanity and alkaline effect of the Lake is much higher than tolerable concentration to use it water for other purpose.
- The growing effect of the Lake has been damaging the surrounding private, public and government properties and becoming the threat for the Awash River on which the government of Ethiopia has built mega irrigation projects.
- The project is integrated environmental friendly energy production plants.

So, considering all these major benefits of the projects the capital investment cost for implementation was roughly estimated. The total capital cost estimation of the project was tried to be subdivided into the following parts;

- Capital cost of Solar Power for pumping.
- Capital cost for pump-turbine, pressure penstock and pump house.
- Capital cost for conventional Hydropower, 6000m long reinforced drainage canal
- Operation and Maintenance cost of the project.

5.1 Capital Cost of Solar Power

The capital investment cost for solar power for the project encompassed all solar energy producing components of the project and system facilities for night time operation. The capital investment cost for solar power was made based on the plant capacity, the quality of panels to be used and light towers for security purpose. Many preliminary cost estimation for solar energy have been made based on the forecasted the capital project cost of solar power /KW for the period 2008-2050.

According to (IRENA 2012) report, the capital cost for silicon PV which has 22% efficiency at 28.3°C per KW is 4340USD which is 95,480Eth Birr. This is about 4.16 Billion Eth Birr for solar plant which has the capacity to generate **43.58MW**. However, by adding the cost of land leveling and earth work, installation and transpiration cost for mechanical equipment was tabulated as shown below.

Table. 5.1. Solar Power Capital cost Estimation

S/N	Item Description	unit	Qty	Unit Price (Birr)	Total
1	Land leveling and earth work	m^2	250,000	106	26.5million
2	Civil structures, materials and installation	ls	1	14.3million	0.65million
3	Mechanical equipment supply and installation	ls	1	55million	2.5million
4	Electrical/I &C materials supply and installation	ls	1	65million	65million
	Total cost				120million
	Contingency (15%)				18million
	Grand Total for Solar Power				**138million**

5.2 Capital Cost Pumping Components

These costs will include all components of electro-mechanicals, construction of powerhouse and upper reservoir and the cost of Penstock or pressure pipe.

Table. 5.2. Capital Cost of Pumped Storage Hydropower

S/N	Item Description	Unit	Qty	Unit Price (Birr)	Total Cost (Birr)
1	Supply and installation electro-mechanicals	ls	1	30miilion	30million
2	Construction of power house	no	1	38.5million	38.5million
3	Supply and lay steel penstock pipe of specified thickness and anchor blocks	m	4200	25,000	113.4million
4	Excavation, compacting, filling impervious clay material in upper reservoir	km^2	7	44million	308million
5	Construction upper reservoir inlet structure and excavation of lake bed at powerhouse for pump-turbine suction facilities	ls	1	27.28million	27.28million
	Total				889million
	Contingency (3%)				26.69million
	Grand Project Total				**916.5million**

5.3 Capital Costs for Hydro power Plant and Drainage System

The hydroelectric power that will be produced in draining water from the upper reservoir to Awash River is only different in its nature and source of water is not from River. As a result, the cost for appurtenance structures like emergency spillway, auxiliaries and diversion structure are not applicable and considered in this research.

The plant is directly, receiving non-turbid water from the upper reservoir which is supposed to be used as a forebay.

The major plant components of the hydropower generating component for this project are;

- Trash rack, penstock inlet and gate structures.
- Penstock and Anchor blocks.
- Power house and Turbine.
- Construction of collection basin next to tailrace so as to guide water to Awash River through drainage concrete pipe.
- Construction of 2.25m diameter and 6000m long concrete channel

Whereas, the costs of lining upper reservoir and earth work were included in the estimated cost for pumping system components.

Table. 5.3. Cost of plant components to generate Hydroelectric Power

S/N	Item Description	Unit	Qty	Unit Price (Birr)	Total Cost (Birr)
1	Supply and installation electro-mechanicals	ls	1	80.3million	80.3million
2	Construction of power house	no	1	38.5million	38.5million
3	Supply and lay steel penstock pipe of specified thickness and anchor blocks	m	5300	28,560	151.4million
4	Supply and lay 2.25m diameter concrete pipe	m	6000	6,500	39million
5	Construction of upper reservoir outlets, trash rack and penstock gates, and collection basin	ls	1	27.28million	27.28million
	Total				336.5million
	Contingency (3%)				10million
	Grand Project Total				**346.5million**

5.4 Operation and Maintenance Cost

The operation and maintenance (O&M) costs of hydropower is between 1.5% and 2.5% of investment cost per year (IRENA 2012). However, the project is multi-disciplinary one and its operation and maintenance cost is much more than the sated percentage. For instance, solar power plants need frequent cleanings of the panels, and where as the operation and maintenance for pumping plants are also high when compared to components that generate hydropower. The water intake, water conduit system, and associated equipment are all vital to the functionality of the hydropower plant. Cavitation and erosion should be monitored, and regular inspection of runners of turbines should be carried out and recorded. Compared to oil-fired generators, operation, maintenance, replacement, and fuel costs are minimal for up rated hydroelectric plants.

In general, the operation cost of the project covers the cost of wage payment for operators and controllers. Whereas, the maintenance costs, include the cost of maintaining parts and re-installing any failed electro-mechanicals in the design period of the project. Considering the cumulative operation and maintenance cost of the three plants combined, the overall O& M cost percentage can be determined. Accordingly, the average operation and maintenance cost for the whole project ranges estimated and ranges 2.5% to 4.5%.

5.5 Cost-Benefit Relationships of the Project

In spite the fact that it needs detail cost benefit analysis, the preliminary cost estimations presented above will give initial clue to describe as the project remarkably benefits the country and the impacted community. For instance, protecting Awash River and downstream Mega irrigation and water supply projects from severe contamination of Lake Beseka and also clearing the submerged public infrastructures and 380ha sugar plantation farm are the key benefits to be underlined.

Besides, the implementation of the project has notable indirect benefit in producing the peak power of 36.52MW which supplements the national power demand. Likewise, the solar power plant capacity of 43.58MW was designed for pumping is also supplement the local energy demand when the pumping demand met. This reality will boldly indicate as the proposed project will generate power and supply an average **28.15MW** to the local community and investments in addition to solving the primary objective of this research in draining **92MCM** of water from Lake Besaka.

Moreover, the implementation of project has also benefits the country in serving as a research centre for technology students from Ethiopia and Africa because of the project uniqueness, multipurpose, and environmentally friend.

6 ENVIRONMENTAL IMPACTS OF THE PROJECT

Like that of any other developmental mega projects, the implementation of the project that control the expansion of Lake Besaka through solar-hydroelectric power has also its own positive and negative impacts to the environment and human life. This research was conducted in aiming to bring remarkable positive impact and sustainable solution in controlling the expansion of Lake Besaka through solar-hydroelectric power generation.

As the environmental impact assessment that conducted by (OWWDSE 2014) indicated, the growing effect of Lake Besaka has been strongly devastating the life of surrounding community and it is also threat for Awash River in future. So, the intention of this research is to sustainably manage the social and environmental crises that the Lake has been bring to the surrounding community, Awash River and to the national mega irrigation projects that constructed at the downstream.

However, the implementation the project and its components will cause the following minimum and tolerable impacts to the environment and human life.

4.2. Impacts of Solar Power Plant

The solar power plant collector proposed in this paper as a pumping energy covers a total area of 490,213m^2. For the installation of panels and tower of security light need clearing of bushes and excavation for land leveling. This will strongly affects the agro-ecology of the area as it is in the boundary of Awash National Park.

Besides, the clearing of bushes also directly contributes to the climatic change and discomforts wild animals found at the periphery of the Awash National park that are coming to the Lake. Besides, the solar reflection from the panels will contribute to the rise in the temperature of the area. However, the area required for solar panels installation will not total displace any part of the community except reduces their grazing land size during rainy season.

6.1 Pumping Water to Upper Reservoir

The project was designed to pump water from Lake Besaka to the upper reservoir located on the peak of mount Fanate. The upper reservoir is a natural depression which created by volcanic eruption as a crater and has the capacity to store $50x10^6m^3$ of water.

When the required volume of water pumped to the upper reservoir, it will create artificial lake which has the surface area of 6.8km^2. The creation of this artificial lake on the peak of mount Fantale will positively and negatively impacts the eco-system of the area. The lake will serve as the habitat for the national park wildlife and relaxation site for visitors of the project.

However, it the natural eco-system of the crater will change due to excavation, backfill and compacting, lining works and filling 50MCM water.

6.2 Releasing Lake Besaka Water to Awash River

As the natural chemical content of Lake Besaka is intolerable for irrigation and drinking, the mixing of any volume of this water to any water body is not recommended. This is because of the severe salinity and alkalinity effects of the Lake which negatively impacts the water quality of Awash River. And this has been the main reason why the Lake has becoming the threat to Awash River and mega irrigation projects at the downstream. And, it is the main reason why Awash Basin Authority and Ministry of Water, Irrigation and Electricity have obliged to work and seek sustainable mitigation measures despite the fact that no promising change has yet come in this regard.

However, the findings of this research indicated as it is possible to control the expansion of the Lake with minimum and tolerable contamination effects of Lake Besaka to Awash River. The degree of contamination of Lake Besaka can be controlled by adopting average safe mixing ratios that indicated in the previous researches.

7 CONCLUSION AND RECOMMENDATION

I. CONCLUSION

Due to the current contamination effects of Lake Besaka and its future risks to Awash River and downstream mega irrigation projects, many researchers and government consulting firms have been seeking mitigation measures to the Lake Expansion. For instance, WWDSE and OWWDSE have been conducting intensive studies to predict the sources of the inflow to the Lake and design long lasting mitigation measure to ensure the safety of the nearby community and mega projects constructed on Awash River at the downstream of the Lake.

However, the changes observed to date were not promisingly tackling the impacts when compared to the rate of the Lake growth and the impacts it has been posing on the environment and the life of the indigenous community. And this is one of the primary reasons motivated me to conduct this research to sustainably manage its flooding effect of the Lake through solar hydroelectric power generation. Consequently, the research final result indicated excellent possibility of Controlling the Lake Expansion without causing severe environmental impact and contamination to Awash River.

Accordingly, this paper indicated as it is possible to control the expansion of Lake Besaka through pumping the flow of 8.44m3**/s** for 8.5hrs per day which accounts 92MCM per year. The solar energy of 43.58MW capacity is designed to pump water from Lake Besaka to upper reservoir which located at 397.2m total head. The finding also indicated as it is possible to generate 36.52MW peak power. The ratios of mixing water from Lake Besaka and Awash River was estimated based on the tolerable EC valued that recommended for irrigation and drinking water by FAO, UNDP, WHO and EPA.

In general, implementing the finding of this research will ensure sustainable Control of Lake Besaka Expansion and minimize its future threats as a contaminant of Awash River. Besides, it benefits the country in generating 36.522MW peak power and 43.58MW solar energy in addition to achieving the primary objective of the paper.

II. RECOMMENDATIONS

It is highly recommended to follow the points that mentioned below, if the project is supposed to be implemented.

- Conducting detail geotechnical investigations should be done at upper reservoir, powerhouses and detail cost estimations.
- Efficient irrigation water application methods should be designed and implemented for the remaining 10,000ha command area of Fantale Irrigation project so as to maintain the current inflow of water to Lake Besaka at $2.92m^3/s$.
- The buffering of water from Lake Besaka to Awash River should not be more than 8% and 92% respectively in all seasons.
- The plant capacity for solar energy should not be less than be a 24MW for the smooth operation and safety of Turbine-pump.
- The electro-mechanicals for pumping and hydroelectric power generation should consider the design results so as to resist the harsh saline and alkaline effects of Lake Besaka to serve for the design period of the project.

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Table A-1: Monthly Rainfall

Year	Jan	Feb	mar	Apr	May	Jun	July	Aug	Sep	Oct	Nov	Dec	Annual
1966	0.0	114.0	3.0	61.0	11.0	44.0	128.0	169.0	93.0	30.0	4.0	0.0	657.0
1967	0.0	0.0	68.0	58.0	51.0	52.0	119.0	231.0	51.0	44.0	110.0	0.0	784.0
1968	0.0	99.0	3.0	54.5	5.5	41.0	150.1	80.0	43.5	10.5	31.0	14.0	532.1
1969	65.0	122.5	34.0	36.5	44.5	25.0	144.0	177.0	21.0	0.0	5.5	0.0	675.0
1970	24.0	24.0	89.0	9.5	6.0	1.0	127.0	171.0	39.2	32.0	0.0	0.0	522.7
1971	0.0	0.0	1.5	21.5	27.5	39.0	159.5	186.0	22.5	0.0	24.0	16.5	498.0
1972	0.0	75.0	45.0	19.5	59.0	31.0	57.0	82.5	25.5	10.0	0.0	0.0	404.5
1973	0.0	0.0	0.0	7.2	22.4	25.4	112.6	95.8	52.0	9.1	0.0	0.0	324.5
1974	0.0	0.0	70.8	4.2	46.9	35.9	52.5	52.1	100.9	1.1	0.0	0.0	364.4
1975	0.5	31.9	52.1	49.8	51.0	58.1	186.4	195.4	49.6	2.8	0.0	0.0	677.6
1976	0.0	0.0	57.2	53.5	52.3	8.9	122.5	146.6	30.5	7.9	30.3	2.7	512.4
1977	110.0	46.4	6.7	68.4	87.6	102.5	74.8	70.0	18.2	223.4	18.2	0.3	826.5
1978	0.0	105.9	0.0	5.9	16.1	3.7	98.7	89.6	51.5	33.7	5.0	29.8	439.9
1979	75.9	0.0	51.4	4.5	59.0	42.0	135.0	74.7	53.2	13.5	0.0	3.0	512.2
1980	9.8	0.0	4.9	46.7	15.4	29.7	131.9	182.9	33.5	6.9	12.0	0.0	473.7
1981	0.0	1.4	129.9	72.1	2.7	0.0	173.6	189.5	61.8	11.1	0.0	0.0	642.1
1982	32.0	67.1	68.1	21.2	101.4	3.1	101.2	304.3	35.9	92.2	36.0	1.0	863.5
1983	6.2	49.2	63.5	49.4	93.0	21.2	139.1	142.6	18.2	8.2	0.0	0.0	590.6
1984	0.0	0.0	13.4	0.0	62.6	33.0	66.3	69.4	70.8	0.0	0.0	7.7	323.2
1985	5.7	0.0	19.5	89.7	71.8	4.6	173.4	131.2	44.2	8.6	0.0	0.0	548.7
1986	0.0	55.9	49.2	26.1	4.7	78.3	75.2	85.4	52.6	5.4	0.0	4.0	436.8
1987	0.0	11.1	70.4	45.5	75.0	0.0	35.6	123.7	9.5	5.0	0.0	0.0	375.8
1988	19.9	19.3	19.1	41.4	7.1	10.9	158.6	131.2	83.6	36.6	0.0	11.2	538.9
1989	0.0	27.9	89.0	102.3	2.6	62.7	54.8	163.3	24.7	0.0	0.0	18.8	546.1
1990	0.5	221.7	97.7	67.9	3.8	1.9	181.6	82.2	73.6	3.5	0.0	1.6	736.0
1991	0.0	54.2	64.6	27.0	51.7	13.8	171.1	108.7	80.7	0.0	0.0	0.0	571.8
1992	22.1	0.0	1.7	77.9	17.3	41.4	97.7	163.7	79.6	49.7	1.9	2.6	555.6
1993	39.9	38.3	0.0	142.1	99.9	24.9	93.9	94.4	46.2	31.3	0.0	49.6	660.5
1994	0.0	0.0	3.0	33.1	44.0	41.1	246.9	115.4	60.6	0.0	11.1	1.5	556.7
1995	0.0	43.6	66.0	42.0	10.9	17.9	74.2	129.9	72.7	0.9	0.0	0.0	458.1
1996	25.3	0.8	92.3	41.0	112.4	30.9	137.5	105.8	69.0	11.9	13.7	0.0	640.6
1997	35.5	0.0	16.0	27.3	12.5	37.6	167.9	49.4	49.0	114.6	9.8	0.0	519.6
1998	39.9	44.5	56.5	34.3	12.5	7.8	77.8	138.6	37.0	79.7	0.0	0.0	522.0
1999	1.0	0.0	76.3	16.0	24.0	47.8	121.9	137.7	16.4	86.9	2.3	0.0	530.3
2000	0.0	0.0	9.0	22.9	41.7	26.7	116.8	166.7	34.2	49.0	5.0	9.7	481.7
2001	0.0	7.6	106.3	19.1	12.2	13.8	152.4	66.7	30.5	7.0	0.0	0.0	415.6
2002	0.0	0.0	61.1	17.5	8.5	18.0	44.9	105.1	25.5	1.3	0.0	26.8	308.7
2003	30.0	12.8	68.7	31.4	0.8	28.8	157.3	232.9	33.4	0.0	1.9	17.2	615.2
2004	42.4	3.5	112.4	116.8	0.0	14.9	95.2	108.3	33.0	7.3	2.3	0.0	536.1
2005	24.7	0.0	44.0	39.1	63.1	34.8	128.1	119.9	28.1	0.0	0.0	0.0	481.8
2006	0.0	69.8	64.1	35.0	8.1	5.4	160.1	100.3	20.9	37.1	0.0	65.7	566.5
2007	8.0	38.6	142.8	36.1	12.2	11.0	131.1	156.0	50.0	4.2	17.1	0.0	607.1
2008	0.0	0.0	0.0	22.6	43.3	13.5	222.7	84.0	10.7	40.4	44.4	0.0	481.6
2009	83.5	0.0	9.0	25.4	11.6	16.9	65.2	79.9	42.2	56.1	2.7	30.1	422.6
2010	0.0	114.0	3.0	61.0	11.0	44.0	128.0	169.0	93.0	30.0	4.0	0.0	657.0
Mean	15.6	31.9	48.2	41.4	36.8	27.1	124.2	128.6	44.9	26.9	8.6	8.4	542.5

Table A-2: Maximum Temperature, °c

Year	Jan	Feb	mar	Apr	may	Jun	July	Aug	Sep	Oct	Nov	Dec	Average
1966	31.3	29.1	31.4	34.3	36.9	36.0	33.4	32.2	32.7	32.7	-	29.7	32.70
1967	28.3	31.4	33.6	32.8	33.8	34.7	29.1	29.3	31.8	30.4	28.6	28.1	30.99
1968	29.4	26.8	30.9	30.4	34.1	34.1	31.6	31.2	32.5	32.1	29.3	30.0	31.03
1969	30.4	29.1	31.6	33.5	35.4	36.5	33.3	31.8	33.2	33.3	31.7	30.9	32.56
1970	30.0	32.1	30.6	34.1	36.2	36.5	33.7	30.7	32.2	32.3	30.3	28.8	32.29
1971	28.6	31.4	32.9	34.0	33.8	34.7	31.7	30.1	31.5	31.7	29.5	28.1	31.50
1972	30.6	28.6	33.6	34.4	34.4	34.2	32.9	32.2	32.8	32.2	31.1	30.1	32.26
1973	30.7	32.8	35.0	35.8	35.2	35.8	32.7	32.3	33.9	33.8	32.2	30.0	33.35
1974	31.1	32.4	31.7	35.0	34.6	35.3	33.0	33.2	33.7	35.1	31.6	30.9	33.13
1975	30.9	32.1	34.6	34.3	35.3	34.6	31.3	30.7	32.0	33.1	30.5	29.2	32.38
1976	30.7	33.1	34.8	34.6	34.4	37.2	32.8	30.8	33.2	33.3	30.5	31.0	33.03
1977	29.7	29.1	33.6	34.4	34.9	35.5	33.4	32.6	34.7	32.5	29.5	30.5	32.53
1978	30.9	31.3	33.3	36.5	36.7	37.1	31.4	31.9	32.9	32.2	31.2	29.2	32.88
1979	27.8	30.9	33.0	33.9	33.7	34.4	33.1	32.6	34.1	33.1	32.3	30.6	32.46
1980	30.1	33.2	35.1	34.2	35.6	37.2	31.9	32.3	34.2	33.9	32.4	30.5	33.38
1981	31.9	32.0	29.8	30.5	34.1	36.3	32.4	30.7	31.6	32.2	31.1	29.6	31.85
1982	30.2	31.0	33.4	32.4	32.9	36.0	33.4	30.1	32.7	30.1	29.9	29.3	31.78
1983	29.2	30.2	32.7	33.7	34.2	35.3	34.2	31.1	33.2	32.9	30.9	28.9	32.21
1984	29.3	30.7	34.7	35.8	34.7	34.2	33.8	33.8	32.9	33.1	31.9	29.7	32.88
1985	31.1	30.8	34.4	32.1	33.2	35.8	31.6	30.4	32.9	32.8	31.5	29.9	32.21
1986	29.9	32.2	32.8	33.5	35.5	34.4	32.0	32.8	33.6	32.9	31.9	29.9	32.62
1987	29.5	32.7	32.1	32.9	33.1	36.2	36.0	33.3	35.4	33.7	32.0	31.0	33.16
1988	30.0	32.8	34.5	34.8	36.4	36.9	31.8	31.8	32.7	32.4	30.9	30.0	32.92
1989	29.4	30.2	32.7	31.0	34.8	36.0	32.2	32.9	33.4	33.2	31.6	29.8	32.27
1990	30.7	29.8	30.7	32.6	36.4	37.4	32.8	32.8	33.7	32.2	32.1	30.1	32.61
1991	31.9	32.0	34.3	33.9	34.5	36.6	32.5	31.8	33.6	33.8	-	30.3	33.20
1992	30.2	29.5	33.0	34.8	35.5	36.4	32.6	31.1	32.2	32.1	31.7	31.3	32.53
1993	29.8	29.5	34.9	33.5	33.1	36.0	32.8	33.2	34.5	33.5	32.7	31.1	32.88
1994	31.2	32.0	35.3	36.1	35.8	35.9	31.4	31.3	32.0	32.3	30.2	29.5	32.75
1995	30.8	30.6	32.0	34.2	36.4	37.3	33.3	32.6	33.4	34.0	32.4	31.8	33.23
1996	30.8	33.5	33.9	34.3	33.5	34.2	32.4	32.6	33.5	33.2	31.2	30.5	32.80
1997	30.7	31.6	34.8	33.4	36.1	35.7	33.4	34.3	36.1	32.2	30.2	30.1	33.22
1998	31.4	32.9	33.9	36.8	36.7	37.9	34.4	32.1	34.5	32.9	31.9	31.0	33.87
1999	31.7	34.7	33.6	35.9	37.4	37.5	32.6	33.4	34.1	31.9	31.2	30.4	33.70
2000	32.2	33.1	35.2	35.8	37.3	38.1	33.4	32.5	34.4	32.5	31.9	30.8	33.90
2001	30.0	32.8	33.4	35.5	37.5	36.9	33.5	31.9	34.2	34.2	31.8	29.9	33.47
2002	30.4	33.3	34.1	35.5	37.8	37.8	36.5	33.4	34.8	34.1	32.7	30.2	34.22
2003	30.6	33.8	35.0	35.0	37.1	36.4	32.5	31.5	33.9	33.8	32.2	30.3	33.51
2004	31.9	31.7	33.7	33.4	37.1	36.9	33.7	33.6	34.9	33.2	32.3	31.2	33.63
2005	31.5	34.8	34.8	36.3	35.0	36.7	32.3	34.4	35.4	34.2	32.8	32.1	34.19
2006	34.3	32.3	35.0	36.1	38.5	37.7	33.9	32.1	34.5	34.1	32.4	30.2	34.26
2007	31.6	34.4	36.0	35.0	38.1	37.2	33.4	32.2	33.1	34.6	32.5	31.5	34.13
2008	33.3	31.9	35.4	39.0	36.6	36.8	34.6	32.4	35.5	34.7	30.6	31.0	34.32
2009	31.7	34.3	36.3	35.6	37.2	37.9	34.5	33.4	35.4	33.2	32.7	30.6	34.40
2010	30.9	31.8	32.0	35.0	36.8	36.3	32.8	33.0	33.0	34.1	32.4	30.2	33.17
Mean	30.6	31.7	33.6	34.4	35.5	36.2	32.9	32.1	33.6	33.0	31.4	30.2	32.9
St.Dv	1.2	1.7	1.5	1.7	1.5	1.1	1.2	1.2	1.1	1.0	1.0	0.9	0.8
Cv	0.04	0.05	0.05	0.05	0.04	0.03	0.04	0.04	0.03	0.03	0.03	0.03	0.03

Table A-3: Minimum Temperature, °c

Year	Jan	Feb	mar	Apr	may	Jun	July	Aug	Sep	Oct	Nov	Dec	Average
1966	12.3	17.1	15.9	19.1	17.8	21.3	20.7	19.3	15.1	16.2	-	8.7	16.7
1967	10.5	14.8	18.5	18.3	18.5	19.1	20.2	19.2	19.4	16.8	17.6	11.3	17.0
1968	10.4	18.2	17.1	18.9	18.1	20.7	19.6	18.5	18.4	15.2	14.8	13.4	16.9
1969	18.3	16.9	18.5	19.3	20.1	22.3	22.1	19.8	19.6	15.3	14.7	10.9	18.2
1970	16.7	16.2	18.6	18.8	19.8	22.9	20.9	19.8	18.4	16.2	10.2	10.7	17.4
1971	13.4	11.4	15.9	17.9	19.4	21.2	19.5	18.5	17.9	14.9	14.9	12.3	16.4
1972	14.1	16.4	16.3	19.3	17.2	19.7	21.0	19.6	18.5	16.0	15.6	15.0	17.4
1973	14.7	13.9	17.8	20.5	20.2	21.7	21.1	21.0	20.1	15.4	13.1	9.5	17.4
1974	13.4	16.3	18.5	15.9	18.6	21.0	20.2	20.3	19.3	15.8	11.1	11.9	16.9
1975	11.9	16.3	18.1	19.0	19.3	20.7	19.3	19.6	19.0	14.4	11.4	10.0	16.6
1976	12.3	17.1	17.9	18.6	19.7	21.4	20.1	18.6	18.4	15.3	14.1	13.6	17.3
1977	18.4	14.3	17.7	18.6	19.3	21.4	21.0	19.8	19.3	18.8	15.2	14.2	18.2
1978	12.9	16.0	18.7	18.5	20.4	21.7	19.6	19.1	18.5	16.5	13.7	14.1	17.5
1979	16.9	13.6	16.9	17.8	18.8	20.8	19.8	19.5	19.2	15.7	11.2	14.0	17.0
1980	14.1	16.8	19.8	19.4	19.2	22.7	20.6	19.6	19.4	16.5	14.7	11.3	17.8
1981	13.3	13.1	17.7	17.7	17.5	20.8	20.5	19.0	18.6	15.0	12.5	11.3	16.4
1982	16.6	18.1	17.3	18.2	18.2	20.4	20.3	19.1	18.4	15.1	15.6	15.4	17.7
1983	13.5	18.1	20.2	19.8	20.1	20.0	20.7	20.2	19.5	15.1	12.8	12.8	17.7
1984	11.1	11.2	15.9	18.0	19.3	20.6	19.4	20.1	18.3	13.9	15.5	14.4	16.5
1985	13.8	14.4	18.2	19.4	18.7	20.9	19.4	18.7	17.8	15.1	13.5	12.7	16.9
1986	10.8	18.2	17.1	19.8	19.4	21.9	19.8	19.4	18.6	15.7	14.0	14.6	17.4
1987	14.0	15.9	20.0	18.1	19.6	22.7	21.0	20.3	19.5	17.4	12.8	13.8	17.9
1988	16.8	18.8	17.4	19.5	18.7	21.9	21.1	19.8	19.4	16.5	10.6	12.7	17.8
1989	13.8	16.1	18.0	19.0	16.2	20.4	20.5	19.6	19.3	15.2	14.4	18.9	17.6
1990	14.9	18.8	17.8	17.5	19.4	22.1	19.7	19.9	19.7	15.1	14.8	12.5	17.7
1991	16.4	17.5	19.1	18.5	18.9	21.0	20.8	20.0	19.0	16.1	-	-	18.7
1992	15.9	17.9	19.8	20.1	19.2	20.8	20.1	19.7	18.1	16.0	14.9	16.4	18.2
1993	17.1	16.4	15.3	19.5	19.0	21.3	20.4	19.9	19.7	17.4	13.9	12.4	17.7
1994	11.9	14.8	19.8	20.4	19.6	21.6	19.6	19.7	18.3	14.7	14.4	12.6	17.3
1995	12.3	16.0	18.7	19.7	19.4	20.4	21.0	19.9	18.6	16.7	13.0	16.8	17.7
1996	17.5	14.9	19.7	19.3	19.2	20.9	20.1	19.9	19.2	14.8	13.7	11.8	17.6
1997	16.0	13.6	18.8	19.2	19.1	20.2	20.4	20.0	19.2	18.7	18.4	14.5	18.2
1998	18.3	19.6	20.0	20.3	20.9	22.6	21.6	20.2	19.8	17.6	12.2	10.0	18.6
1999	13.7	14.6	19.3	18.0	18.9	20.2	19.4	19.2	18.8	17.4	12.0	11.9	17.0
2000	13.1	12.3	15.9	18.8	20.1	21.9	19.9	19.8	19.9	17.0	14.8	13.5	17.3
2001	13.6	15.0	18.9	18.5	20.6	21.9	19.8	19.9	17.9	17.3	13.4	14.1	17.6
2002	15.9	15.0	19.5	19.4	20.3	22.4	21.2	19.9	18.8	17.0	14.1	18.0	18.5
2003	15.6	17.0	19.0	20.5	18.5	21.4	20.0	19.6	20.0	15.7	15.9	13.9	18.1
2004	17.8	15.0	15.6	19.0	17.7	21.5	19.8	20.1	19.4	16.3	15.0	15.4	17.7
2005	15.2	15.5	18.1	18.3	20.3	21.4	20.2	19.8	20.3	16.0	14.3	10.9	17.5
2006	15.9	18.6	20.0	18.5	18.6	21.9	20.0	20.1	19.5	17.6	14.9	15.7	18.4
2007	16.1	17.0	16.8	18.0	20.6	21.5	20.1	19.1	19.3	15.4	13.4	10.5	17.3
2008	13.3	30.0	11.8	18.5	19.9	21.1	19.8	19.4	20.1	15.3	13.3	10.5	17.8
2009	13.5	14.8	17.3	18.1	18.3	21.8	20.8	20.2	19.9	17.5	13.9	17.7	17.8
2010	15.2	18.4	18.0	20.2	21.4	22.3	20.8	20.3	19.4	17.6	14.9	15.7	18.7
Mean	14.5	16.3	17.9	18.9	19.2	21.3	20.3	19.7	19.0	16.1	13.9	13.2	17.6
St.Dv	2.2	2.9	1.6	0.9	1.0	0.8	0.6	0.5	0.9	1.1	1.7	2.4	0.6
Cv	0.15	0.18	0.09	0.05	0.05	0.04	0.03	0.03	0.05	0.07	0.12	0.18	0.03

Table A-4: Minimum Relative Humidity, %

Year	Jan	Feb	mar	Apr	may	Jun	July	Aug	Sep	Oct	Nov	Dec	Average
1966	21.0	30.0	22.0	19.0	16.0	17.0	21.0	24.0	26.0	29.0	-	-	22.5
1967	24.0	22.0	21.0	21.0	17.0	18.0	33.0	29.0	28.0	26.0	33.0	29.0	25.1
1968	27.0	48.0	32.0	39.0	25.0	23.0	27.0	32.0	34.0	29.0	32.0	28.0	31.3
1969	34.0	35.0	32.0	33.0	28.0	29.0	35.0	38.0	33.0	26.0	29.0	27.0	31.6
1970	42.0	34.0	41.0	32.0	27.0	22.0	22.0	31.0	26.0	23.0	21.0	25.0	28.8
1971	30.0	23.0	24.0	22.0	24.0	22.0	27.0	30.0	25.0	23.0	28.0	29.0	25.6
1972	25.0	36.0	22.0	25.0	25.0	27.0	31.0	33.0	30.0	27.0	30.0	35.0	28.8
1973	34.0	28.0	16.0	14.0	13.0	21.0	31.0	34.0	29.0	21.0	21.0	21.0	23.6
1974	31.0	31.0	34.0	21.0	31.0	28.0	31.0	33.0	30.0	24.0	22.0	27.0	28.6
1975	25.0	28.0	25.0	24.0	22.0	25.0	34.0	38.0	33.0	22.0	30.0	33.0	28.3
1976	34.0	36.0	29.0	31.0	33.0	29.0	36.0	40.0	34.0	29.0	34.0	32.0	33.1
1977	44.0	36.0	33.0	31.0	30.0	30.0	36.0	34.0	25.0	29.0	28.0	26.0	31.8
1978	22.0	30.0	29.0	23.0	26.0	27.0	38.0	34.0	29.0	27.0	29.0	35.0	29.1
1979	46.0	37.0	33.0	26.0	26.0	26.0	28.0	29.0	27.0	24.0	21.0	30.0	29.4
1980	34.0	27.0	26.0	25.0	18.0	21.0	29.0	31.0	28.0	23.0	26.0	27.0	26.3
1981	28.0	29.0	37.0	33.0	25.0	22.0	27.0	34.0	30.0	20.0	23.0	25.0	27.8
1982	34.0	31.0	26.0	26.0	26.0	21.0	24.0	35.0	26.0	26.0	29.0	33.0	28.1
1983	30.0	34.0	29.0	23.0	24.0	19.0	23.0	34.0	22.0	20.0	23.0	29.0	25.8
1984	24.0	19.0	16.0	20.0	21.0	18.0	18.0	18.0	20.0	17.0	27.0	25.0	20.3
1985	22.0	25.0	21.0	25.0	19.0	18.0	-	-	-	-	-	33.0	23.3
1986	30.0	33.0	31.0	32.0	27.0	31.0	32.0	32.0	28.0	26.0	31.0	38.0	30.9
1987	37.0	35.0	43.0	37.0	39.0	32.0	32.0	36.0	32.0	32.0	31.0	35.0	35.1
1988	41.0	36.0	24.0	26.0	21.0	22.0	33.0	33.0	30.0	25.0	24.0	28.0	28.6
1989	31.0	34.0	33.0	35.0	24.0	-	34.0	34.0	32.0	26.0	32.0	44.0	32.6
1990	35.0	50.0	42.0	33.0	24.0	26.0	34.0	33.0	28.0	24.0	31.0	28.0	32.3
1991	33.0	35.0	29.0	28.0	28.0	24.0	33.0	35.0	28.0	23.0		22.0	28.9
1992	38.0	39.0	32.0	26.0	25.0	23.0	-	-	-	-	-	-	30.5
1993	-	-	-	-	-	-	-	-	-	-	-	-	
1994	-	-	34.0	31.0	32.0	32.0	39.0	43.0	38.0	30.0	36.0	36.0	35.1
1995	37.0	37.0	37.0	33.0	30.0	29.0	38.0	34.0	31.0	31.0	31.0	33.0	33.4
1996	30.0	27.0	34.0	33.0	31.0	30.0	31.0	32.0	30.0	26.0	39.0	33.0	31.3
1997	44.0	36.0	39.0	40.0	34.0	31.0	32.0	-	-	34.0	39.0	34.0	36.3
1998	41.0	37.0	35.0	30.0	31.0	31.0	35.0	43.0	37.0	-	-	34.0	35.4
1999	40.0	-	-	-	-	-	-	-	-	-	-	34.0	37.0
2000	35.0	36.0	35.0	34.0	32.0	35.0	40.0	37.0	30.0	34.0	39.0	37.0	35.3
2001	45.0	37.0	37.0	32.0	29.0	34.0	36.0	42.0	35.0	38.0	38.0	39.1	36.8
2002	43.0	35.0	35.0	31.0	29.0	30.0	41.0	39.0	36.0	35.0	37.0	46.0	33.4
2003	37.0	36.0	35.0	35.0	32.0	34.0	40.0	44.0	35.0	31.0	34.0	34.0	35.6
2004	40.0	35.0	33.0	35.0	29.0	28.7	32.0	34.0	28.0	28.0	29.0	36.0	32.3
2005	33.0	28.0	29.0	29.0	28.0	30.0	35.0	29.0	28.0	28.0	30.0	28.0	29.6
2006	33.0	32.0	26.0	30.0	26.0	26.0	30.0	35.0	29.0	28.0	27.0	33.0	29.6
2007	32.0	26.0	26.0	26.0	25.0	25.0	27.0	32.0	30.0	23.0	24.0	24.0	26.7
2008	27.0	30.0	26.0	26.0	25.0	24.0	30.0	39.0	55.0	53.0	51.0	39.0	35.4
2009	42.0	41.0	43.0	43.0	36.0	46.0	47.0	41.0	27.0	28.0	30.0	34.0	38.2
2010	32.6	34.1	30.2	26.9	25.2	19.5	28.1	30.8	27.9				
Mean	33.7	33.1	30.6	29.0	26.5	26.3	32.0	34.2	30.2	27.4	30.2	31.7	30.5
St.Dv	6.8	6.0	6.7	6.0	5.3	5.8	5.8	5.1	5.5	6.1	6.3	5.5	4.3
Cv	0.20	0.18	0.22	0.21	0.20	0.22	0.18	0.15	0.18	0.22	0.21	0.17	0.14

Table A-5: Maximum Relative Humidity, %

Year	Jan	Feb	mar	Apr	may	Jun	July	Aug	Sep	Oct	Nov	Dec	Average
1966	21.0	30.0	22.0	19.0	16.0	17.0	21.0	24.0	26.0	29.0	-	-	22.5
1967	24.0	22.0	21.0	21.0	17.0	18.0	33.0	29.0	28.0	26.0	33.0	29.0	25.1
1968	27.0	48.0	32.0	39.0	25.0	23.0	27.0	32.0	34.0	29.0	32.0	28.0	31.3
1969	34.0	35.0	32.0	33.0	28.0	29.0	35.0	38.0	33.0	26.0	29.0	27.0	31.6
1970	42.0	34.0	41.0	32.0	27.0	22.0	22.0	31.0	26.0	23.0	21.0	25.0	28.8
1971	30.0	23.0	24.0	22.0	24.0	22.0	27.0	30.0	25.0	23.0	28.0	29.0	25.6
1972	25.0	36.0	22.0	25.0	25.0	27.0	31.0	33.0	30.0	27.0	30.0	35.0	28.8
1973	34.0	28.0	16.0	14.0	13.0	21.0	31.0	34.0	29.0	21.0	21.0	21.0	23.6
1974	31.0	31.0	34.0	21.0	31.0	28.0	31.0	33.0	30.0	24.0	22.0	27.0	28.6
1975	25.0	28.0	25.0	24.0	22.0	25.0	34.0	38.0	33.0	22.0	30.0	33.0	28.3
1976	34.0	36.0	29.0	31.0	33.0	29.0	36.0	40.0	34.0	29.0	34.0	32.0	33.1
1977	44.0	36.0	33.0	31.0	30.0	30.0	36.0	34.0	25.0	29.0	28.0	26.0	31.8
1978	22.0	30.0	29.0	23.0	26.0	27.0	38.0	34.0	29.0	27.0	29.0	35.0	29.1
1979	46.0	37.0	33.0	26.0	26.0	26.0	28.0	29.0	27.0	24.0	21.0	30.0	29.4
1980	34.0	27.0	26.0	25.0	18.0	21.0	29.0	31.0	28.0	23.0	26.0	27.0	26.3
1981	28.0	29.0	37.0	33.0	25.0	22.0	27.0	34.0	30.0	20.0	23.0	25.0	27.8
1982	34.0	31.0	26.0	26.0	26.0	21.0	24.0	35.0	26.0	26.0	29.0	33.0	28.1
1983	30.0	34.0	29.0	23.0	24.0	19.0	23.0	34.0	22.0	20.0	23.0	29.0	25.8
1984	24.0	19.0	16.0	20.0	21.0	18.0	18.0	18.0	20.0	17.0	27.0	25.0	20.3
1985	22.0	25.0	21.0	25.0	19.0	18.0	-	-	-	-	-	33.0	23.3
1986	30.0	33.0	31.0	32.0	27.0	31.0	32.0	32.0	28.0	26.0	31.0	38.0	30.9
1987	37.0	35.0	43.0	37.0	39.0	32.0	32.0	36.0	32.0	32.0	31.0	35.0	35.1
1988	41.0	36.0	24.0	26.0	21.0	22.0	33.0	33.0	30.0	25.0	24.0	28.0	28.6
1989	31.0	34.0	33.0	35.0	24.0	-	34.0	34.0	32.0	26.0	32.0	44.0	32.6
1990	35.0	50.0	42.0	33.0	24.0	26.0	34.0	33.0	28.0	24.0	31.0	28.0	32.3
1991	33.0	35.0	29.0	28.0	28.0	24.0	33.0	35.0	28.0	23.0		22.0	28.9
1992	38.0	39.0	32.0	26.0	25.0	23.0	-	-	-	-	-	-	30.5
1993	-	-	-	-	-	-	-	-	-	-	-	-	
1994	-	-	34.0	31.0	32.0	32.0	39.0	43.0	38.0	30.0	36.0	36.0	35.1
1995	37.0	37.0	37.0	33.0	30.0	29.0	38.0	34.0	31.0	31.0	31.0	33.0	33.4
1996	30.0	27.0	34.0	33.0	31.0	30.0	31.0	32.0	30.0	26.0	39.0	33.0	31.3
1997	44.0	36.0	39.0	40.0	34.0	31.0	32.0	-	-	34.0	39.0	34.0	36.3
1998	41.0	37.0	35.0	30.0	31.0	31.0	35.0	43.0	37.0	-	-	34.0	35.4
1999	40.0	-	-	-	-	-	-	-	-	-	-	34.0	37.0
2000	35.0	36.0	35.0	34.0	32.0	35.0	40.0	37.0	30.0	34.0	39.0	37.0	35.3
2001	45.0	37.0	37.0	32.0	29.0	34.0	36.0	42.0	35.0	38.0	38.0	39.1	36.8
2002	43.0	35.0	35.0	31.0	29.0	30.0	41.0	39.0	36.0	35.0	37.0	46.0	33.4
2003	37.0	36.0	35.0	35.0	32.0	34.0	40.0	44.0	35.0	31.0	34.0	34.0	35.6
2004	40.0	35.0	33.0	35.0	29.0	28.7	32.0	34.0	28.0	28.0	29.0	36.0	32.3
2005	33.0	28.0	29.0	29.0	28.0	30.0	35.0	29.0	28.0	28.0	30.0	28.0	29.6
2006	33.0	32.0	26.0	30.0	26.0	26.0	30.0	35.0	29.0	28.0	27.0	33.0	29.6
2007	32.0	26.0	26.0	26.0	25.0	25.0	27.0	32.0	30.0	23.0	24.0	24.0	26.7
2008	27.0	30.0	26.0	26.0	25.0	24.0	30.0	39.0	55.0	53.0	51.0	39.0	35.4
2009	42.0	41.0	43.0	43.0	36.0	46.0	47.0	41.0	27.0	28.0	30.0	34.0	38.2
2010	32.6	34.1	30.2	26.9	25.2	19.5	28.1	30.8	27.9				
Mean	33.7	33.1	30.6	29.0	26.5	26.3	32.0	34.2	30.2	27.4	30.2	31.7	30.5
St.Dv	6.8	6.0	6.7	6.0	5.3	5.8	5.8	5.1	5.5	6.1	6.3	5.5	4.3
Cv	0.20	0.18	0.22	0.21	0.20	0.22	0.18	0.15	0.18	0.22	0.21	0.17	0.14

Table A-6: Sunshine hours

Year	Jan	Feb	mar	Apr	may	Jun	July	Aug	Sep	Oct	Nov	Dec	Average
1966	10.3	10.3	10.6	9.7	9.5	10.3	9.5	9.5	9.0	9.0	9.6	10.7	9.8
1967	10.6	11.3	9.7	9.6	9.4	8.9	5.6	7.0	7.0	8.3	7.6	9.4	8.7
1968	10.9	4.7	10.0	6.9	9.5	7.5	7.8	8.0	8.5	8.9	7.7	9.3	8.3
1969	7.9	7.0	8.0	8.2	9.0	8.9	7.0	7.7	6.7	9.0	9.0	9.7	8.2
1970	6.5	8.9	7.2	8.7	9.0	9.2	7.4	6.4	8.3	9.0	9.8	9.0	8.3
1971	7.9	8.8	8.7	8.4	7.9	8.4	7.5	7.5	8.0	7.7	8.2	53.0	11.8
1972	8.8	7.0	8.9	7.8	9.3	8.3	7.3	7.4	8.3	9.0	9.5	9.4	8.4
1973	9.5	10.0	10.0	9.3	8.3	8.7	7.6	7.8	7.2	9.0	10.3	9.8	8.9
1974	9.6	9.9	6.3	10.2	7.9	9.3	7.9	7.3	7.8	9.0	10.3	9.9	8.8
1975	9.6	9.3	9.0	8.0	8.5	8.0	7.3	6.4	7.2	9.2	9.7	9.9	8.5
1976	9.8	9.7	9.0	9.0	8.3	8.8	6.9	6.9	7.8	8.3	8.7	9.4	8.5
1977	6.9	8.3	8.3	7.6	8.7	8.7	7.4	8.0	7.7	7.5	9.7	9.6	8.2
1978	9.6	7.9	8.6	8.8	9.2	8.7	6.8	7.9	8.0	8.4	9.2	8.7	8.5
1979	6.5	8.8	8.6	9.0	8.9	8.9	8.5	8.3	8.2	8.0	9.7	9.4	8.6
1980	9.7	-	9.0	8.6	9.6	9.0	7.0	8.7	7.0	7.8	9.0	9.8	8.7
1981	9.9	8.8	6.2	7.5	9.8	9.8	7.2	7.9	7.3	8.8	9.7	9.7	8.6
1982	8.8	7.7	7.4	8.9	8.8	8.8	7.2	6.5	8.0	7.8	7.9	8.2	8.0
1983	8.2	7.8	8.9	7.6	8.0	9.0	8.5	5.7	7.8	8.7	9.6	8.9	8.2
1984	9.2	9.4	8.9	10.0	7.9	8.6	8.9	8.8	7.7	9.5	9.4	8.7	8.9
1985	9.4	8.7	7.0	7.0	8.6	9.4	7.6	7.6	7.9	8.0	9.3	9.5	8.3
1986	9.7	8.6	8.9	7.7	9.3	5.4	7.9	7.8	8.6	8.7	9.6	9.0	8.4
1987	9.3	8.7	7.5	8.6	8.2	8.3	9.0	7.0	8.7	8.8	9.7	9.6	8.6
1988	8.4	9.0	9.0	7.9	9.8	9.0	5.3	7.0	7.3	8.3	9.8	9.4	8.3
1989	8.5	8.2	8.0	6.9	9.7	9.3	6.5	8.2	7.7	8.0	9.0	7.3	8.1
1990	9.0	6.0	7.7	8.0	9.8	9.8	7.3	8.3	7.9	8.9	9.3	9.3	8.4
1991	8.8	8.4	7.7	8.0	8.0	8.6	7.2	7.0	8.0	8.9	9.2	8.5	8.2
1992	6.2	6.5	9.2	8.5	9.4	8.9	7.3	6.0	7.6	8.3	8.2	8.5	7.9
1993	7.0	6.7	10.0	8.8									8.1
1994													
1995													
1996													
1997	8.4	10.0	8.7	7.2	8.0	9.3	7.7	8.0	7.9	6.9	7.9	8.9	8.2
1998	6.6	7.8	7.8	9.3	9.2	9.6	7.0	7.2	7.5	8.2	9.7	10.0	8.3
1999	9.4	10.2	7.4	9.5	8.4	8.8	6.8	7.9	7.7	6.9			8.3
2000									7.0				7.0
2001			7.5										7.5
2002													
2003				7.3	8.9	8.2	6.0	5.7	7.0	8.6	9.0	8.9	7.7
2004	8.5	8.8	8.0										8.4
2005					8.0	8.7	6.4	8.6	7.2	9.0	9.0	10.0	8.4
2006	9.3	8.7	8.0	7.8	8.6	8.5	7.2	5.7	7.4	7.5	9.4	9.4	8.1
2007	8.0	9.3	8.8	8.2	9.0	7.3	8.5	6.7	7.0	8.4	8.6	9.9	8.3
2008	8.6	9.0	9.7	8.3	8.0	8.7	7.0	7.6	7.7	8.5	8.4	9.7	8.4
2009	9.8	9.5	9.5	9.5	9.5	9.5	8.0	8.0	8.4	7.2	9.5	6.9	8.8
2010	9.0	6.6	7.8	8.4	7.6	8.4	6.7	6.7	7.0				7.6
Mean	8.8	8.5	8.5	8.4	8.8	8.7	7.4	7.4	7.7	8.4	9.1	10.5	8.4
St.Dv	1.2	1.4	1.0	0.9	0.7	0.8	0.9	0.9	0.5	0.7	0.7	7.4	0.7
Cv	0.1	0.2	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.7	0.1

Table A-7: Wind speed, m/s

Year	Jan	Feb	mar	Apr	may	Jun	July	Aug	Sep	Oct	Nov	Dec	Average
1973										0.66	0.71	0.70	0.69
1974	0.74	0.90	0.72	0.72	0.73	0.92	1.09	0.92	0.67	0.63	0.66	0.69	0.78
1975	0.68	0.72	0.65	0.56	0.65	0.93	1.11	0.80	0.57	0.50	0.59	0.59	0.70
1976	0.60	0.35	0.60	0.56	0.51	1.19	1.01	0.85	0.57	0.53	0.47	0.55	0.65
1977	0.60	0.55	0.65	0.51	0.45	0.77	1.03	0.76	0.60	0.51	0.45	0.48	0.61
1978	0.53	0.58	0.52	0.60	0.58	0.91	0.88	0.60	0.47	0.47	0.56	0.53	0.60
1979	0.52	0.50	0.53	0.53	0.56	0.76	0.85	0.58	0.43	0.45	0.51	0.49	0.56
1980	0.53	0.57	0.63	0.51	0.55	0.80	0.80	0.63	0.44	0.39	0.47	0.46	0.56
1981	0.44	0.56	0.44	0.35	0.44	0.64	0.78	0.56	0.38	0.33	0.46	0.44	0.48
1982	0.45	0.40	0.42	0.33	0.33	0.53	0.77	0.40	0.19	0.24	0.29	0.37	0.39
1983	0.45	0.37	0.33	0.33	0.29	0.44	0.65	0.49	0.30	0.26	0.31	0.35	0.38
1984	0.37	0.42	0.38	0.49	0.35	0.59	0.56	0.36	0.28	0.36	0.37	0.37	0.41
1985	0.39	0.48	0.47	0.28	0.25	0.44	0.54	0.57	0.26	0.29	0.27	0.31	0.38
1986	0.26	0.33	0.33	0.42	0.41	0.77	0.62	0.53	0.35	0.35	0.39	0.42	0.43
1987	0.42	0.40	0.38	0.34	0.42	0.63	0.60	0.49	0.32	0.35	0.35	0.37	0.42
1988	0.40	0.35	0.44	0.38	0.44	0.71	0.79	0.44	0.30	0.28	0.38	0.42	0.45
1989	0.44	0.52	0.44	0.33	0.21	0.59	0.77	0.58	0.39	0.37	0.45	0.47	0.46
1990	0.51	0.37	0.38	0.33	0.45	0.72	0.72	0.64	0.42	0.34	0.35	0.36	0.46
1991	0.40	0.37	0.40	0.31	0.33	0.51	0.72	0.53	0.33	0.29	0.28	0.33	0.40
1992	0.37	0.39	0.42	0.33	0.25	0.47	0.63	0.48	0.28	0.19	0.23	0.30	0.36
1993	0.38	0.33	0.22	0.32	0.26	0.56	0.73	0.56	0.42	0.28	0.31	0.31	0.39
1994	0.33	0.43	0.44	0.37	0.35	0.67	0.56	0.39	0.22	0.25	0.25	0.28	0.38
1995	0.27	0.26	0.20	0.11	0.19	0.26	0.41	0.33	0.13	0.24	0.18	0.20	0.23
1996	0.19	0.19	0.15	0.12	0.08	0.19	0.13	0.08	0.10	0.10	0.10	0.09	0.13
1997	0.08	0.20	0.13	0.13	0.13	0.16	0.17	0.14	0.08	0.10	0.11	0.17	0.13
1998	0.13	0.13	0.12	0.14	0.15	0.20	0.19	0.15	0.10	0.08	0.12	0.19	0.14
1999	0.15	0.18	0.12	0.19	0.15	0.15	0.25	0.15	0.13	0.10	0.14	0.23	0.16
2000	0.26	0.30	0.32	0.27	0.25	0.44	0.44	0.33	0.18	0.15	0.23	0.37	0.29
2001	0.28	0.31	0.19	0.25	0.36	0.54	0.40	0.34	0.23	0.27	0.35	0.39	0.33
2002	0.36	0.37	0.28	0.33	0.28	0.45	0.45	0.41	0.22	0.24	0.28	0.30	0.33
2003	0.28	0.30	0.36	0.28	0.28	0.50	0.59	0.35	0.29	0.21	0.30	0.25	0.33
2004	0.24	0.30	0.24	0.20	0.28	0.48	0.56	0.41	0.25	0.22	0.29	0.29	0.31
2005	0.27	0.30	0.33	0.33	0.28	0.45	0.45	0.33	0.33	0.22	0.23	0.24	0.31
2006	0.31	0.29	0.19	0.22	0.27	0.51	0.41	0.35	0.24	0.19	0.17	0.16	0.28
2007	0.22	0.16	0.19	0.13	0.21	0.43	0.39	0.27	0.26	0.11	0.17	0.16	0.22
2008	0.16	0.28	0.18	0.23	0.14	0.32	0.42	0.27	0.16	0.08	0.08	0.12	0.20
2009	0.09	0.09	0.21	0.19	0.30	0.83	0.46	0.64	0.59	0.63	0.69	0.67	0.45
2010	0.24	0.19	0.19	0.18	0.19	0.26	0.26	0.20	0.13	0.00	0.00	0.00	0.15
Mean	0.36	0.37	0.36	0.33	0.33	0.56	0.60	0.46	0.31	0.30	0.33	0.35	0.39
St.Dv	0.16	0.16	0.17	0.15	0.15	0.24	0.25	0.20	0.15	0.16	0.17	0.17	0.17
Cv	0.44	0.44	0.46	0.45	0.46	0.43	0.42	0.44	0.49	0.55	0.52	0.47	0.42

Table A-8: Pan Evaporation, mm

Year	Jan	Feb	mar	Apr	may	Jun	July	Aug	Sep	Oct	Nov	Dec	Average
1966	88	6.5	7.3	8.4	10.3	9.9	8.5	7.4	6.5	7.9	-	6.2	8.0
1967	6.1	6.3	6.8	7.3	6.9	7.3	5.7	5.6	6.0	6.4	5.8	5.9	6.3
1968	6.7	5.3	6.4	5.9	8.1	10.4	10.4	8.1	8.7	9.7	7.0	6.8	7.8
1969	7.1	6.1	7.1	7.8	8.8	10.2	9.2	7.6	7.4	8.7	8.5	7.7	8.0
1970	6.2	8.2	6.9	9.2	10.0	10.8	8.8	6.4	7.5	8.6	8.2	7.8	8.2
1971	7.8	9.4	9.3	8.4	9.6	10.8	9.7	7.8	7.1	8.0	7.4	6.1	8.5
1972	7.2	6.1	8.6	8.2	8.8	8.7	7.5	7.0	7.1	7.9	7.2	6.6	7.6
1973	7.0	8.4	9.5	9.9	8.4	9.4	8.3	7.0	6.9	7.8	7.9	6.9	8.1
1974	6.7	7.9	6.4	8.7	7.4	8.0	7.5	6.9	6.3	7.5	7.2	7.1	7.3
1975	7.3	6.7	7.6	6.3	7.4	7.5	6.7	5.7	5.5	6.9	7.0	6.1	6.7
1976	6.6	7.4	7.5	7.2	6.5	8.7	6.5	5.6	6.2	6.8	5.9	6.0	6.7
1977	5.0	6.0	7.6	6.6	6.3	7.6	7.1	6.6	6.6	6.6	5.3	5.6	6.4
1978	6.4	5.6	6.6	8.1	7.9	8.9	6.4	6.4	6.0	6.9	6.6	6.2	6.8
1979	4.7	6.4	6.7	7.5	6.3	7.9	7.5	6.2	6.3	6.5	6.9	6.2	6.6
1980	6.2	7.5	8.5	7.1	7.9	8.2	6.2	6.4	7.4	6.6	6.6	6.5	7.1
1981	6.3	7.0	5.8	5.7	7.2	7.7	6.7	5.8	5.4	6.0	6.4	6.7	6.4
1982	6.0	5.5	6.9	6.5	7.3	7.5	7.0	5.5	6.4	6.2	5.3	5.4	6.3
1983	6.5	6.0	6.4	6.1	7.0	7.9	7.4	5.6	5.8	6.7	6.4	5.8	6.5
1984	6.2	7.7	8.2	8.9	7.9	7.7	7.5	7.6	6.9	8.5	8.4	7.0	7.7
1985	7.4	8.7	9.3	6.7	7.9	7.6	6.4	6.2	6.1	6.6	7.0	6.5	7.2
1986	7.0	6.7	6.8	7.7	8.6	8.0	6.5	6.6	7.1	7.5	7.8	7.0	7.3
1987	6.8	7.4	6.8	8.1	6.9	8.5	8.3	7.0	7.6	8.0	7.9	6.3	7.5
1988	6.0	7.0	7.7	6.9	8.6	8.5	6.7	6.6	5.6	6.0	7.1	6.2	6.9
1989	6.6	6.6	7.3	5.3	7.5	7.8	7.1	5.5	5.4	7.2	7.2	5.8	6.6
1990	7.2	5.1	6.0	6.9	8.0	8.6	6.5	7.1	6.2	6.8	6.7	6.5	6.8
1991	7.1	6.1	7.2	7.0	6.8	7.7	7.0	6.4	5.8	6.5	5.7	5.7	6.6
1992	5.4	5.5	7.9	7.7	8.0	7.9	6.2	5.6	5.3	5.8	5.8	5.3	6.4
1993	4.9	5.5	8.1	6.7	6.3	6.9	6.7	6.2	5.9	6.2	6.1	5.8	6.3
1994	6.0	6.9	7.1	7.9	7.2	7.7	5.5	6.1	5.8	6.4	5.5	6.0	6.5
1995	6.5	6.1	5.7	6.5	8.4	7.7	6.9	5.7	5.9	7.4	6.3	6.1	6.6
1996	5.9	6.6	6.4	6.5	5.5	6.7	5.2	6.0	6.5	6.5	5.8	6.3	6.2
1997	5.6	7.2	7.3	6.0	7.8	6.9	6.1	6.3	6.1	5.3	4.7	5.4	6.2
1998	4.7	6.1	6.0	6.9	7.6	7.7	6.3	5.5	5.7	6.0	6.2	5.7	6.2
1999	6.1	7.4	5.5	7.7	7.5	7.0	6.0	6.2	6.6	5.7	5.6	6.0	6.4
2000	6.0	6.3	7.2	6.6	7.3	8.2	6.0	5.9	5.9	6.3	6.2	5.8	6.4
2001	5.7	7.0	6.1	7.0	7.9	7.8	6.1	5.2	5.9	6.7	6.9	6.1	6.5
2002	5.9	7.4	6.8	7.1	7.8	8.0	7.4	6.1	6.3	6.7	6.8	5.1	6.8
2003	5.7	6.6	7.3	6.5	7.9	7.4	5.5	4.7	5.8	6.8	6.4	5.2	6.3
2004	5.3	6.4	6.4	5.9	7.4	6.1	5.8	6.3	5.9	5.8	6.1	6.3	6.1
2005	6.2	7.7	7.9	8.3	7.0	8.5	6.2	6.8	7.2	7.4	7.2	7.1	7.3
2006	6.9	7.1	7	7.3	7.7	8.3	5.8	4.9	5.7	5.8	5.9	4.8	6.4
2007	5.3	6.2	7.2	6.3	8.4	7.6	5.5	4.5	4.8	5.9	5.9	5.6	6.1
2008	5.9	6.1	7.1	6.7	6.3	7.0	6.3	5.2	5.9	6	5	5.4	6.1
2009	4.8	6.2	7.0	6.2	7.8	9.8	7.5	6.9	8.5	7.5	7.2	5.8	7.1
2010	7.0	6.3	6.9	7.6	7.2	8.4	6.5	6.1	39.8	5.8	5.9	4.8	9.4
Mean	6.3	6.7	7.2	7.2	7.7	8.2	6.9	6.2	7.1	6.9	6.6	6.1	6.9
St.Dv	0.9	0.9	0.9	1.0	1.0	1.1	1.1	0.8	5.1	0.9	0.9	0.7	0.7
Cv	0.14	0.14	0.13	0.14	0.12	0.13	0.16	0.13	0.71	0.14	0.14	0.11	0.11

Table A-9: Awash flow data in MCM

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
1982	48.0	37.6	35.1	38.5	54.7	37.7	78.4	173.8	96.6	78.9	37.1	40.6	757.0
1983	35.2	35.7	45.0	44.2	132.2	79.6	138.0	206.3	408.8	155.2	105.2	131.6	1517.0
1984	100.7	40.1	38.6	27.9	59.1	62.2	75.1	139.0	127.2	54.9	56.1	56.6	837.4
1985	39.6	35.3	34.0	57.2	85.7	44.2	223.1	456.7	717.0	163.9	43.0	51.7	1951.4
1986	73.1	70.9	90.0	74.1	75.3	103.5	183.7	145.0	86.6	49.5	57.4	62.4	1071.6
1987	49.6	42.8	86.8	71.1	50.0	33.5	26.9	93.4	57.9	16.4	14.5	13.4	556.3
1988	24.3	16.3	14.5	26.4	4.5	4.8	39.9	101.8	183.3	63.5	13.5	14.1	506.9
1989	17.2	24.9	20.3	57.3	34.7	43.0	122.6	144.5	319.1	76.5	52.9	61.9	974.8
1990	42.2	111.7	69.8	72.1	48.5	178.1	169.1	169.5	375.5	82.0	47.2	41.7	1407.5
1991	35.5	50.9	85.2	69.1	68.7	39.9	99.2	158.5	374.9	97.2	99.1	85.5	1263.6
1992	132.4	132.7	32.8	50.6	56.2	35.3	56.7	118.2	292.8	85.1	46.5	57.4	1096.6
1993	83.7	76.6	48.6	59.5	122.2	100.1	131.7	361.7	543.6	165.4	67.7	71.9	1832.9
1994	65.6	52.8	53.0	55.1	58.6	53.7	150.5	123.7	129.8	70.5	44.1	45.4	902.7
1995	37.8	51.9	103.2	59.8	44.1	77.9	160.7	188.0	130.1	42.9	41.5	40.8	978.7
1996	50.1	39.5	82.0	48.0	51.1	55.6	92.2	401.5	369.5	62.8	26.9	58.2	1337.7
1997	68.6	47.1	70.4	80.5	59.6	59.9	114.7	132.0	47.1	45.4	30.7	18.9	774.8
1998	25.3	25.3	41.7	29.9	24.4	25.7	64.8	382.1	314.9	162.1	53.3	63.3	1212.6
1999	63.9	56.7	71.7	60.3	73.1	99.8	200.1	324.3	187.0	183.2	86.2	73.6	1480.0
2000	98.7	74.0	71.5	70.0	133.2	109.6	105.4	199.3	161.0	161.3	94.4	67.0	1345.6
2001	76.0	72.0	72.9	61.2	131.6	67.1	97.9	167.8	175.0	70.7	80.0	83.2	1155.4
2002	87.6	64.7	77.2	71.9	66.4	71.9	96.7	158.5	95.9	52.1	22.9	26.5	892.2
2003	44.4	29.0	37.7	43.7	30.1	84.4	96.6	112.9	98.2	77.6	52.3	22.7	729.4
2004	69.0	54.6	71.9	98.3	23.1	36.6	71.4	94.5	211.2	58.2	40.6	40.6	870.1
2005	56.9	50.2	66.6	56.6	66.1	46.9	94.8	132.7	145.1	51.6	44.4	49.4	861.3
2006	52.0	66.5	89.4	130.3	106.1	107.2	64.4	261.1	422.7	66.3	53.6	59.4	1478.9
2007	53.1	65.3	83.3	106.5	69.7	78.3	120.4	310.6	428.6	68.8	60.4	82.1	1526.9
2008	76.3	64.7	60.8	69.5	81.8	54.7	134.1	202.1	321.9	98.6	117.2	71.7	1353.2
2009	98.7	74.0	71.5	70.0	133.2	109.6	105.4	199.3	161.0	161.3	94.4	67.0	1345.6
2010	76.0	72.0	72.9	61.2	131.6	67.1	97.9	167.8	175.0	70.7	80.0	83.2	1155.4
Mean	59.5	55.2	61.3	62.6	67.1	66.3	111.5	202.2	252.6	87.4	55.1	55.2	1136.0

Table A-10: Awash River flow (2002-2014)in m^3/s

Month	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec
Mean	34.73	29.32	28.94	29.21	46.37	40.98	29.26	46.33	44.80	42.15	25.56	19.73
Flow (MCM)	93.01	73.45	77.52	75.71	124.20	106.22	78.36	124.08	116.13	112.90	66.25	52.86
Maximum	42.57	35.26	38.23	39.57	59.35	56.36	42.50	54.31	48.85	46.35	32.59	23.11
Minimum	28.63	25.43	23.85	21.50	30.65	20.93	22.33	40.98	41.94	39.19	19.58	18.02
Mean	17.49	20.80	17.63	18.84	18.56	23.84	34.68	42.41	84.08	44.55	38.40	46.82
Flow (MCM)	46.84	50.32	47.21	48.83	49.70	61.80	92.90	113.59	217.94	119.31	99.53	125.39
Maximum	27.54	73.74	28.96	39.22	26.16	32.68	75.69	65.98	103.37	55.19	69.24	55.72
Minimum	14.51	15.30	13.99	14.51	15.09	17.97	23.59	30.82	38.59	37.00	33.95	40.03
Mean	40.34	37.93	36.77	40.60	36.75	41.84	44.93	41.05	32.83	30.22	25.58	26.02
Flow (MCM)	108.03	95.03	98.48	105.24	98.43	108.46	120.34	109.95	85.09	80.95	66.29	69.69
Maximum	48.16	52.32	39.49	52.33	45.65	65.92	63.42	79.36	38.65	39.44	28.70	29.41
Minimum	31.98	33.35	32.00	36.39	33.05	32.39	38.02	32.54	29.24	24.26	23.91	23.00
Mean	31.93	39.02	43.86	38.90	31.72	36.94	34.54	36.83	67.71	31.51	31.00	29.37
Flow (MCM)	85.53	94.40	117.48	100.83	84.95	95.74	92.52	98.64	175.50	84.41	80.36	78.66
Maximum	54.99	73.81	87.42	65.74	40.20	81.69	61.96	61.45	106.78	61.22	38.32	44.52
Minimum	22.17	26.73	29.08	27.38	25.47	23.71	25.43	28.70	29.80	24.23	23.07	22.02
Mean	27.60	33.97	35.79	47.41	46.27	48.18	32.35	98.03	157.07	38.79	28.25	29.61
Flow (MCM)	73.92	82.18	95.86	122.88	123.93	124.87	86.63	262.55	407.12	103.89	73.22	79.30
Maximum	35.18	58.22	49.35	61.05	74.75	72.11	41.62	225.61	223.52	83.43	34.65	34.78
Minimum	23.40	26.08	27.83	32.00	29.42	33.39	26.97	35.17	75.89	29.25	24.19	25.20
Mean	25.52	28.49	32.39	32.32	35.87	40.06	57.92	123.02	188.38	42.57	37.12	36.81
Flow (MCM)	68.34	68.92	86.75	83.77	96.07	103.82	155.13	329.50	488.27	114.01	96.20	98.58
Maximum	31.69	34.17	37.09	41.00	43.44	56.29	79.92	187.29	262.99	81.64	45.49	48.91
Minimum	17.91	24.42	26.04	28.25	29.42	32.88	36.04	56.87	86.26	27.47	29.79	26.71
Mean	36.17	37.14	37.32	44.93	46.13	30.89	36.62	59.61	139.05	54.65	34.79	38.72

Flow (MCM)	96.88	93.05	99.95	116.46	123.55	80.07	98.09	159.66	360.42	146.37	90.18	103.70
Maximum	41.73	39.67	45.91	56.42	68.10	41.88	67.41	91.18	268.66	110.17	45.16	46.99
Minimum	30.55	29.54	33.91	36.39	34.66	23.35	19.87	30.60	66.28	28.63	28.21	34.35
Mean	45.91	35.94	36.89	39.65	43.09	35.87	31.11	29.04	30.07	37.42	34.88	44.68
Flow (MCM)	122.97	86.94	98.80	102.78	115.40	92.97	83.31	77.78	77.93	100.22	90.41	119.68
Maximum	64.11	40.16	39.71	58.09	60.55	48.52	44.24	56.34	40.15	63.11	49.07	54.11
Minimum	28.92	32.28	33.10	33.14	33.91	26.83	25.87	23.45	18.39	32.11	32.11	21.21
Mean	36.67	30.71	31.16	32.82	38.23	36.36	50.53	84.07	129.27	43.50	37.13	39.73
Flow (MCM)	98.22	74.30	83.47	85.07	102.39	94.23	135.33	225.17	335.06	116.52	96.24	106.41
Maximum	52.71	39.44	39.81	39.81	66.22	47.03	88.56	155.69	194.39	86.19	48.66	50.73
Minimum	28.99	26.85	23.21	29.71	27.25	27.92	33.09	55.46	63.71	19.04	31.01	17.47
Mean	35.50	34.41	36.69	38.14	48.17	39.52	35.20	33.13	37.95	35.50	37.47	40.14
Flow (MCM)	95.08	83.24	98.28	98.87	129.01	102.42	94.28	88.72	98.37	95.09	97.12	107.51
Maximum	42.05	38.11	45.16	42.15	60.94	58.47	42.28	46.12	53.58	37.93	39.35	44.15
Minimum	31.73	31.43	31.94	32.41	41.15	25.46	27.72	27.59	30.67	32.54	33.74	35.78
Mean	38.11	34.41	32.94	33.76	34.12	38.96	53.07	99.10	148.80	42.37	39.82	52.83
Flow (MCM)	102.08	86.21	88.22	87.49	91.39	100.99	142.13	265.43	385.69	113.47	103.21	141.49
Maximum	49.35	40.17	37.40	38.91	38.55	47.15	70.21	228.40	204.27	83.44	46.40	62.73
Minimum	33.39	31.73	30.08	28.96	31.51	31.43	37.71	39.49	84.79	31.77	32.84	37.14
Mean	49.90	43.65	41.00	55.24	56.42	54.67	68.68	63.12	86.62	61.57	42.01	-
Flow (MCM)	133.65	105.60	109.82	143.17	151.12	141.72	183.95	169.07	224.51	164.91	108.89	-
Maximum	59.75	52.14	61.65	66.94	68.77	69.40	100.45	85.95	136.39	74.27	55.49	-
Minimum	36.46	28.21	31.09	43.61	47.35	36.84	48.79	48.05	43.06	41.45	55.49	-