Addis Ababa University
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Electrical and Computer Engineering Department

ASSESSMENT OF MICRO HYDRO POWER POTENTIAL OF SELECTED ETHIOPIAN RIVERS- A CASE STUDY IN THE NORTH-WEST PART OF THE COUNTRY

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Nomenclatures and Abbreviations

α	radial rim width
A	stream cross sectional area
AC	alternating current
ASTER	Advanced Spaceborne Thermal Emission and Reflection Radiometer
B	number of blades
c	stream mapping constant
cfs	cubic feet per second
cms	cubic meter per second
C	nozzle constant
C_v	velocity correction factor
D	runner diameter
DA	Drainage Area
D_b	depth of settling basin
DC	Direct Current
D_e	penstock economic diameter
DEM	Digital Elevation Model
D_t	distance traveled
EELPA	Ethiopian Electric Light & Power Authority
EEPCo	Ethiopian Electric Power Corporation
h	head above crest of spill way
H	rated head
H_g	Gross head
HOMER	Hybrid Optimization Model for Electric Renewables
IG	Induction Generator
IM	Induction Motor
K	empirical coefficient
l	spill way length
lps	liters per second

L	runner length
L_b	length of settling basin
MHP	Micro Hydro Power
MoWE	Ministry of Water &Energy
n	Manning‟s coefficient
N	turbine running speed
p	wetted perimeter of canal
P	rated plant capacity
P_d	penstock design pressure
P_g	pressure gauge reading
P_s	static pressure
psi	pound per square inch
Q	flow rate
r	hydraulic radius
R_c	radius of blade curvature
s	space between the blades
S	Canal slope
S_A	surge allowance factor
SG	Synchronous Generator
S_0	jet thickness
t	time taken
t_{min}	minimum thickness of penstock
V	average water velocity in canal
$\overline{V}$	average stream velocity
V_c	volume of container
V_s	surface velocity of stream water
W	stream width
W_b	width of settling basin

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Abstract

Ethiopia is a country situated in the horn of Africa with a total population of 80 million of which 85% is living in rural areas where access to modern electricity is difficult, with electrification status negligible and being < 1% [2, 3]. These communities are settled farther away from the national grid and are sparsely populated which makes extending the national grid to them uneconomical because of the high cost of transmission and the very low load factor. However, improving the life style of the population as well as the economy of a country will be difficult without energy especially modern energy systems. For their day to day life these rural people are using kerosene, fire wood, cow dung and other traditional fuels which have the problem of high and day-by-day increasing price and negative environmental effect because of deforestation and soil erosion. However, if these communities are supplied with modern electricity from renewable energy resources, their life will be improved and the environment will be protected from damage and this is the basic idea for this thesis.

In this thesis, assessment of micro hydropower (MHP) potential of selected rivers is done. The selected rivers are, “Jedeb”, “Temcha” and “Gilgel_Abay” for the communities living in the villages called “Belem‟na”, “Muketin” &“Yewubesh”, and “Killaj” respectively. Some of the data for this work are gathered from Ministry of Water & Energy of Ethiopia (MoWE) and the rest is collected/ measured from field work. Some part of the data from MoWE is treated with ArcGIS software. Local load estimation for each of the villages is done with lighting, cocking and radio &TV loads considered as home loads and pumping, church and school lighting considered as general community loads and all these loads are scheduled appropriately. HOMER is used for designing and simulating the general system of each village.

The simulation result shows that the generated electricity is more than enough to supply the local loads considered with an excess energy of at least 50% of the generated energy and this shows that the systems are tolerable for this amount of demand increase in the future.

Key words: Assessment, hydropower potential, micro hydropower, HOMER, small scale power generation, renewable energy.

CHAPTER ONE
INTRODUCTION

1.1. Background

Energy is one of the most fundamental elements of our universe. Especially for developing countries like Ethiopia, the significance of energy is vital. On-demand energy in the form of electricity and modern fuels is the lifeblood of modern civilization and is critical factor for economic development and employment. It amplifies human efforts enables humans to produce more, travel farther and faster, communicate more broadly and quickly, and live at a higher standard of living than is possible through human efforts alone. Hence it can be said that like the other basic needs for human beings, energy is the basic inevitability for survival.

Energy sources such as current oil, coal, and natural gas that provide about 86% of the world‟s energy can be said as *easy energy* [1]. The benefits of easy energy are all around us, it powered the rise of modern civilization by increasing human productivity and, especially, freeing a large percentage of the population from pre-modern agriculture systems and primitive energy recovery systems from biomass which are labor intensive tasks. As a consequence of the on-going expansion of modern civilization, the worldwide demand for easy energy is increasing more but degrading the resources of nature‟s gifts easy energy and if some action is not taken, energy crisis will be the fate of the world (even if it is happening now especially in developing countries). On the other hand, for every tone of combustion of fossil fuels, the world is introducing in the atmosphere a few tons of carbon dioxide, an environmental polluting gas. The above two reasons necessitates for a search of new energy sources that satisfy both the increase in load demand and environmental issues.

Even if Ethiopia has a huge potential of renewable energy (hydro, wind, solar, geothermal), it is suffering from energy crisis throughout characterized by frequent blackouts and interruption. This is because of the unbalance between the increasing energy demand versus energy production in the country. Even due to the limited energy production, there are places, especially remote areas, where energy is not available/accessed and the people around are living in darkness. In addition to this limited production, the country was concentrating in large energy production systems. But it is difficult to use most of the potentials that the country had and to make all people access to this modern energy without small energy production systems. And it is the aim of this thesis work to assess the potentials and viability of a micro hydropower (MHP) system of the selected sites.

The selected sites are:

- Jedeb River for the community living in the village called “Belem“na” at (latitude, longitude) of (10.458694°, 37.59676°).
- Temcha River for the community living in the villages called “Muketin” and “Yewubesh” at (latitude, longitude) of (10.541315°, 37.500629°).
- Gilgel Abay River for the community living in the village called “Killaj” at (latitude, longitude) of (11.444442°, 37.026243°).

1.2. Water Power

Water is a natural resource which has been used to generate power in one form or another for centuries. In its simplest form hydro power was used to grind grain, provide shaft power for textile plants, sawmills and other manufacturing operations. In Ethiopia, the application of water power for grinding is common especially in rural areas, the direct way of utilization of water power. Another way of harnessing the power of water is by direct conversion to electricity called hydro electric power generation. In this respect we can have large hydro, medium hydro, small, mini, micro hydro and even pico hydro.

MHP is defined as the generation of electricity in the range 1 kW to 100 kW. Till 1990s, EELPA, (now EEPCo), used to install and operate a number of standalone, mini and micro hydropower systems to supply rural towns. Most of these hydropower systems were built in 1950s and 1960s. Today, only one of them is operational [2].

1.3. Statement of the problem

Most of the people of Ethiopia (around 85 % of the population) lives in rural areas where energy access is almost negligible, <1% [2, 3]. A possible reason is that these areas are either farther away from the national grid or the people living there are sparsely populated. Extending the national grid to these areas is not up to the economic capacity of the country because of the high cost of transmission and the very low load factor in these areas. But electrification of the rural communities is very essential especially to ensure the socio economic development of the community and hence of the country. To satisfy their energy needs, these people are using kerosene which is becoming difficult to afford because of the high and day-by-day increasing price of kerosene; and fire wood, cow dung and other traditional biomass resources which are causing deforestation and soil degradation.

These two contradictory factors, need of power and un affordability &effect of traditional power systems, necessitates for a new method of electrification of the rural areas. This new method can be use of potential of renewable energies such as hydro, wind, solar or a combination of the three.

1.4. Objective of the study

To accomplish this thesis work, the following are the general and specific objectives:

General objective: To analyze the viability of a MHP for rural electrification at selected sites.

The Specific Objectives are:

- Assess micro-hydro power resources and collect a preliminary data for micro hydro power generation.
- Analysis of the data collected
- Local load estimation for each site and designing and simulating a micro hydropower system using HOMER software.

1.5. Methodology

The methodologies to accomplish this thesis work will be as follows:

- Literature review: Published materials about mini and micro hydropower systems and assessment of hydropower potential of rivers will be studied.

- Data collection: The data required for the potential assessment of rivers will be collected from the respective organization. For example, flow rate of the selected rivers is collected from the Ministry of Water & Energy of Ethiopia (MoWE) and head of the river and appropriate position for system installation is done from field work.
- Data analysis: The potential of each river will be analyzed from the collected data.
- Design: An overall design of the power system will be carried out using HOMER.
- Conclusion and recommendations will be made based on simulation studies.

1.6. Organization of the thesis

This thesis work consists of five chapters.

The first chapter discuses the general overview of energy, its advantage and what easy energy is and its relevance both in the past and today. The selected sites for this thesis, the power of water and the way of utilizing it, statement of the problem, the objective of the study and methodology are also presented in this chapter.

Chapter two is a literature review part. In this chapter different literatures related to energy issues both in developing and developed countries, the energy potential as a whole in Ethiopia specially the hydropower resources and more about MHP are briefly discussed. Moreover, the methods used in the determination of head and flow rate of flowing rivers are covered in this chapter.

The methodologies used while collecting data and the approaches applied while analyzing these data as well as while designing the system both in the software for simulation and to determine the type and size of components used in each system are covered in chapter three.

Chapter four is all about the results & discussion of the results obtained based on the work of Chapter three.

Conclusions, recommendations and suggestions for future work are presented in chapter five.

CHAPTER TWO
LITERATURE REVIEW

Most of the developing countries are suffering from what many call the energy crisis, characterized by depletion of locally available energy resources and dependence on imported fuel. In addition, the energy crisis is exacerbating the food crisis by increasing the rate of deforestation and thereby causing degradation of farmlands. Furthermore, dependence on imported fuel is weakening the capacity of the concerned countries to buy food whenever the need arises. The fate of our country Ethiopia is not different from this issue. Ethiopia is a country with a population of 80 million and a total area of 1.1 million sq.km. Of these 80 million people 85% is living in rural area where it is hard to get modern energy, electricity [4]. Therefore, these people are forced to use traditional energy sources for their demand. However, still there are people living in urban areas but relying on traditional fuels. This may be because that these people are not able to afford for the modern fuel and/or are unaware of the advantage that the modern fuels have over the traditional ones. With this in mind, Ethiopia is a country that relies extremely on traditional fuel, third from Africa, led by Chad and Eritrea. Moreover, more than 99.9% of the rural energy consumption of the country is mate by traditional fuels. The table below shows some examples of traditional fuel types and how much they contribute for the rural people.

Table2.1. Contribution of traditional fuels for rural people in Ethiopia [5]

Fuel type	Percentage share
Wood	81.8%
Dung	9.4
Crop residue	8.4
Charcoal	The rest

On the whole, over 95% of the domestic energy needs are met from biomass thereby contributing to deforestation, loss of soil nutrients and organic matter [3, 6, 7].

Nevertheless, there is a huge renewable energy resource potential in Ethiopia, which, if utilized, could minimize the present energy crisis prevailing in the country and enhance the process of rural electrification. The total exploitable renewable energy that can be derived annually from primary solar radiation, wind, forest biomass, hydropower, animal waste, crop residue and human waste is about 1,959x10^3 Tcal per year (EEA, 2002, quoted by B. Tamrat).

Table2.2. An over view of renewable energy resources in ethiopia [8]

No	Energy Resources	Energy in 10^3 Tcal per year			
		Potential	% share	Exploitbale	%share
1	Primary solar radiation	1,953,550	99.7	1,954	73.08
2	Wind	4,779	0.24	239	8.94
3	Forest Biomass	800	0.005	240	8.97
4	Hydropower	552.1	0.03	138.00	5.16
5	Animal waste	111.28	0.01	33.73	1.26
6	Crop Residue	81.36	0.0004	40.63	1.52
7	Human waste	28.18	0.00014	28.18	1.05
Total		1,959,901.93	100.00	2,673.54	100

2.1. Hydropower resource of Ethiopia

Because of the many rivers flowing in the country, Ethiopia is often described as the water tower of eastern Africa [9]. It was estimated that the country has a hydropower potential of 40,000 MW (installable potential). Not more than 1390 MW (2009) has been utilized so far thus leaving more than 75% of the potential unused [10]. In the past years, the Ethiopian Electric Power Corporation was concentrating on large hydro systems and they are meant to feed the national grid. Still it is going on power production with these systems having almost no room for small, mini and micro scale hydropower systems, which is more suited to rural electrification. Of the above exploitable potential in the country, about 1500 MW to 3000 MW would be suitable for small scale power generation including Pico and Micro hydropower. Of this, about 1000 projects of micro hydropower with a typical capacity of 100 kW are revealed [2]. At present, there are only about ten small-scale hydroelectric plants (0.25-1 MW capacity) in the entire country. The table below shows the regional distribution of micro hydropower potential of Ethiopia.

Table2.3. Summary of technical MHP potential in Ethiopia per region [8]

Region	Approximate MHP potential (technical) MW
Oromia	35
Amhara	33
Benshangul Gumuz	12
Gambella	2
SNNP	18

2.2. General Description about Hydro Power Generation

Water had power by both its motion (kinetic energy) and by its position (potential energy). To make this energy usable, prime movers such as water wheels are required. Earlier in the history of energy development and use, water wheels provide power by direct connection or with pulley and gear systems to drive various machines, such as grist mills and textile mills. Moreover, since ancient times, water wheels have been used for lifting water from a lower to a higher elevation in irrigation systems.

In this fast developing world however the way of consumption of water power is changed from its direct way to indirect way, a more efficient and easily usable electrical energy. In the case of hydropower electricity production, the prime movers are water wheels or turbines. These prime movers convert the power available in the water to rotational/mechanical energy of the shaft of the generator. Then the generator acting as an energy converting unit converts the rotational/mechanical energy to electrical energy. This is the principle of hydropower engineering, the technology involved in converting the pressure energy and kinetic energy of water into electrical energy [11].

2.3. Types of Hydro Power Schemes

Hydroelectric plants are classified commonly by their hydraulic characteristics, that is, with respect to the water flowing through the turbines that runs the generators. Depending on this, there are four basic types of hydro power generation:

Impoundment: In large hydropower systems the construction of large dams is necessary to store water and to provide sufficient head for the turbine. In addition of maintaining a sufficient head for the turbine, the dam/reservoir had the advantage that the stored water is used during dry seasons. And also these water storage schemes enable the power station to generate at times of peak power demand, and then allow the water level to rise again during off peak time. Schemes with large dams are better suited to larger, gently graded rivers.

Run-of-river type: The first hydro facilities were known as Run of River schemes. These schemes do not include any significant water storage, instead use the natural stream flow of the river and therefore make use of whatever water is flowing in the river. Generally, these plants would be feasible only on such streams which have a minimum dry weather flow of such magnitude which makes it possible to generate electricity throughout the year. However because the flow may vary throughout the year, during low flow seasons when the flow may be insufficient, run of river schemes are unable to generate power if they are not provided with some regulatory means. Hence typical run of river schemes involve either a low level diversion weir (a small dam) or a stream bed intake, and are usually located on swift flowing streams [12]. The low level diversion weir has the advantage that it raises the water level in the river sufficiently to enable an intake structure to be located on the side of the river, and to take care of the variation of flow in the river. During off-peak hours of electricity demand, as in the night, some of the units may be closed and the water conserved in the storage space, which is again released during peak hours for power generation.

Diversion and Canal type: The water is diverted from the natural channel into a canal or a long penstock, thus hanging the flow of the water in the stream for a considerable distance

Pumped Storage Type: Pumped storage plants utilize a reversible pumping turbine to store hydro energy during off-peak electricity hours by pumping water from a lower reservoir to an upper reservoir. This stored energy is then used to generate electricity during peak hours, when electricity is costly to produce, by distributing water from the upper to the lower reservoir.

2.4. Micro-Hydro Power Generation

Micro-hydro schemes are smaller in size and they refer to systems with capacity ranging from 1 kW just enough to provide domestic lighting to a group of houses through a battery charging to 100 kW which can be used for small factories and to supply an independent local mini-grid but do not necessarily supply electricity to the national grid [2, 8]. These small units have been used for many years, especially for their mechanical power, but recent increases in the value of electrical energy and incentive programs have made the construction and development of micro-hydro power plants much more attractive. For villages very far from the national grid where connection to it may not be economical and for which a constant stream flow is available, micro hydropower systems are the best options for electrification [13].

Except their power rating, micro hydropower systems are not different from large or small hydro systems. For example the principles of operation, types of material units, and the mathematical equations used in selection and design of system components are essentially the same as conventional/large hydro systems [7].

2.4.1. Micro hydropower plants in Ethiopia

Even if Ethiopia is concentrating on large hydropower systems, it is not ignoring smaller scale hydropower systems. Table 2.4 on the next page shows mini and micro hydropower plants which were installed long ago in Ethiopia;

2.5. Components of the Micro Hydro Power System

The components for a MHP system can be grouped in to civil work components and electromechanical components, as shown in Figure 2.1.below. These components are presented in detail in the following subsections.

Table2.4. Small hydro power plants in Ethiopia [8]

Name, Location	Head (m)	Type of scheme	Installed capacity (kW)	Commissioning year	Current status
Yadot, Bale Zone	23	ROR	350	1991	Operational
Welega, Weliso town	16	ROR	162	1965	Not operational
Sotosomere, Jimma	30	ROR	147	1954	Not operational
Hulka, Ambo town	40	ROR	150	1954	Not operational
Deneba,Buno bedele	14	ROR	123	1967	Not operational
Gelenmite, Denbi Dollo	42	ROR	195	1966	Not operational
Chemoga, Debre Markos	55	ROR	195	1962	Not operational
Debre Berhan		ROR	130	1955	Not operational
Jibo, Harar Zone		ROR	420		Not operational
Total capacity		ROR	1872		
Operational		ROR	350		
Not operational		ROR	1522		

Figure2.1. Typical layout of micro hydropower system [13]

2.5.1. Civil Works Components

Weir and intake: In the run-of-the river schemes, on which most of small and micro hydropower systems are based, a low diversion structure is built on the streambed prior to the intake to divert/channel the required amount of flow to the intake for power generation whilst the rest of the excess water continues to overflow it [12, 14]. This structure, commonly known by the name weir, is not constructed to store water rather to increase the level of water so that the water can inter to the intake structure in a reliable and controllable way. Weirs can be constructed permanently or temporarily using traditional water management techniques. If the terrain in the vicinity of the site is relatively flat, to get the required head for power generation, the water may be conveyed by using pressure pipes or penstocks from a long distance. But this may be costly and constructing weirs to get the required head may be an economical option.

The intake of a hydro scheme is a structure designed to permit and control the required amount of water flow to a water way without producing a negative impact on the local environmental and with minimum head loss.

Minimizing the amount of debris and sediment carried by the incoming water is one of the major functions of the intake and in performing this action trash racks are often placed at the entrance of the intake. Skimmers are sometimes used to prevent floating debris from entering the intake.

Flow control action of an intake structure is performed by control gates integrated with it. Sometimes due to failure of these control gates, the water flow may not be controlled appropriately in which case spill ways in the intake are built as back up to the control gates and to spill out the excess water.

Points to be considered while designing the intake:

- The desired flow must be diverted to the channel,

- During peak flow conditions the intake should be able to divert only the design flow with the excess flow passing the intake and weir without causing damage to them.
- Almost no maintenance and repairing requirement,
- The trash rack at the start of the intake should prevent large quantities of loose material from entering and damaging the channel.
- Some smaller materials may not be blocked by the trash rack hence during their way to the channel, they may get sedimentted. Therefore the intake must have the possibility to remove these piled up sediment.

Canals and Channels: These are components of a hydropower scheme used to convey water a relatively larger distance from the stream to the inlet of the penstock, with minimum of loss of head and at minimum cost.

Settling Basin: Depending on soil type and geographical feature of the area, a flowing river can usually carry a suspension of small particles. If these particles are not removed before they enter to the penstock pipe, they will cause damage and rapid wear to turbine runners. To remove these materials, the water flow must be slowed down in a settling basin so that the silt particles will settle on the basin floor. The deposit formed is then periodically flushed away. If they are situated at the inlet of the intake pipe, settling basins are used to prevent any sediment from the incoming water from settling in the intake or power conduit. The absence of settling basins in sites where they must be constructed will allow a large accumulation of sediments there by reducing the flow available for power generation overflowing the intake or canal thereby requiring a huge effort to clean [15, 16].

Forebay Tank: The forebay is a basin located just before the entrance to the penstock and which forms the connection between the channel and the penstock. This structure can serve as a final settling basin to allow the last particles, water borne debris which either passed through the intake or were added in the canal, to settle down before entering to the penstock and to the turbine. To remove the settled particles, gates/valves are incorporated to drain the forebay [14].

Depending on its size it can also serve as a reservoir to store water to cope with water demand created by a sudden increase in loading on the turbine.

Penstock: The penstock is the pipe between the forebay and the turbine and which conveys water under pressure to the turbine and, depending on factors such as the nature of the ground itself, the penstock material, the ambient temperatures and the environmental requirements; they can be installed over or under the ground [16, 17]. The penstock often constitutes a major expense in the total micro-hydro budget, as much as 40% is not uncommon in high head installations, and it is therefore worthwhile optimizing the design [13]. The trade-off is between head loss and capital cost. When the pipe diameter increases, the friction head lose in the pipe decreases dramatically. Conversely, pipe costs increase steeply with diameter. Therefore while designing the penstock a compromise between cost and performance must be taken in to consideration. The table below shows some materials used for penstock manufacturing and some of their characteristics.

Table2.5. Different penstock materials and their characteristics [16]

Material Type	Young‟s modulus of elasticity E (n/m^2)E9	Coefficient of linear expansion α ($mm/^oc$)E6	Ultimate tensile strength (N/m^2)E6	n^*
Welded steel	206	12	400	0.012
Polyethylene	0.55	140	5	0.009
PVC	2.75	57	13	0.009
Asbestos cement	n/a	8.1	n/a	0.011
Cast iron	78.5	10	140	0.014
Ductile iron	16.7	11	340	0.013

**-Manning's coefficient of the material*

2.5.2. Electro Mechanical Components

The principal electro mechanical components of a micro hydro plant are the turbine and generator.

2.5.2.1. Turbines

A hydro turbine is a rotating machine that converts the potential energy of the water to mechanical energy. Hydro turbines can be of either impulse or reaction types the difference being the manner of head/energy conversion [12].

In "impulse turbine" the potential energy of water is converted in to kinetic energy in the nozzle before reaching the turbine. The high velocity flowing jets from the nozzle strikes the buckets of the turbine there by transferring the kinetic energy of water in the jet to mechanical/rotational energy of the turbine using the principle of "Newton‟s Second Law". Turbines that fall under this category are *Pelton, Turgo* and *cross flow* turbines [11, 13, 15].

The "reaction turbine" develops power from the combined action of pressure energy and kinetic energy of the water. The runner is completely submerged in the fluid and has nozzles that discharge the working fluid attached to the rotor of the turbine. Using the principle of "Newton‟s Third Law", the acceleration of the fluids leaving the nozzle produces a reaction force on the pipes, causing the rotor to move in the opposite direction to that of the fluid. Turbines that fall under this category are *Kaplane, Propeller,* and *Francis* turbines [18].

In both of these turbine types, the rotary action of the water turbine drives an electrical generator that produces electrical energy or could drive other rotating machinery to use the mechanical energy directly.

Pelton Turbine: The Pelton turbine consists of three basic components: a stationary inlet nozzle, a runner and a casing. The runner consists of a set of specially shaped buckets mounted on a periphery of a circular rotating disc. The buckets are shaped in a manner to divide the flow in half and turn its relative velocity vector nearly 180°. i.e. The buckets are split into two halves so that the central area does not act as a dead spot incapable of deflecting water away from the oncoming jet so that the return jet will not interfere with the following bucket for the oncoming jet. The jet from the nozzle strikes the buckets and imparts momentum there by turning the turbine. The cutaway on the lower lip allows the following bucket to move further before cutting off the jet propelling the bucket ahead of it and also permits a smoother entrance of the bucket into the jet. Pelton turbines are not used at lower heads because their rotational speed becomes very slow and the runner required is very large and unwieldy. To operate at a turbine efficiency of 90% the water flow must operate between 70% and 80% of the maximum flow [13].

Figure2.2. Pelton turbines [18]

Turgo Turbine: The Turgo turbine is similar in design to and like a Pelton turbine it is an impulse turbine, but unlike the Pelton turbine it was designed to have a higher specific speed and its bucket are shaped differently and the jet of water strikes the plane of its runner at an angle of 20^o. The Turgo turbine can operate under a head in the range of 30 to 300 meter. In this case, the jets are aimed to strike the plane of the runner on one side and exit on the other. Therefore the flow rate is not limited by the discharged fluid interfering with the incoming jet (as is the case with Pelton turbines). As a consequence, a Turgo turbine can have a smaller diameter runner than a Pelton for equivalent power. The higher runner speed of the Turgo, due to its smaller diameter compared to other types, make direct coupling of turbine and generator more likely. The Turgo is efficient over a wide range of speeds and like a Pelton turbine it can be mounted either horizontally or vertically.

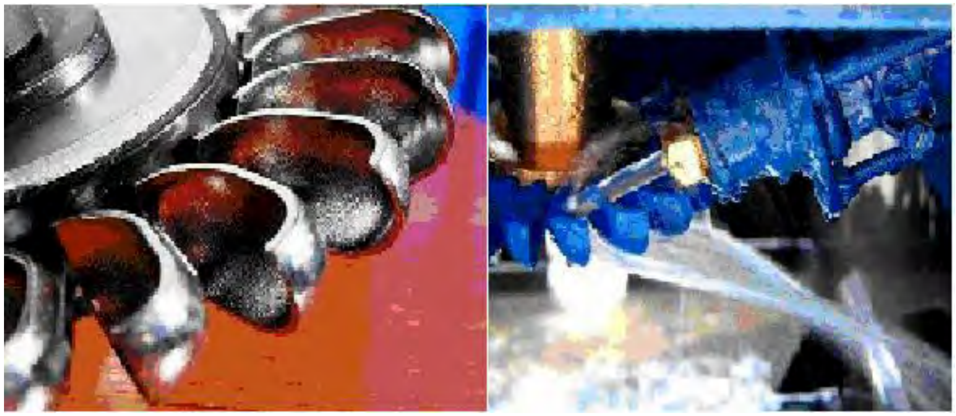

Figure2.3. Turgo turbine

Cross flow Turbine: Cross flow turbines, an impulse type turbines, are also known as Banki-Michell in remembrance of its inventors and Ossberger after a company which has been making it for more than 50 years [17]. These types of turbines consist of two parts, a nozzle and a turbine runner. The runner is built up of two parallel circular disks joined together at the rim with a series of curved blades. The nozzle whose cross sectional area is rectangle discharges the jet the full width of the wheel and enters the wheel at an angle of 16^{o} to the tangent of the periphery of the wheel for maximum efficiency. The water strikes the blades on the rim of the wheel, flows over the blade, leaving it, passing through the empty space between the inner rims, enters the blade on the inner side of the rim, and discharges at the outer rim striking the blades again impacting some of the rest energy before exit. A cross flow turbine has its runner shaft horizontal to the ground in all cases (unlike Pelton and Turgo turbines). One advantage of cross flow turbines is that they can be easily manufactured even in developing countries using locally available materials. The other advantage of cross flow turbines is that during low flow conditions, the water can be channeled through either two thirds or one third of the runner, thereby sustaining relatively high turbine efficiency.

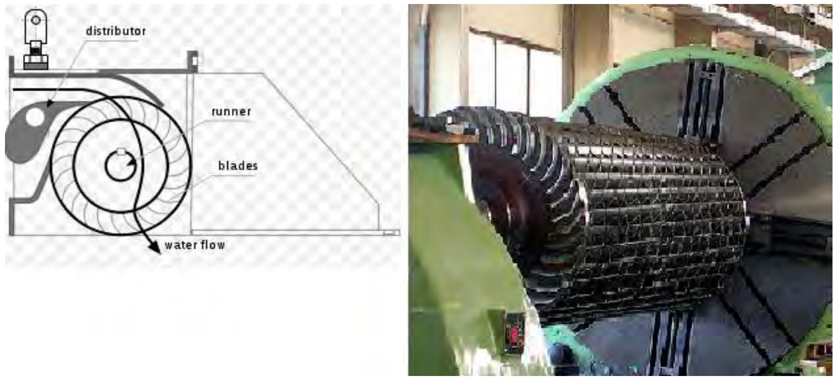

Figure2.4. Cross flow turbine [17]

Kaplan and Propeller Turbines: Kaplan and propeller turbines are axial-flow reaction turbines; generally used for low heads from 2 to 40 m. The Kaplan turbine has adjustable runner blades (enables them to work efficiently over a range of flow rates) and depending on whether the guide- vanes are adjustable or not, they can be described as "double-regulated" or "single-regulated" respectively. Fixed runner blade Kaplan turbines are called propeller turbines. They are used when both flow and head remain practically constant, which is a characteristic that makes them unusual in small hydropower schemes [20]. Propeller turbines are designed to work in a fully submerged condition and their design is optimized for a particular flow rate and efficiencies drop of rapidly if the flow rate falls below the design rating. Kaplan turbines have a high specific speed which means that direct coupling to the generator is possible but only at higher heads and lower flows [17]. This means that a speed increase will be necessary on most applications on which direct coupling is not possible.

Figure 2.5. Kaplan runner [20]

Francis Turbines: Francis turbines are radial flow reaction turbines, with fixed runner blades and adjustable guide vanes, used for medium heads. Francis turbines can be set in an open flume or attached to a penstock. The water enters the turbine by the spiral case that is designed to keep its tangential velocity constant along the consecutive sections and mobile guide vanes control the discharge going into the runner and adapt the inlet angle of the flow to the runner blades angles. The runner of a Francis turbine is composed of buckets of complex curves. These turbines work within a narrow range of flow variation from designed one. i.e. When the turbine"s flow rate is reduced below 85% of the maximum flow, the efficiency of the turbine falls away.

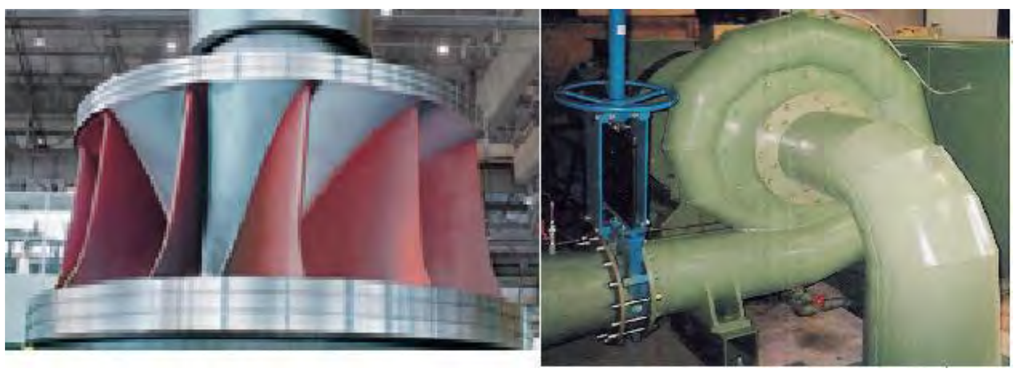

Figure2.6.Francis turbine [21]

2.5.2.2. Generators

Electrical generators are electrical equipments that produce either alternating current or direct current. The basic principle in physics that makes a generator produce electricity is that when a conductor is moved across a magnetic field, a voltage pressure is developed that forces electrons to move from atom to atom[22]. i.e. either or both the magnetic field and the conductor should move so that a net effect of relative motion between the two should be present for electricity to be generated.

AC generators produce electricity which is varying sinusoidally from positive peak to negative peak with constant amplitude and period while DC generators produce electricity whose current flow is in single direction and with non varying amplitude. There are two types of AC generators suitable for use in a micro hydro electricity supply scheme. These are synchronous generators (or „alternators") and induction generators (in which induction motors (IMs) used as a generator). These generators are constructed in the same fashion as a motor having the stationary stator which contains coils that produce the magnetic field.

A motor will develop torque on its shaft when connected to electrical power from an outside source, while the same machine will generate power when torque is applied to the motor shaft from an external source. It is this application that is of interest to the micro hydro power development.

Synchronous Generators (SG): In AC SGs, the magnetic field is produced by passing direct current through a second, smaller coil of wire called the field winding. The coil in which the electricity is actually produced is called the armature. The field current source may be an external battery or a belt or a direct driven DC generator. It could also be a small rotating AC generator with rectifier.

These generators operate at a speed directly related to the frequency. However, when the micro-hydro turbine is connected to a SG, which is directly connected to the electric grid, a speed variation is not possible. If speed variation is not possible, system efficiency is reduced because the generator cannot adapt to a partial load.

The main advantage of SGs in MHP systems is that it can operate in a "stand alone" situation, where a power developer is looking for energy that is independent of a power grid. One disadvantage of SGs is that they are costly than the induction generators (IGs). However their main disadvantage is that it requires a special synchronizing equipment to tie it in to power grid. Because of this, when the MHP system under construction is to be connected to the grid, IGs are chosen instead of SGs. However, IGs have efficiencies 2 to 4% lower than the efficiency of SGs over the entire operating range [23].

Induction Generators (IG): IGs have the same construction as IMs. These motors have their stator windings, which are the source of voltage, connected to the electrical system that receives the power and their rotor short circuited. The machine can"t function as a generator until a revolving magnetic field has been produced in the machine, and the current that produces the rotating magnetic field must therefore be supplied to the stator winding from an external source. For this reason an IG of this type must be operated in parallel with an existing power system. However in the absence of grid connection it is possible to operate IGs using capacitors designed for that particular generator.

One advantage of using IGs for MHP systems is because the machine used is a readily available motor and hence less expensive, and contains fewer parts. The other advantage of IGs is that they can withstand 200% runway speeds without harm, and has no brush or other parts which require maintenance. A disadvantage of the IG is its inherent requirement for reactive power. When induction motors act as a generator in MHP systems, they will require 0.3 to 0.6 magnetizing KVARs per horsepower for operation. Another disadvantage of using IGs in MHP systems connected to a grid is that a poor power factor of the power grid which may happen in the grid during certain times of the year, such as summer irrigation because of high motor loads, can affect the economics of the power from an IG. As the reactive power production of IGs due to poor power factor increase, the real power production is reduced, as a result of which the revenue production of the MHP station is reduced [22].

2.5.2.3. The Control System

Like large hydropower systems, due to continuous variation in load demand and the presence of deferent types of loads with different characteristics, the voltage and frequency of micro hydropower systems may not be constant. Therefore, similar to that of large power systems, the voltages and frequency of micro hydropower systems should be kept at scheduled values. To keep these parameters at the scheduled values, the micro hydropower systems should be controlled. In a power system, usually, voltage and frequency are controlled separately. Voltage is maintained by control of reactive power of the synchronous generator with the voltage regulator built-in in most of these synchronous generators while frequency is maintained by balancing generation and demand, i.e. by balancing generation and demand of active power. The balance between generation and demand is achieved in two different ways: by controlling either the mechanical input power or the load connected to the synchronous generator. The following are deferent types of active power control systems [2].

Automatic generation control: Automatic generation control is achieved through different types of speed governors. The most common ones are mechanical-hydraulic, electro-hydraulic, mechanical and servo motor governors.

Automatic load control: Generally, electrical loads change randomly as a result of which frequency of the system is changed randomly. It is possible to compensate the change in the electrical load, consequently the change in frequency, using ballast loads.

Flow control: Traditionally, flow control mechanisms similar to that of larger hydropower systems have been used to control the frequency of micro and mini hydropower systems. Nevertheless, over the last two decades, because of their complexity, slow response and costs, hydraulic or mechanical speed governors have been replaced by electronic load controllers (ELCs).

2.6. Turbine Selection

The type and size of a turbine for a hydropower system is different for one project than the other depending on many factors. Therefore using these factors as criteria, the type and size of turbine for a particular hydropower system can be determined.

2.6.1. Using Flow, Head and Power as a criteria

The selection of type, geometry and dimensions of the turbine for a particular micro hydro site depends on the site characteristics; the dominant factors being the head available and the power required. Selection also depends on the speed at which it is desired to run the generator or other devices loading the generator. Some turbines like the cross-flow and the Kaplan work efficiently with a large range of flow variation while others like the propeller turbines work only for a narrow range of flow variation with their efficiency falling rapidly with a little variation in flow. Therefore this criterion may help for selection of the turbine type to be used especially in standalone MHP systems. The selection of type of turbine for a hydropower site is assisted by the following tables, charts and graphs taking all the above variables in to consideration.

Table2.6. classification of hydro turbines according to head flow and power [12]

Classification	Turbine Name	Head (m)	Flow (m3/s)	Power output (kW)
Impulse	Pelton	50-1000	0.2-3	50-15000
	Turgo	30-200	0.2-5	20-5000
	Cross-flow	2-50	0.01-2	0.1-600
Reaction	Kaplan & Propeller	3-40	3-20	50-500
	Francis radial flow	40-200	1-20	500-15000
	Francis mixed flow	10-40	0.7-10	100-5000

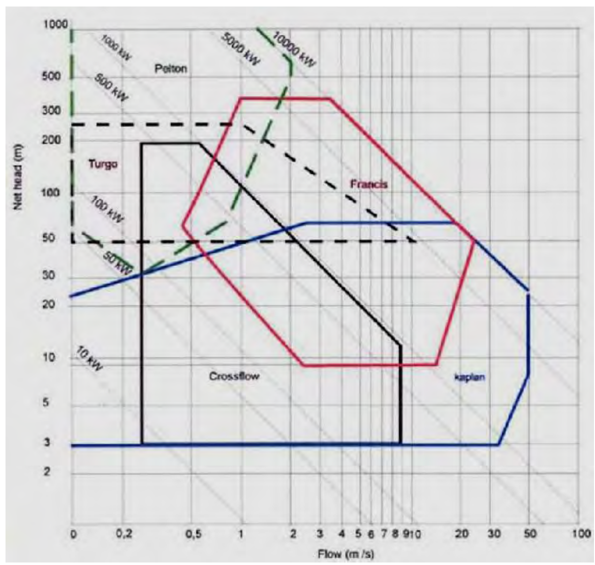

Figure2.7. Turbine Selection Chart [16]

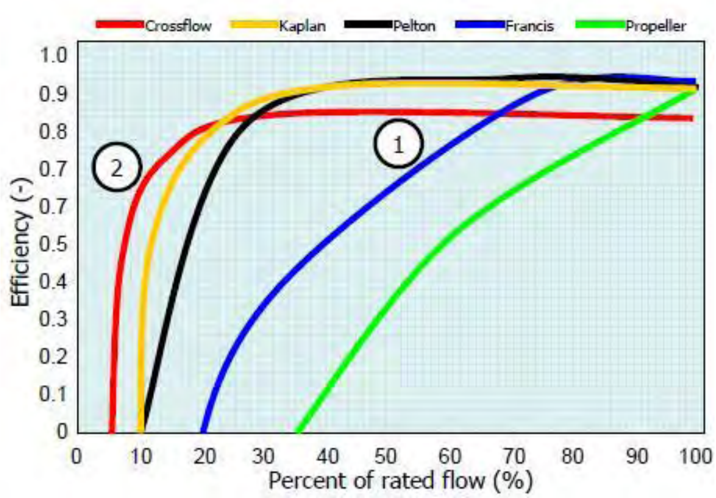

Figure2.8. Turbine efficiency curves for typical small hydro turbine types [24]

2.6.2. Using specific speed as a selection criteria

The specific speed constitutes a reliable criterion for the selection of the turbine. If we wish to produce electricity in a scheme with H_{net} m net head, using a P kW turbine directly coupled to a standard N rpm generator we should begin by computing the specific speed according to Equation 2.1:

$$n_s = N \frac{\sqrt{p}}{H_{net}^{5/4}} \tag{2.1}$$

where,

n_s is turbine specific speed

N is running speed of the turbine (rpm)

H_{net} is net head (m)

P is the power to be generated (kW)

After computing the specific speed, it is possible to choose which turbine type to use or to decide whether to use a speed increaser like belts and gears. Using all this tools one can have appropriate selection of the turbine that is to be used for the site at hand.

Table2.7. Selection of turbines based on specific speed [12]

Type of runner	Specific speed (rpm)
Pelton	12-30
Turgo	20-70
Cross flow	20-80
Francis	80-400
Propeller & Kaplan	340-1000

2.7. System output Power calculation

For a power system the theoretical power available from the falling water can be calculated using conservation of energy as a guiding rule

2.7.1. Stored Potential Energy

For water to generate electricity, it has to move from higher elevation to a lower elevation. This water flow from an upper to a lower level represents a hydraulic power potential. The difference between the level of the upper reservoir h_{res} and the level of tail water h_{tw} is defined as the available head, H_{av}

$$H_{av} = h_{res} - h_{tw} \tag{2.2}$$

Therefore, the potential energy to move a mass m of water from the upper to the lower level is given by:

$$PE = mgH_{av} \tag{2.3}$$

where,

PE is the potential energy (Joules)
m is the mass of water (kg)
g is the acceleration due to gravity (9.81m/s^2)

2.7.2. Power Output

The potential energy calculated above is not all/the_only energy that may be available at the hydro generator output. Rather the output power is a function of the potential energy, the amount of water that flows and the types of electromechanical and civil work components used in the system. One thing, all the available head is not consumed for power generation instead some part of it will be lost in the system while flowing, especially in the canals and penstock pipes. The other thing is the electro mechanical components used in the system are not 100% efficient. Therefore, for the calculation of the available power from the hydro system we need to consider the effective head, $H_{eff,}$ and the efficiencies of system components. With all these considerations, the power output of the hydropower system can be obtained as.

$$P = \eta g Q H_{eff} \tag{2.4}$$

where,

P power output (kW)
η is the system efficiency
Q stream flow (m^3/s)
H_{eff} is the effective head (m)

2.8. Determining Head and Flow Rate

All hydroelectric generation depends on falling water. Stream flow is the fuel of a hydropower plant and without it, generation ceases. Accordingly, the study of any potential hydroelectric scheme must first of all address the availability of an adequate water supply. Secondly, the topographic conditions of the site chosen must ensure availability of sufficient head for the water to fall before the power house.

2.8.1. Measuring Head

Head is the vertical distance that water falls from the forebay or intake to the turbine, measured in meters or feet. For hydropower potential assessment in general, measuring the available head can be done by a quick and less costly methods which can be used for its preliminary determination. For MHP schemes, however, these methods often can be used for final determination of gross head, but needs selecting appropriate methods from the many available [13, 14].

Maps: Detailed topographic maps are useful for locating potential sites and for obtaining a rough estimate of head levels at the proposed intake, tailrace water levels, and the length of the pipelines, the size of the drainage area and the origin and destination of the stream. Smaller-scale maps if available are better because they have a higher contour resolution [13].

Pressure-Gauge Method: This method can be used for high heads and low heads; the choice of pressure gauge depends on the head to be measured. This method for measuring head relies on the fact that for every meter vertically below the free surface of water, water pressure increases by 9.84 kPa, independent of the shape of the container which confines the water.

The elevation difference (gross head) "Hg" between the free water surface and the pressure gauge can be equated by [14]:

$$Hg = \frac{P_g}{9.8} \tag{2.5}$$

where,

P_g is the pressure gage reading (kPa)

Measuring head using this method requires a long flexible tubing with its one end connected to the gauge. After the tubing is filled with water, the pressure reading of the gauge is done and applying the formula above will give an approximate estimate of the head.

Altimeters: Altimeters can be useful when undertaking high head pre-feasibility studies. The principle used is that for an increase in elevation of 100 m, atmospheric pressure decreases by approximately 9 mm of mercury. An altimeter used to measure the change in elevation is essentially a barometer calibrated in meters. The difference in elevation between the two points can be determined by reading the altimeter at the two points and the difference of the two readings, with some atmospheric effect considered, gives gross head required.

Sighting Meters: Hand-held sighting meters, also known as inclinometers or Abney levels, measure the angle of inclination of a slope.

Dumpy Levels and Theodolites: A dumpy level is the conventional tool used for measuring head. An experienced operator who can check the level‟s calibration should make the measurement. When using a dumpy level, the operator takes a horizontal sight on a staff held by a colleague and needs an unobstructed view. A theodolite can measure vertical and horizontal angles.

Using clear hose method: This method is reliable, reasonably accurate and inexpensive and is suitable for low-head sites. For this method a water filled clear hose is used and the difference of the water level of the two points gives the elevation difference of the two points.

2.8.2. Measuring Water Flow

2.8.2.1. Flow Data

Flow rate is the quantity of water available in a stream or river and may vary widely over the course of a day, week, month and year. When a site has been identified as topographically suitable for hydropower, the first task is to investigate the availability of an adequate water supply. In order to adequately assess the minimum continuous power output to be expected from the micro-hydropower system, the minimum quantity of water available throughout the year must be determined.

Some hydropower sites may be gauged and have recorded data of stream flow for a long period of time by an appropriate governmental body as the case in Ethiopia is by MoWE where as others may not be gauged and have no such recorded data. For sites which have no appropriate recorded data, stream flow data should be measured daily and recorded for at least one year; two to three years is ideal. If not, a few measurements should be made during the low flow season. For those watercourses which are not gauged, where observations of discharge over a long period are not available, and also measurement of stream flow is not possible, determination of stream flow data involves the science of hydrology whose purpose is to predict flow variation, the mean stream flow and the extreme high- and low-flow rates, the study of rainfall and stream flow, the measurement of drainage basin, catchment area, evapotranspiration and surface geology [14].

2.8.2.2. Flow-Rate Measurement Methods

If appropriate stream flow time series cannot be found, the discharge should preferably be directly measured. There are a variety of techniques for measuring stream flow rate:

Container Method: A common method for measuring flow for very small streams is the container method which involves diverting the whole flow into a container such as a bucket or barrel. This method involves recording the amount of time required for the discharge in the stream to fill the bucket. With Vc the volume of the container and t the time taken for the container to be filled, the discharge in the stream is found to be Q:

$$Q = \frac{V_c}{t} \tag{2.6}$$

where,

Q stream flow in m^3/s
Vc volume of container m^3
t time taken in s

Velocity area method: This is a conventional method for medium to large rivers, involving the measurement of the cross-sectional area of the river and the mean velocity of the water through it. In this method both velocity of water in the stream and the area of cross section of the stream at the selected point will be measured. The stream flow is then found to be the product of the average stream velocity and the area of cross section of the stream:

$$Q = A * \overline{V} \tag{2.7}$$

where,

Q stream flow (m^3/s)
A stream cross sectional area (m^2)
$\overline{V}$ average stream velocity (m/s)

Determining area: Especially in prospecting for potential sites, it is necessary only to obtain an approximate value of the cross sectional area of the stream. This is easily done by measuring both the width "W" and what to the eye appears to be the average depth "d" of the stream.

The accuracy of the above approach depends on the accuracy with which the average depth can be determined. If greater accuracy is required, the shape of the stream‟s cross section is approximated by a series of parabolas, where the stream‟s width is divided by an odd number of equally spaced points. Summing the individual partial areas, the total stream cross sectional area can be shown to equal the expression below:

$$A = \left(4d_1 + 2d_2 + 4d_3 + \ldots + 4d_n\right)\frac{W}{3} \tag{2.8}$$

where,

W-the width of the stream (m)

A-stream cross sectional area (m^2)

d_i-is the depth of the stream at the i^{th} point (m)

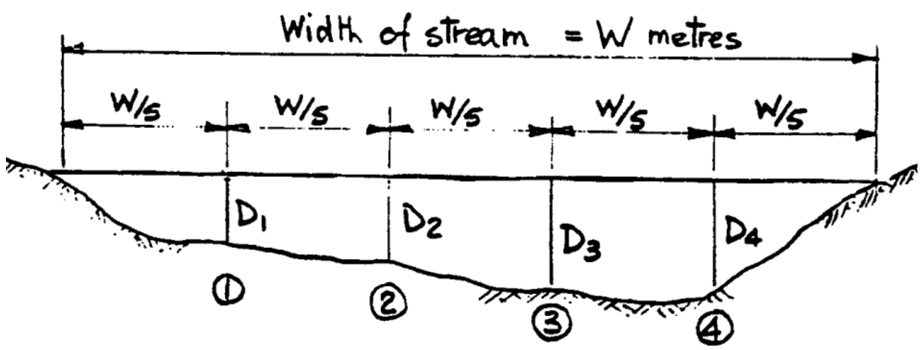

Figure2.9. cross section of a stream [26]

Determining stream velocity: One of the methods used in the determination of the stream velocity at the chosen point is the float method. In this method the length of the stream which is relatively straight and uniform is selected. The surface velocity V_s is then obtained by the equitation [12]:

$$V_s = \frac{D_t}{t} \tag{2.9}$$

where,

D_t distance traveled by the floating object (m)

t is the time taken by the object to cover the distance (s)

However, the velocity of the object at the surface V_s neither represents the average velocity of the stream $\overline{V}$ nor the average velocity $\bar{v}$ through the partial area of interest. An approximate value for the average velocity of the stream can be obtained by":

$$\overline{V} = C_v V_s \tag{2.10}$$

where,

C_v correction factor which depends on the type and shape of the stream bed

Approximate correction factors to convert measured surface velocity to mean velocity are as follows:

Table2.8. correction factor for different stream bed profiles [12]

Type of stream bed	Correction factor (C_v)
Concrete channel, rectangular, smooth	0.85
Large, slow, clear stream	0.75
Small, slow, clear stream	0.65
Shallow (less than 0.5 m) turbulent stream	0.45
Very shallow, rocky stream	0.25

There are also other methods of stream velocity measurement like velocity head road method and current meter method [12]

Weir Method: This method requires construction of a low wall or weir across the steam to be gauged, with a notch through which all the water in the stream flows. Over the years, numerous laboratory investigations have been conducted to calibrate notches of several standard designs so that discharge Q through these notches can be determined from a single linear measurement: the difference in elevation in the water surface h upstream of the weir and bottom of the notch. Table 2.9 below shows the developed formulas used for different types of weirs for the determination of flow.

Rectangular weirs are more suitable for large flows in the range of 1000 letters per second, and triangular weirs are suitable for small flows that have wide variation. A combination triangular/rectangular compound weir may be incorporated into one weir to measure higher flows; at lower flows the water goes through the triangular notch.

Table 2.9 Flow determination using developed formulas

Type of weir	Flow formula
Rectangular	$Q = 1.8(L - 0.2h) * h^{3/2}$
Triangular	$Q = 1.4h^{5/2}$
Cipoletti (trapezoidal)	$Q = 1.9Lh^{3/2}$

Slope area method: This method uses an open channel flow equation such as the Manning"s equation which is frequently used to determine discharge in power canals:

$$V = \frac{r^{2/3} S^{1/2}}{n} \tag{2.11}$$

where,

V average flow velocity in channel (m/sec)
r hydraulic radius (m)
S Slope of the stream"s surface
n Manning"s coefficient

This equation gives the average flow velocity. The stream discharge is then simply found to be the product of this velocity with the mean cross sectional area.

There are also other methods used for stream discharge measurement like the stage discharge method and salt dilution method.

CHAPTER THREE
METHODOLOGY AND APPROACHES

This chapter deals with the methodologies used for data collection both during field work and in Addis Ababa from MoWE. The data analyses as well as the simulation studies using HOMER are also presented in this chapter. It also covers design of some civil work and electromechanical components required for the selected hydro sites.

3.1. Primary Data Collection

Primary data are those collected by the direct involvement of the researcher for the purpose that is intended to be done, may be conducted during field survey.

During field survey, primary data is collected through direct measurement (of the head) of each river, interview with the local people (of the number of households nearby the rivers and the average number of people per household and, the type of loads they may use).

For the village around Jedeb River, called Belem"na there are 150 households. This area is suitable for irrigation for the crops like potato and barley. These irrigating lands are supplied by transporting water a long distance away from the farm land using plastic tubes. Some of the peasants use diesel pumps. Therefore, in this area, 5 water pumps are assumed to be used for irrigation purpose.

The Muketin and Yewubesh villages which are found near the Temcha River, one is opposite to the other village across the river, have a total of 200 households people with 2 churches but because the area is not suitable for irrigation, irrigation pumps are not assumed. Therefore, the loads which are considered in this area are only home loads.

The last river for this thesis work is the Gilgel Abay with the village called Kilaj near by the site. In this village there are 200 households. Like the villages in the Temcha River, there is no land used for irrigation purpose but because the land is suitable for irrigation 5 irrigation pumps are assumed for the HOMER model.

For all the three sites, even if there is no night schooling which may be due to resource shortage, a night schooling of one class with 6 florescent lamps with in the class in each village is considered.

3.2. Secondary Data Collection

Secondary data are those collected by someone else (may be the researcher) for another purpose. For this thesis the secondary datum that is collected is the flow rate of the rivers collected for many years by MoWE of Ethiopia. The data consists of monthly average, monthly minimum and monthly maximum flow data for more than 30 years. Sample of these data is placed at the appendix.

3.3. Data Analysis Approach

Both the primary data measured from field work and secondary data obtained from MoWE as well as information gathered during field survey from interviewing the local people are used for this thesis.

The MoWE is logging data for most of the streams in Ethiopia using gauging station. Luckily enough two of the three sites chosen are at the locations of the corresponding gauging station, except the one which is on the river Jedeb found near amanuel wereda at kebele Belem‶na. As already said before, the stream data for the river is found from MoWE, but because the gauging station and the power plant site are not at the same location, a hydrological analysis is used to map the data from the gauging station to the plant site.

3.3.1. Mapping of stream data

To estimate the hydropower potential of a site, the availability of stream flow data recorded for a long time throughout each year of concern is very important. But it may not be true that the sites for the hydropower plant and that for the gauge station are exactly at the same place. Therefore, it is important to transfer the data of the gauging station to the site. Equation (3.1) [27] describes this methodology.

$$Q_{site} = \left[\frac{DA_{site}}{DA_{gauge}} \right]^c * Q_{gauge} \tag{3.1}$$

where,

DA_{site}	drainage area of the power plant site (m^2)
DA_{gauge}	drainage area of the gauge (m^2)
Q_{site}	discharge at the site (m^3/s)
Q_{gauge}	discharge at the gauge (m^3/s)
c	a parameter typically varies between 0.6 and 1.2

The thing here is the selection of the value of c. [27] put this as follows:

1. If the drainage area (DA) of the site is within 20% of the drainage area of the gauge ($0.8 \le \frac{DA_{site}}{DA_{gauge}} \le 1.2$), use c=1. The estimated discharge at the site will probably be within 10% of the actual discharge, which is normally sufficient.
2. If the DA site is within 50% of the DA gauge, consider whether the data of the two gauges (upstream and downstream gauges, if any) can be combined. In addition, when a weighted average between upstream and downstream gauges is possible, the following linear interpolation (Equation 3.2) may be applied for a site lying between upstream and downstream gauges.

$$Q_{site} = \frac{(DA_{guage1} - DA_{site})Q_{guage1} + (DA_{site} - DA_{guage2})Q_{guage2}}{DA_{guage1} - DA_{guage2}} \tag{3.2}$$

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

As can be seen from the above equations, the drainage areas of both the gauge station and the plant site are required. For this purpose ArcGIS software and ASTER DEM data of the site found from internet are used. Using the hydrology tools of the spatial analyst toolbox of ArcGIS like Fill, Flow Accumulation, Flow Direction, Flow Length, Snap Power Point, Stream Link, Stream Order and watershed the drainage areas of both points at the gauge station and the site are found. Therefore, by using Equation 3.1, the stream flow data is mapped to the site and input to the software.

3.4. Load estimation and scheduling

As one of the data input to the software, the load demand of each corresponding village is very essential. With data of the total number of households in each village and using a typical household appliances that these people may use, this section discuses how the load profile of each area is obtained. For all the three sites, the types of loads used are similar, except for some of them which may vary with the number of households in each village and hence while giving input to the software; this is taken into consideration. Table 3.1 below shows the type of loads used and their power consumption.

Table3.1. Typical loads used and their scheduling

Type of Load	Power Rating	Schedule
Home lighting	11 W (Energy saving type)	18:00-0:00
Night lighting	5 W	0:00-6:00
Cooking	3 kW	8:00-16:00
Television	70 W (19‘‘ color TV)	16:00-18:00, 20:00-23:00
Irrigation Water pump	180W (34L/min)	6:00-8:00
Radio	15 W	16:00-18:00, 20:00-23:00
Well Pump	373 W (68L/min)	*
Megaphone	15 W (for church use)	4:00-8:00, 12:00-15:00
Church lighting	60 W	4:00-8:00,12:00-15:00
School lighting	60 W (florescent)	17:00-20:00

*_ *the schedule for this load is site dependent*

For each household in all the villages, the lighting loads are of 11 W power saving florescent lamps three in number per house hold operating from 18:00-0:00 while the night lights are of 5 W rating 1 per household operating from 0:00-6:00.

In all the villages, night schooling of one large class per village having 6 florescent lamps each rated for 60 W are proposed and are to be operational in the hours 17:00-20:00.

The cooking loads are rated for 3 kW for each house hold working from the hours 8:00-16:00. The scheduling for the cooking loads is such that each household is assumed to cook 3 times a week for 40min per cook. Therefore, the total time of the week from 8:00-16:00 is fairly distributed and for each household and that each household is assumed to cook during the time which it is allocated for.

The television and the radio receiver is taken to be 19" color TV with a power rating of 70 W and 15 W respectively both operating from hours 16:00-18:00 and 20:00-23:00 per day.

In each village there are churches and some loads are proposed for them. These loads are lighting 6 in number per church with a power rating of 60 W florescent lamp and a megaphone 3 in number rated for 15 W. Both the lighting and megaphone are to be operated from 4:00-8:00 and 12:00-15:00 except during Saturday and Sunday that they are to be operated from 4:00-8:00 and this is taken into consideration while inputting to the software.

The other types of loads to be considered are well pumps. The scheduling of these pumps and the determination of the length of their operating time depends on their pumping capacity and the daily water demand of each village under study. From [28] on average the daily water consumption of a household is figured to be 80 l. Using this data and the number of households in each village, the daily water requirement of the corresponding villages is calculated. From [29] the power rating of well pumps and their pumping capacity is available. For this thesis, the one in Table 3.1 above is chosen whose power rating and pumping capacity shown in the table. For example the people in kebele Belem‟na with 150 households, their daily water demand is:

$$150 * 80l / day = 12m^3 / day$$

With the selected pump with capacity of 68 l/min, the time taken that the pump should run is:

$$12m^3 / day * 68l / day * \frac{60}{1000} hr / m^3 = 3hr \, .$$

Therefore, for this village the selected pump should run for three hours per day. The same calculation is done for the other two villages. These loads can run during light load hours or can be considered as deferrable loads.

Exception to the load profile: For the rainy months from June to October, irrigation pumps are not required to work. So the power consumption of these pumps for these months is taken to be zero and it is input to HOMER.

3.5. Design and simulation with HOMER

3.5.1. Introduction to HOMER

HOMER is a micro power optimization model used for both off-grid and grid connected power systems so as to ease the task of evaluating their design [30]. While designing a power system one need to decide what components to add, the size and number of the components to be used, which technology with respect to cost and availability from the many available technologies. These and other conditions make it hard for an individual or a group to decide precisely and accurately which system configuration to use. But HOMER's optimization and sensitivity analysis algorithms make it easier to evaluate the many possible system configurations.

While using HOMER one is required to create a new model and give the new model created with inputs such as the technology options available (wind, solar, hydro etc), component cost and resource availability (for example flow rate, head for hydro resources). Using these inputs, HOMER simulates different system configurations, or combinations of components, and generates results that can be viewed as a list of feasible configurations sorted by net present cost. The simulation result from the software can be obtained in a verity of forms such as tables and graphs that one can export to use in reports and presentation.

"HOMER simulates the operation of a system by making energy balance calculations for each of the 8,760 hours in a year. For each hour, HOMER compares the electric and thermal demand to the energy that the system can supply and calculates the flows of energy to and from each component of the system. For systems that include batteries or fuel-powered generators, HOMER also decides for each hour how to operate the generators and whether to charge or discharge the batteries. HOMER performs these energy balance calculations for each system configuration that you want to consider. It then determines whether a configuration is feasible, i.e., whether it can meet the electric demand under the conditions that you specify, and estimates the cost of installing and operating the system over the lifetime of the project. The system cost calculations account for costs such as capital, replacement, operation and maintenance, fuel, and interest" [31].

HOMER has a sensitivity analysis capability where one can see the effect of change of resources availability and economic conditions on the cost effectiveness of the technology options used. Using the range of resource availability and economic conditions input to the software as sensitivity values, HOMER simulates each system configuration over the range of these values. The results of a sensitivity analysis are very important to identify the factors that have the greatest impact on the design and operation of a power system.

3.5.2. HOMER model of the systems

Except one site which is found in Temcha River, the other two sites have the same type of HOMER model. While simulation is carried out for the Temcha site with a hydro-only case, there is 7 kW unmate load. To supply this unmate load, a diesel generator is used and is added in the system model for simulation. That"s why a model difference happened. For these models, the inputs to the software are hydro resources such as head and flow rate data and the load profile. There are other inputs to the software such as type of penstock pipe to be used and its dimension. This helps HOMER to calculate the head loss in the penstock pipe. With all these data input to HOMER, the software simulates and gives a list of feasible solutions sorted by net present cost. Figure 3.1 below show HOMER models of the three sites while Figure 3.2 shows the hydro resource &the load profile input data. Similarly, Figure 3.3 shows a flow duration curve for the hydro resource shown in Figure 3.2

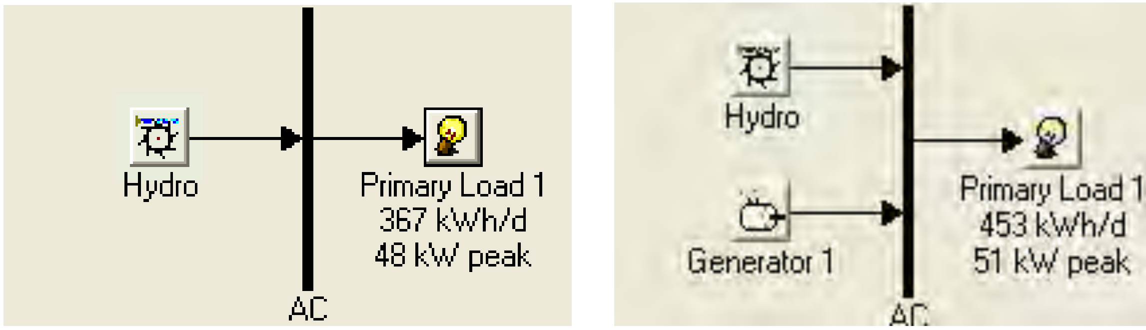

Figure3.1. HOMER model of MHP for sites at river: a) jedeb and G/abay b) temcha

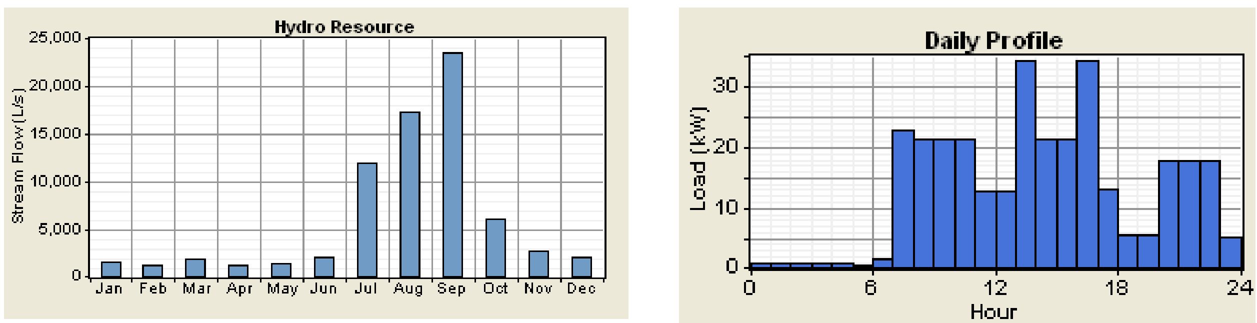

Figure3.2. a) Hydro resource b) daily load profile

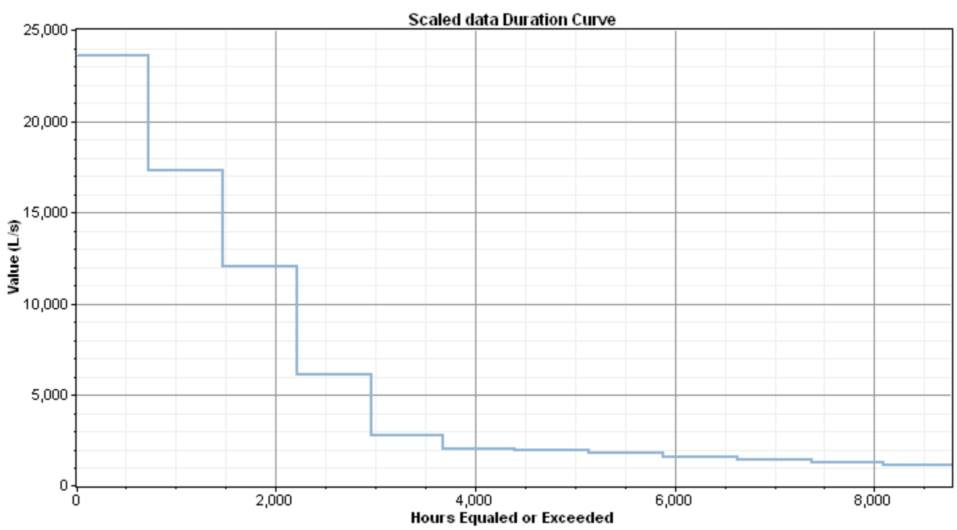

Figure3.3. Flow duration curve for the hydro resource in figure 3.2(a)

3.6. Electromechanical and civil work components Design

3.6.1. Electromechanical component design

The electromechanical components of MHP system are the turbo generator and other equipments such as the control system in the power house [2, 8, 25].

3.6.1.1. Generator design

From china synchronous generator manufacturers, Fujian Mindu Motor Co., Ltd., it is possible to order the required type of generator. Moreover, Marelli Motors from Italy can also supply the required type of generator for this application. The product details of both the manufacturers are attached in the Appendix B for more reference.

3.6.1.2. Turbine design

The type of turbine used in the sites considered is the Banki cross flow turbine. The selection of this turbine from others is based on the criteria that are discussed in the literature part. For example, its efficiency is almost constant with in a relatively high variation of design flow as shown in Figure 2.8. The other reason is because of possibility that the turbine can be manufactured from locally available materials and with local people having got simple training. Therefore, this section will go through the parameter design for this turbine.

For sizing of cross flow turbines the main parameters of interest are the runner length which may be equal to the nozzle length, runner diameter and jut thickness. With Q as a design flow for the turbine, N the rotational speed of the turbine in rpm and H_{net} as the net head, cross flow turbine parameters will be calculated as follows:

Runner diameter D: From [32, 33]

$$D = \frac{39.819\sqrt{H_{net}}}{N} \tag{3.3}$$

where,

D diameter of runner (m)

H_{net} net head (m)

N speed of turbine (rpm)

Here the rotational speed of the turbine depends on the type of system used. In systems where a speed increaser is not used, the speed of the turbine will be the same as the rotational speed of the generator to which the turbine is coupled. However it may not be true that the turbine is directly coupled to the generator, in which case a speed increaser of either gear train or belt type may be used with selected gear (speed) ratio, the selection of which depends on the turbine type used. For example, for cross flow turbines this ratio should not be greater than 3. With this and the rotational speed of the generator selected, the value of N can be found.

Runner length L:

$$L = \frac{Q}{D * C * K * \sqrt{2 * g * H_{net}}} \tag{3.4}$$

where

D is runner diameter (mm)

C is the velocity constant for the nozzle, which is defined as the ratio of the actual average velocity achieved to that predicted from Bernoulli"s equation. This constant cannot be determined accurately before doing the experiment for the turbine to be manufactured. This value signifies the smoothness or roughness of the nozzle. The higher the C the smoother the nozzle is and vice versa. Typically this constant ranges from 0.95 (rougher nozzle) to 0.99 (smoother nozzle)

K is an experimental coefficient which relates the jet thickness to the diameter of the runner. According to [33], this value ranges from 0.075 to 0.1. The value of this constant ultimately determines the number of blades required. For example a value of K=0.075 corresponds to blade number of 21 while that value of K=0.1 corresponds to 16 blades. For calculation purpose, as the author did, the value of K to be taken is the mean of the value of K. therefore, the mean value is K=0.087 corresponding to which the number of blades is 18.

Thickness of jet, S_0 **:** As has been briefly discussed in the literature review part, the nozzle is one of the main parts of a cross flow turbine. The function of this component is to drive a water jet of rectangular cross section to the turbine runner. Therefore, the thickness of the nozzle can alternatively be defined as thickness of jet.

$$S_0 = \frac{jet\_area(A)}{L} \tag{3.5}$$

where,

$$jet\_area(A) = \frac{Q}{C\sqrt{2 * g * H_{net}}} \tag{3.6}$$

There are also other parameters of the turbine to be considered while designing the whole system. For example,

Space between the blades can be obtained by the formula:

$$s = 2 * K * D \tag{3.7}$$

Number of blades B:

$$B = \pi \frac{D}{s} \tag{3.8}$$

Radial rim width, α, the distance between the starting and ending points of the blades measured radially:

$$a = 0.17 * D \tag{3.9}$$

Radius of blade curvature R_c **:**

$$R_c = 0.163 * D \tag{3.10}$$

3.6.2. Civil work design

Because the share of the civil work to the total cost of a MHP system is not less than 30%, proper design of this part of a system is vital. Not only is this but also there are other reasons that make the proper and careful design of the civil work vital. For example, it is this component which will drive the required flow rate to the turbine and which creates the required head. Therefore, the design of the civil work components should be done carefully and most of the time it accounts more than 60% of the overall design of a MHP system.

Here under is the design of some civil work components depending on the available head and flow rate at the sites considered.

3.6.2.1. Canal dimension and slope

Figure 3.3 below is a flow chart showing how to determine canal cross sectional dimensions and canal slope. For this figure, the input parameters are the required flow rate which is the design flow rate for the system, the velocity of the water through the canal which depends on the type and made up of the canal and the soil condition of the area [34], the canal profile which is the type of canal considered during design, and the roughness coefficient which is constant and depends on the material from which the canal is made.

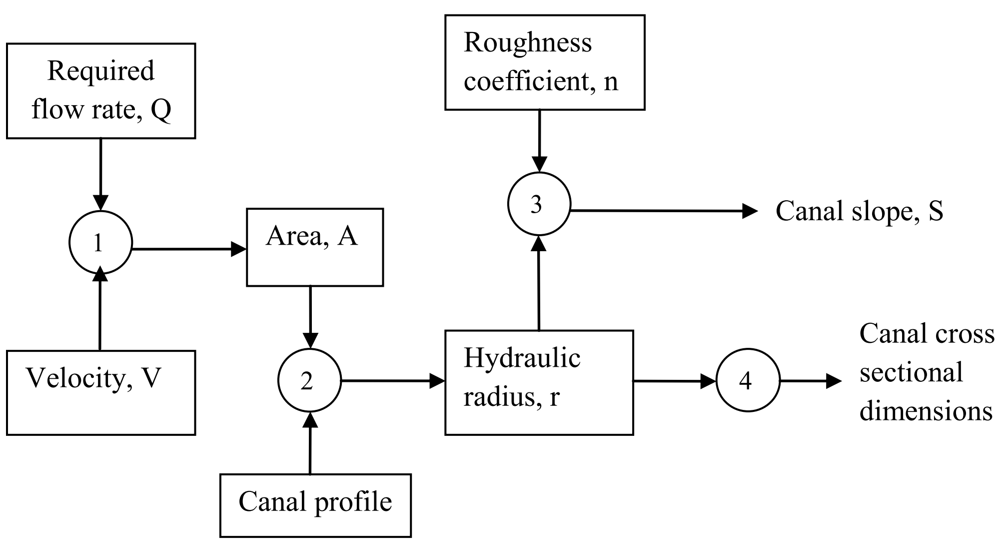

Figure3.4. Determining canal dimensions

Hydraulic radius, r is defined as:

$$r = \frac{A}{p} \tag{3.11}$$

where,

A is the cross sectional area of the water in the canal or pipe (m^2)

p wetted perimeter of the canal or pipe (m)

Hydraulic radius depends only on the profile selected and the cross sectional area of water it contains. The table below shows the hydraulic radius for the most efficient canal cross sections for common profiles:

Table3.2. hydraulic radius for different canal profiles [12]

Profile Type	Hydraulic radius, r
Semicircular	$0.40\sqrt{A}$
Trapezoidal	$0.5\,0.5\sqrt{\frac{\sin\theta}{2-\cos\theta}} * \sqrt{A}$, θ^*
Rectangular or triangular	$0.35\sqrt{A}$

** The angle of the banks of the canal above the horizontal*

Cross sectional area, A can be determined from the basic relation:

$$A = \frac{Q}{V} \tag{3.12}$$

where,

Q is the design flow rate in the canal (m^3/s)

V the speed of water in the canal (m/s)

Canal slope, S: using Manning"s equitation

$$S = \left(\frac{nV}{r^{2/3}}\right)^2 \tag{3.13}$$

Cross sectional area of the wetted area for different canal profiles can be obtained using the expression in the table below:

Table3.3. cross sectional dimensions of different canal profiles [12]

Type of Profile	Dimensions
Semicircular	Diameter, $d = 4r$
Rectangular	Depth, $d = 2r$, Width, $w = 4r$
Triangular	Depth, $2.8r$, Width, $w = 5.7r$
Trapezoidal	Depth, $2r$, Width, $w = \frac{4r}{\sin\theta}$

3.6.2.2. Sizing of overflow spillways

Canals are designed to transport water of required flow from the intake to the settling basin. But in the route to the settling basin or from the intake itself water of excess magnitude may be added to the channel. This excess water should be removed before going to the turbine. This will be done by using correctly designed spill ways.

The discharge in a spill way can be determined by:

$$Q = C_d l h^{3/2} \quad (3.14)$$

where,

Q discharge through the spill way (m^3/s)

l length of the spill way (m)

h head above crest of the spill way (m)

C_d coefficient of discharge whose value depends on the type of weir profile used and is from 1.4 to 2.3

Therefore, having the discharge in the spillway, it is possible to determine the size of the spillway required. The selection of the pipe material as well as the wall thickness depends on the pressure (static which depends on the head and wave pressures which depends on how fast the water is flowing) that the pipe will experience.

3.6.2.3. Sizing of penstock pipes

The size of penstock pipe to be selected is dependent on the material type to be used and its availability. To serve a given task with all the design parameters the same, penstocks designed from different materials will have different size [22].

In addition, as has been discussed in the literature part, the sizing of a penstock depends on factors like; head loss due to friction, cost of the pipe and installation and pressure limitation of the pipe as a factor of wall thickness.

As the diameter of the pipe decreases the velocity of the water flow increases which increases the head loss due to the increased friction. And when the size of the penstock is increased, the head loss will be decreased; however the cost of the penstock increases drastically. Therefore, a proper sizing of the penstock is very important [24].

Hence, it is good to see the types of materials used in penstock construction before going directly to the design.

The most commonly used penstock materials are:

- PVC (PolyVinyl Chloride)
- Steel
- Polyethylene
- FRE (Fiber Reinforced Epoxy)
- Transite (asbestos cement)

The first thing to consider while sizing the penstock is selecting the proper penstock diameter corresponding to the design flow. This can be done in two ways: one by using developed formulas and the second method is by using graphs developed for this purpose [35].

3.6.2.3.1. Procedures of diameter selection in the graphical method

To determine the diameter of the penstock pipe in this method, the graph placed in appendix C is used. This graph had flow rate in cfs in the horizontal axis, head loss factor per 100 feet of pipe length in the vertical axis and a line running diagonally from bottom left to top right showing deferent penstock diameters in inch and a line running from the vertical axis diagonally down to the horizontal axis showing water velocity in the pipe in fps.

From the point of design flow a line is drawn vertically upward till it touches the velocity line. The intersection point of the vertical line drawn and the velocity line may or may not be at the penstock diameter line. If the intersection point is not in any of the penstock diameter lines choose the greater pipe diameter to insure market availability of penstock pipes and to minimize head loss due to friction.

From the intersection point of the vertical line and the diameter line, a horizontal line to the left is drawn to get the head loss factor per 100 feet of pipe length. Finally the actual value of the penstock diameter, water velocity and head loss factor are recorded.

After the selection of the pipe diameter, the next step is to chose the material type to be used which helps in the determination of the actual head loss in the penstock.

To aid in determining the design pressure of the penstock and selecting the suitable pipe material, Table C.1 in the appendix shows the wall thickness (t_w), pressure rating (P_R), and surge allowance factor (S_A) for several sizes of commonly available pipe materials.Using the penstock diameter selected earlier, select the three material parameters of the available penstock materials from TableC.1 with P_R of greater or equal to the static pressure calculated as under in Equation 3.16.

For the selected materials, the design pressure of the penstock can be calculated as:

$$P_d = P_s + (S_A + v) \tag{3.15}$$

where,

Pd penstock design pressure (psi)

P_s static pressure (psi)

S_A surge allowance factor

v velocity of water in the pipe in fps from figure C.1& C.2

The static pressure can be determined by:

$$P_s = 0.433H \tag{3.16}$$

where,

P_s static pressure (psi)

H design head (ft)

3.6.2.3.2. Penstock size determination using developed formulas

For the assessment of a preliminary design or a feasibility level, the most economical diameter can be estimated using the following formula [36]:

$$D_e = 0.52\frac{P^{0.43}}{H^{0.60}} \tag{3.17}$$

where,

D_e is the most economical diameter of penstock in m

H rated head in m

P rated capacity of plant in kw

To determine the minimum thickness of the penstock, based on the need for stiffness, corrosion protection, and handling requirements, the following formula can be used:

$$t_{\min} = \frac{D_e + 500}{400} \tag{3.18}$$

where,

$t_{\min}$ is minimum thickness of penstock in mm

D_e penstock diameter in mm

Until this we are able to determine the size of a penstock pipe for the design flow and head required. But we are not able to know how much head will be lost in the system.

The total head lose is a function of both friction loses and turbulence where the intake structure and bend & obstructions in the pipe contribute to this lose. To consider turbulence factors:

- Multiply the number of 90 degree bends by 30feet
- Multiply the number of 45 degree bends by 15 feet
- Add 15 feet for entrance at the intake
- Add 100 feet for the turbine isolation valve
- Sum all the additions and add to the total penstock length

This procedure gives the adjacent length (La) of the system.

However friction losses are function of the pipe size, length and material. The friction effect is accounted for with the pipe material correction factors shown in the table below:

Table3.4. friction loss correction factor (f_c) for different pipe materials [22]

Pipe material	Correction Factor (f_c)
Steel	1.16
PVC	0.77
PE	0.77
AC	0.87
FRP	0.77

To determine the total head loss due to both friction and turbulence, use the equation below:

$$H_{loss} = \frac{f_c * L_a * H_f}{100} \qquad (3.19)$$

where,

H_{loss} head loss in feet

H_f head loss factor which can be read from figure appendix C.1 & C.2

This will help in determining the net head available used for turbine size determination.

3.6.2.4. Sizing of the settling basin

Even if there are structures in the intake designed to remove particles which are carried by the flowing stream, it may not be possible to remove all, especially smaller ones. Therefore these small particles should be settled and removed through flashing before the water is driven to the turbine. To settle out, the particle in to consideration should have a settling velocity of at least "v_0" [37, 38]. Any particle with velocity less than this velocity will not settle in the basin.

$$\frac{60v}{v_0} = \frac{L_b}{D_b} \tag{3.20}$$

where,

v average velocity of water through the basin and given by

$$v = \frac{Q}{D_b W_b} \tag{3.21}$$

v_0 particle settling velocity (in m/min), dependent on particle size to be settled

$$v_0 = \frac{60Q}{A_s} \tag{3.22}$$

where,

$A_s = L_b W_b$ is basin surface area

$$L_b = 60\frac{Q}{v_0 W_b} \tag{3.23}$$

CHAPTER FOUR
RESULTS AND DISCUSSIONS

All the three sites, with their load and the generating system, have been modeled and simulated with HOMER. As well, designs of the components of the micro hydropower systems have been done. Therefore, this chapter deals with the results and discussion of HOMER simulation and component design for all the three sites.

4.1. HOMER Simulation results

As has been discussed in chapter three, all the models of the three sites are the same with their respective data input. The only exception to this is the case for the Temcha River where during April the system is found to be insufficient to supply the load. Therefore, after simulating the system with hydro only system, a diesel generator is added to work with the hydro so that there is no shortage during this month of the year. The reason for this is briefly discussed in Section 3.5.2.

4.1.1. Results of Jedeb River

The site for this river with a gross head of 10 m and a design flow rate of 1.091m^3/s is simulated with HOMER. It is found that the community loads can be supplied only by the hydro system without any other supplementary power source requirement. The figures below show the results taken from the simulation of the system for this site.

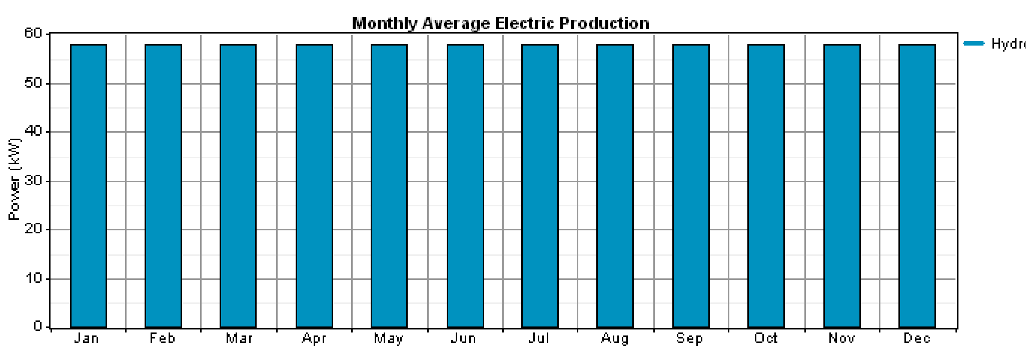

Figure4.1. Monthly average electric production

As is shown in the figure above, the system is able to produce 57.8 kW of power. But the peak load demand 37 kW, which is much less than the generated power by the power plant. As shown in Figure 4.2, at least 30 kW of excess energy is obtained from the system. This shows that the hydro system can supply the load without any problem, regarding capacity. The figure below shows the excess electricity from this site.

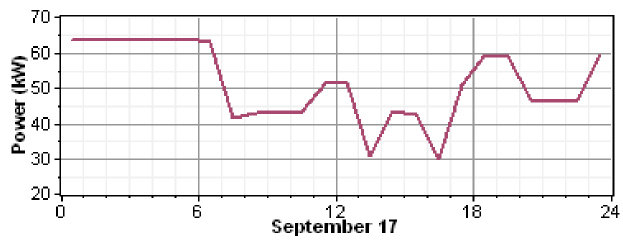

Figure4.2. Excess electricity

In the above figure, the excess electricity during the mid night is high. This does not mean that the system is always functional. Rather this is the result of HOMER simulation without control applied. If control is applied the generation during this time will be minimal.

4.1.2. Results of Temcha River

The site for this river with a gross head of 12 m is simulated first with a hydro only case. For this site two design flows were chosen, $0.8m^3/s$ with a 99.56% of exceedence in the flow duration curve; i.e. this flow rate is available throughout the year and $1.6m^3/s$ with a 98% exceedence in the flow duration curve. These two values were given to the software as a sensitivity values. With this into consideration, the HOMER model is simulated and the system was not able to supply the load especially during April during which the flow rate in the area is lower. The simulation result shows that there are unmet loads of about 7 kW during this month; Figure 4.3 below shows the unmet load of a day during April.

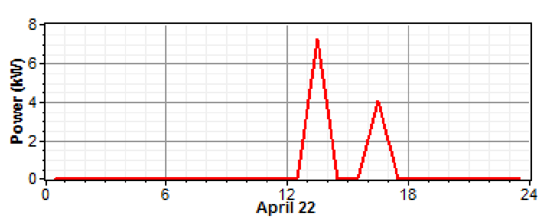

Figure4.3. Unmet load

To make the system reliable, the unmet loads must be supplied. There were many options to do this, for example wind, solar diesel or a hybrid of these can be tried. In this design, only the diesel system is used and from Cummunis manufacturer‟s data of diesel generators, generators of 8 kW, 16 kW, and 22 kW are used for simulation. The result shows that a diesel generator of 8 kW scheduled properly can supply the system load and there were no unmet loads found in this case. The overall simulation results for the two design flows with the generator added to the system is shown in the consecutive figures below.

Diesel Price ($/L) 0.7 Design Flow Rate (L/s) 800

Double click on a system below for simulation results. Categorized Overall

Hydro (kW)	Label (kW)	Initial Capital	Operating Cost ($/yr)	Total NPC	COE ($/kWh)	Ren. Frac.	Capacity Shortage	Diesel (L)	Label (hrs)
57	8	$ 101,222	460	$ 107,100	0.051	0.99	0.00	1,092	510
57	16	$ 102,445	1,297	$ 119,022	0.056	0.99	0.00	2,183	510
57	22	$ 103,361	1,925	$ 127,969	0.061	0.98	0.00	3,001	510
57	53	$ 108,098	5,171	$ 174,198	0.082	0.96	0.00	7,231	510

Figure4.4. Overall simulation result for design flow of 800 lps

Diesel Price ($/L) 0.7 Design Flow Rate (L/s) 1,600

Double click on a system below for simulation results. Categorized Overall

Hydro (kW)	Label (kW)	Initial Capital	Operating Cost ($/yr)	Total NPC	COE ($/kWh)	Ren. Frac.	Capacity Shortage	Diesel (L)	Label (hrs)
115	8	$ 101,222	989	$ 113,867	0.057	0.99	0.06	1,783	720
115	16	$ 102,445	2,311	$ 131,990	0.065	0.99	0.04	3,502	720
115	22	$ 103,361	3,276	$ 145,237	0.070	0.98	0.03	4,752	720
115	53	$ 108,098	7,642	$ 205,784	0.097	0.97	0.00	10,329	720

Figure4.5. Overall simulation result for design flow of 1600 lps

As can be seen from the above figures, when the design flow rate is $0.8m^3/s$, the hydro system will generate 57 kW power and using a diesel of 8 kW the system will supply the load sufficiently. However, we can generate 115 kW of hydro power, twice of the previous, by using a design flow rate of $1.6m^3/s$. With this design flow, however, the hydro system will be insufficient to generate power during April because the flow rate during this month is less than half of the design flow thereby necessitating a diesel of 53 kW instead of 8 kW.

Both the electric productions and the excess electricity for the two design flows considered are shown below.

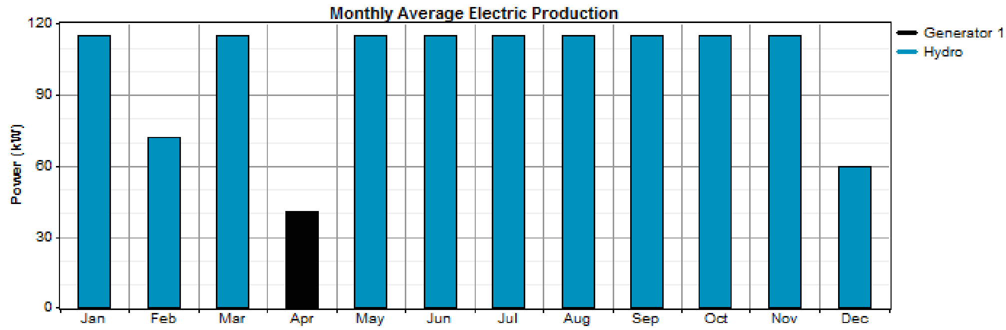

Figure4.6.Monthly electric production for design flow of 1600 lps

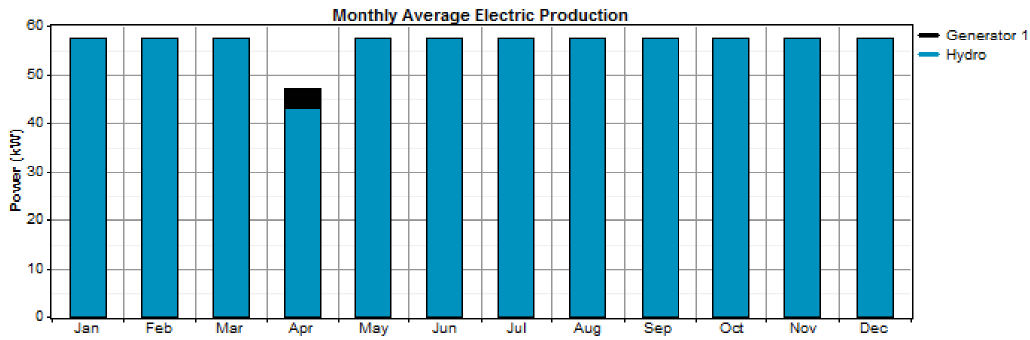

Figure4.7. Monthly electric production for design flow of 800 lps

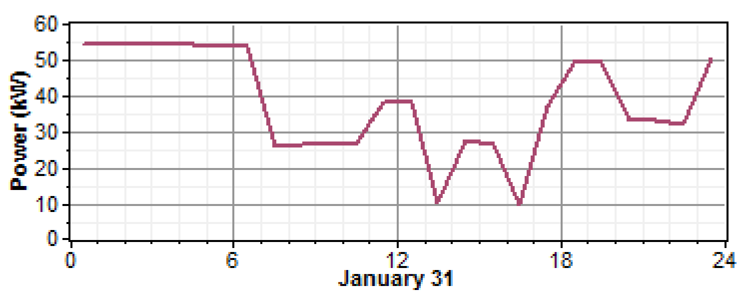

Figure4.8. Excess electricity produced for design flow of 800 lps

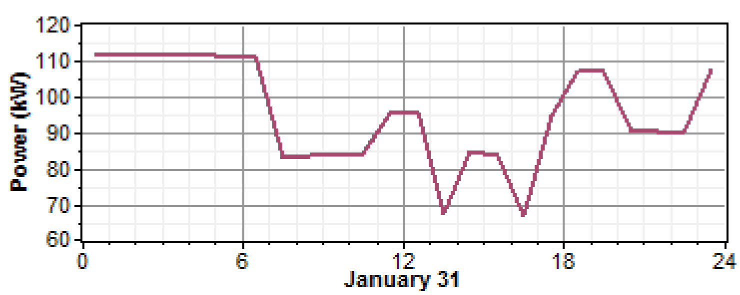

Figure4.9. Excess electricity produced for design flow of 1600 lps

Therefore, the system with the two design flow rates is able to supply the loads using properly selected and scheduled diesel generators.

4.1.3. Results of G/Abay River

The site in this river is found to have a gross head of 8m and a design flow of 1.791m^3/s which persists throughout the year. With all the data required input to HOMER, the system is simulated. The simulation result shows that this river can supply the local people safely and reliably without any requirement of supplementary sources.

The hydro turbine output is 738,395kWh/yr while the total community load is 155,856kWh/yr which leads to an excess electricity of 582,503kWh/yr around 78.9% of the generating capacity. This shows that the hydropower system in this site can supply other nearby villages or can handle other types of system loads in the village already considered such as milling loads. The following figures show results taken from the simulation result.

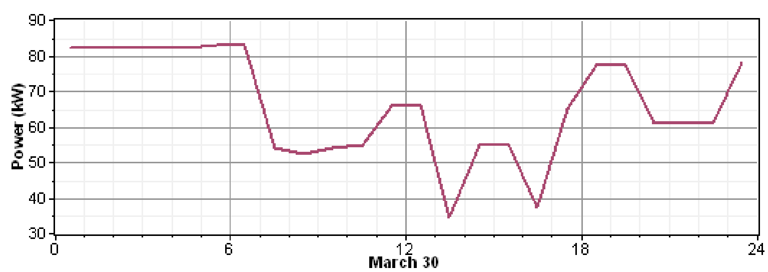

Figure4.10. Excess electricity

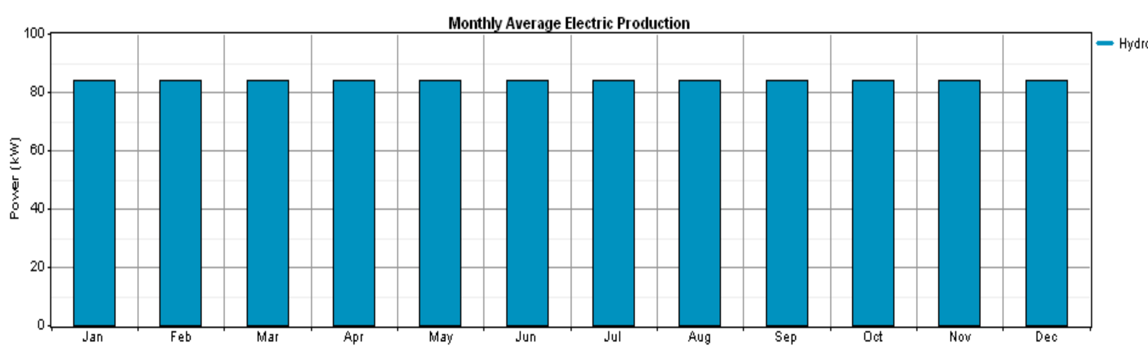

Figure4.11. Monthly average electric production

4.2. Design results of civil work and electro-mechanical components

4.2.1. Results for Jedeb River

As has been indicated in chapter 3 &4, the river has a gross head of 10 m and a design flow rate of 1.091m^3/s. therefore the design results for this site will be based on this two values; there will be other constants/values but they will be indicated at the necessary place. This section starts from the calculation for the net head.

$$H_{net} = H_{gross} - H_{loss} \tag{4.1}$$

where,

H_{loss} is the head loss in the system.

L_a is the adjacent length calculated as

$$L_a = 15 + 100 + 10 * 3.28$$
$$= 147.8084 ft$$

With the value of f_c taken to be 1.16 for steel penstock as from Table 3.4 and the value of H_f taken 0.4 from Figure C.2, Equation (3.17) gives the head loss to be 0.209041m.

Therefore, the net head is found to be $H_{net} = 9.79m$

The next is the result for the turbine design.

Runner diameter, from Equation (3.3)

$$D = \frac{39.819\sqrt{9.79}}{750}$$
$$= 0.166m$$

Jet area, $A_{jet} = \dfrac{Q}{C\sqrt{2gH_{net}}}$

$$= 0.08033m^2$$

Runner length, from Equation (3.4)

$$L = \frac{3.623 * 1.091}{\sqrt{9.79}}$$
$$= 1.2633m$$

Therefore, jet thickness from Equation (3.5),

$$S_0 = \frac{0.08033}{1.2633}$$
$$= 0.06358m$$

The space between the blades from Equation (3.7),

$$s = 2 * 0.087 * 0.166$$
$$= 0.0289m$$

The number of blades from Equation (3.8),

$$B = \frac{\pi * 0.166}{0.0289}$$
$$= 18.055 \sim 18$$

Radial rim width from Equation (3.9),

$$\alpha = 0.17 * 0.166 = 0.02824m$$

Radius of blade curvature from Equation (3.10),

$$R_c = 0.163 * 0.166 = 0.0271m$$

Next is the design result for the civil work

Canal slop: From Equation (3.13)

$$S = \left(\frac{0.014 * 1.5}{r^{2/3}}\right)^2$$
$$= 0.002211$$

where,

0.014 is the value of the Manning‟s coefficient for a concrete canal and

r is the hydraulic radius from Equation (3.12) and Table 3.2 and whose value is 0.2985

Canal cross sectional dimensions: From Table 3.3 for a rectangular canal profile and with the value of r as above:

Depth, d=0.597m

Width, w=1.194m

Overflow spillway: From [35] for a round over fall spill way, the weir coefficient is taken to be 2.1 and from Equation (3.14)

$1.091 = 2.1lh^{2/3}$ from this
$l = 0.65m$ and
$h = 0.710m$

Penstock: From Equation (3.18)

$$D_e = \frac{0.52 * 62.9^{0.43}}{9.79^{0.6}}$$
$= 0.785\text{m}$ and from Equation (3.19)

$$t_{\min} = \frac{785 + 500}{400}$$
$$= 3.2125\,\text{mm}$$

Settling basin: From Equation (3.21)

$$1.5 = \frac{1.091}{D_b W_b} \text{ from this}$$

$D_b = 0.5$ m and

$W_b = 1.5$ m, and from Equation (3.32)

$$L_b = \frac{60Q}{v_o W_b}$$

From [34] for a hydropower scheme of head in the range from 10m to 100m a particle size of diameter 0.2mm to 0.3mm are needed to be settled. From [38] for a particle size of 0.2mm, the settling velocity must be greater than 0.2m/sec (12m/min). Therefore,

$$L_b = \frac{60 * 1.091}{12 * 1.5}$$
$$= 3.637m$$

4.2.2. Results of G/Abay River

All the calculations done for Jedeb River are also applied for this river with some values unique to the respective site.

$Q = 1.79m^3/\sec$

$P = 83kw$

$H_f = 0.25$

$f_c = 1.16$

With these values and with all the equations used for the Jedeb case, the results for this site are as under:

$H_{net} = 7.81135m$

The table below shows the design results for the cross flow turbine considered for this site.

Table4.1. results for cross flow turbine design for G/Abay River

Parameter	Design Result	Parameter	Design Result
Renner Diameter, D	0.1484m	Space between blades, s	$0.0258m$
Runner length, L	2m	Number of Blades, B	18
Jet Area, A_{jet}	$0.147542m^2$	Radial Rim Width, α	$0.0252m$
Jet Thickness, S_0	$0.074m$	Radius of blade curvature, R_c	$0.0242m$

Civil work design

Hydraulic radius,

$$r = 0.35\sqrt{\frac{1.79}{1.5}} = 0.38234m$$

Canal slop,

$$S = \left(\frac{0.014*1.5}{0.38234^{2/3}}\right)^2 = 0.00159$$

Cross sectional dimension,

Depth, $d = 0.765m$

Width, $w = 1.53m$

Overflow spillway,

$h = 0.85$ and $l = 0.95$

Penstock,

$D_e = 0.999m$ and $t_{\min} = 3.7475mm$

Settling basin,

Depth, $D_b = 0.6m$; Width, $W_b = 2m$ and length, $L_b = 4.475m$

CHAPTER FIVE

CONCLUSIONS, RECOMMENDATIONS &SUGGESTION FOR FUTURE WORK

5.1. Conclusions

In this thesis, MHP potential assessment and preliminary design of some basic components of MHP systems of the selected sites is done.

To assess the potentials of these sites both primary data from field survey and secondary data from MoWE were gathered, analyzed and used. With these data a HOMER model of each system is built and simulation is carried out. As is discussed in Chapter 4, the simulation results show that the generated energy from each power plant site is more than enough of the load to be supplied and even can supply for an increase in demand of at least 100% of today‟s demand.

Except one of the sites at Temcha River, the rest are taken to be a hydro-only system. The site in this river, however, is simulated as a hybrid system of hydro and diesel. The reason is that, as discussed in detail, during April the river flow for this site is found to be so small that there are be about 7 kW load not to be supplied. With the addition of the diesel to the hydro system, however, these loads are met and therefore no unmate loads.

The result for the design of both electro mechanical and civil work components is based on the data found from simulation result of HOMER for the power output, from field survey for the gross head and others from literatures. Both the graphical and formula methods used for the design procedure are taken from literatures which are referenced appropriately.

5.2. Recommendation

Most of the rivers in Ethiopia are not gauged which makes finding relevant data difficult. So, I recommend the concerned governmental body to give due attention for this work.

5.3. Suggestion for future work

Of the renewable energies which may be relevant for rural electrification, only MHP is considered for the selected sites. However, there may be other renewable potentials such as solar and wind. Therefore, as a future work, I suggest somebody interested to see the feasibility of these sources as a hybrid system to the hydro-only case.

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APPENDICES

Appendix A: Sample of Flow rate data taken from MoWE

Table A.1. Flow rate data for Jedeb River

SUMMARY OF HYDROMETRIC DISCHARGE DATA

STATION NAME: JEDEB Nr./@ AMANUEL BASIN ABBAY DRAINAGE AREA 305

YEAR		JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
1960	I	3.780	3.570	3.400	2.080	2.520	4.200	50.710	78.450	41.830	14.690	3.710	2.550
	II	2.580	3.190	2.440	0.870	2.440	9.940	51.600	50.000	20.720	11.600	2.030	1.120
	III	0.870	0.870	0.710	0.630	0.540	0.540	2.030	16.780	13.100	2.170	1.040	0.540
1961	I	1.390	1.150	1.000	2.380	1.380	3.290	93.890	129.460	78.300	47.600	7.060	4.770
	II	0.610	0.680	0.610	2.480	0.910	4.550	95.200	95.200	49.790	57.400	4.550	2.480
	III	0.430	0.380	0.280	0.430	0.380	0.330	11.370	25.090	19.820	3.240	2.070	1.210
1962	I	2.520	1.460	1.700	0.960	1.660	3.500	44.860	89.970	59.890	28.910	4.040	2.420
	II	1.210	0.750	1.100	0.490	2.070	4.150	51.350	67.090	53.350	46.360	1.940	1.560
	III	0.750	0.550	0.490	0.330	0.330	0.610	2.480	18.220	12.720	2.070	1.210	0.680
1963	I	1.670	1.220	1.140	1.450	3.420	6.280	43.890	97.930	73.660	11.050	5.620	12.810
	II	0.830	0.750	1.560	1.320	3.590	9.520	56.170	76.500	102.500	10.560	6.730	21.460
	III	0.490	0.430	0.280	0.330	0.380	0.680	4.960	20.800	11.370	1.810	1.440	1.940
1964	I	4.620	5.140	2.000	2.170	2.010	13.190	68.840	90.410	60.520	36.380	5.700	4.970
	II	3.590	8.220	1.000	2.340	1.440	11.640	58.220	54.150	39.580	28.080	3.410	4.360
	III	1.210	1.000	0.610	0.430	0.550	1.000	8.480	18.860	14.400	3.259	1.680	1.210
1965	I	2.570	1.700	1.400	1.470	1.200	1.490	26.210	93.940	48.680	21.490	9.320	15.950
	II	1.210	0.830	0.680	0.910	0.910	1.100	35.880	80.000	82.000	15.020	6.040	24.430
	III	0.830	0.550	0.430	0.490	0.380	0.380	0.750	15.020	6.970	3.410	2.480	2.200
1966	I	1.330	1.100	1.110	1.140	0.796	2.770	21.500	74.300	54.200	7.910	3.620	2.140
	II	0.780	1.290	0.780	0.580	0.470	3.880	32.550	69.200	68.460	5.120	1.980	1.010

	III	0.370	0.220	0.255	0.330	0.255	0.290	1.500	14.280	5.350	1.850	1.010	0.640
1967	I	1.560	1.160	1.800	1.240	1.960	2.369	53.220	69.400	62.270	42.580	6.900	3.390
	II	0.640	0.520	2.390	0.640	1.980	4.270	67.720	52.340	59.800	75.500	3.010	1.850
	III	0.520	0.420	0.420	0.370	0.420	0.420	2.540	16.600	14.860	3.180	1.980	0.930
1968	I	2.120	1.690	1.570	1.400	1.640	9.530	45.740	74.770	51.190	24.700	4.900	2.970
	II	0.930	0.850	1.010	0.710	1.390	14.570	58.750	59.800	53.660	32.260	2.540	1.390
	III	3.710	0.580	0.470	0.420	0.420	0.850	9.180	14.280	18.850	1.390	1.390	0.930
1969	I	3.230	2.820	1.610	4.360	4.440	5.630	49.500	84.910	39.120	8.080	3.380	2.160
	II	2.250	1.730	8.640	4.900	3.880	5.350	38.060	57.720	31.680	5.350	2.250	1.100
	III	0.850	0.780	1.010	0.780	0.930	0.930	4.680	16.600	5.830	1.730	1.010	0.640
1970	I	1.580	1.120	1.500	1.490	1.150	2.520	40.640	105.090	70.450	32.810	11.99	18.290
	II	0.640	0.520	1.850	2.390	1.610	1.980	40.380	104.000	63.340	25.880	6.320	3.180
	III	0.520	0.420	0.370	0.330	0.255	0.330	0.780	20.660	12.250	6.320	3.350	2.390
1971	I	5.900	3.900	3.980	2.800	4.710	23.760	88.630	96.290	52.660	27.030	17.580	7.100
	II	2.540	1.850	2.850	1.390	5.120	30.230	67.720	83.500	46.800	31.100	23.850	3.700
	III	1.850	1.390	1.290	0.930	1.010	1.390	12.540	22.980	10.570	6.070	3.700	2.250
1972	I	2.940	1.440	0.960	1.320	1.820	3.890	35.870	67.930	52.830	11.680	10.150	3.410
	II	1.540	0.940	0.510	1.940	2.700	9.580	39.940	52.680	53.200	8.180	9.580	1.940
	III	0.750	0.390	0.270	0.330	0.270	0.180	3.730	9.580	9.100	2.700	1.940	0.940
1973	I	2.050	0.787	0.419	0.682	2.223	8.256	41.770	72.070	60.050	19.540	6.220	3.180
	II	1.120	0.520	0.318	1.540	1.760	16.230	47.200	92.000	82.000	22.660	3.590	1.540
	III	0.520	0.201	0.108	0.108	0.318	0.318	2.480	11.860	8.040	3.310	1.430	0.920
1974	I	1.396	0.696	1.269	0.905	6.086	10.040	63.020	104.310	45.900	18.700	4.810	1.730
	II	0.740	0.318	1.650	0.380	10.730	8.230	66.000	78.000	66.500	29.400	3.890	1.020
	III	0.318	0.201	0.201	0.257	0.318	2.000	10.290	18.020	8.610	3.450	1.120	0.450
1975	I	1.183	1.460	1.042	0.536	0.552	5.572	29.370	148.280	82.750	17.240	5.650	3.690
	II	0.550	1.280	0.700	0.400	0.500	6.980	43.730	167.400	89.400	14.500	2.710	1.770
	III	0.318	0.450	0.333	0.080	0.080	0.301	4.350	13.720	13.200	2.710	1.770	0.920
1976	I	2.520	1.460	2.020	0.993	5.638	14.520	69.860	104.770	47.880	9.550	13.600	3.860
	II	1.390	0.750	1.860	0.550	12.470	35.420	63.500	73.800	53.500	6.150	9.530	2.130
	III	0.550	0.450	0.400	0.301	0.450	0.990	11.750	16.060	6.630	2.040	2.130	1.060

1977	I	2.100	1.710	1.060	0.769	2.900	12.390	68.790	104.780	70.220	37.620	19.650	7.540
	II	1.060	1.060	0.860	0.990	6.310	17.100	75.000	78.600	91.800	29.340	15.280	4.210
	III	0.700	0.500	0.270	0.181	0.301	1.060	9.950	18.950	11.270	7.550	4.210	1.680
1978	I	2.670	1.060	1.172	0.881	1.920	8.742	75.920	110.760	92.890	43.910		
	II	1.680	0.550	0.550	0.920	1.680	11.750	69.000	95.800	77.400	73.800		
	III	0.550	0.333	0.270	0.181	0.400	0.333	6.470	19.540	15.280	4.210		
1979	I		1.580	1.866	0.682					67.340	14.060	4.600	2.230
	II		1.360	4.210	0.450					76.800	12.950	3.660	1.130
	III		0.400	0.270	0.125					13.720	3.420	1.130	0.550
1980	I	1.162	1.398	1.091	1.793	2.621	7.040	65.860	72.120	54.080	27.790	6.200	4.000
	II	0.550	1.200	1.860	3.540	8.500	15.280	71.400	67.500	67.500	66.000	2.940	1.860
	III	0.333	0.301	0.270	0.210	0.270	0.450	6.800	12.470	6.470	3.060	1.860	1.200
1981	I	2.020	0.840	1.140	0.890	2.010	3.210	38.420	122.290	101.100	34.650	5.680	1.850
	II	1.280	0.450	1.130	1.440	2.820	6.310	83.400	118.000	68.000	57.500	2.820	1.520
	III	0.450	0.301	0.270	0.181	0.153	0.181	1.200	18.140	13.820	2.940	1.520	0.333
1982	I	3.360	1.620	1.230	0.549	3.130	2.330	52.550	74.920	82.640	34.890	9.920	4.680
	II	11.03	0.800	1.060	0.500	7.550	3.180	115.600	70.800	87.600	28.200	12.71	7.550
	III	0.500	0.600	0.015	0.098	0.125	0.181	1.860	11.990	8.900	4.070	0.600	0.600
1983	I		1.387	0.727	0.350	6.690	4.460	19.860	119.920	92.640	26.280	8.400	2.370
	II		1.950	0.450	0.181	51.500	23.050	42.820	95.800	76.800	18.670	7.360	1.440
	III		0.301	0.125	0.080	0.125	0.270	0.450	9.110	13.720	3.180	1.520	0.450
1984	I	1.900	0.956	0.725	0.561	1.590	16.490	57.570	69.400	42.440	6.540	3.180	2.190
	II	0.895	0.533	0.322	0.296	4.079	19.080	60.020	49.940	39.720	5.480	2.090	2.138
	III	0.568	0.296	0.206	0.168	0.118	1.306	9.510	11.520	6.000	1.249	0.895	0.533
1985	I	1.048	0.650	0.357	0.535	1.560	5.270	46.570		45.920	13.210	4.070	2.390
	II	0.568	0.604	0.206	0.499	1.422	10.400	82.760		56.697	12.100	2.090	1.306
	III	0.227	0.118	0.078	0.091	0.186	0.227	6.550		6.000	2.017	1.306	0.568
1986	I	1.070	0.744	0.803	0.453	0.596	10.530	46.380	71.850	79.540	12.200	3.030	1.800
	II	0.568	0.348	0.568	0.227	0.466	26.030	58.580	78.780	88.040	23.390	1.738	1.195
	III	0.272	0.272	0.150	0.134	0.134	0.272	5.480	9.510	7.120	1.672	0.681	0.405
1987	I	1.040	0.762	2.160	1.210	7.380	23.110	53.000	76.050	24.500	18.470	4.300	2.270

	II	0.435	0.376	2.559	0.805	13.310	37.460	61.960	69.560	26.640	68.520	3.084	1.249
	III	0.348	0.272	0.272	0.296	0.206	2.993	3.562	11.320	3.765	2.397	1.089	0.604
1988	I	1.110						52.030					3.720
	II	0.499						57.630					1.390
	III	0.322						0.989					3.720
1989	I	1.625	0.661	1.603	2.459	1.044	4.960	42.388	48.809	29.950	6.194	0.636	6.864
	II	0.805	0.376	1.608	2.728	0.568	9.861	48.213	59.058	62.947	16.401	0.941	11.324
	III	0.435	0.206	0.206	0.322	0.272	0.376	4.870	9.004	3.366	0.405	0.104	0.104
1990	I	0.395	1.163	0.623	1.151	0.982	3.645	50.994	57.786	56.841	11.539	3.691	1.748
	II	0.568	1.195	0.808	1.112	0.956	3.518	45.573	50.474	52.839	8.568	2.002	1.112
	III	0.057	0.031	0.000	0.261	0.061	0.093	2.201	11.612	8.249	2.201	0.956	0.364
1991	I	1.311	0.570	1.390	5.257	2.342	11.746	68.209	119.718	85.449	12.213	3.388	2.458
	II	1.111	0.419	0.880	55.846	6.718	28.492	55.846	96.808	70.389	30.940	1.534	1.111
	III	0.213	0.061	0.310	0.061	0.169	0.419	0.093	14.377	6.137	1.534	1.111	0.669
1992	I	1.267	0.277	0.019	0.362	3.348	8.091	41.834	78.980	59.566	35.009	15.387	19.798
	II	0.669	0.260	0.093	0.955	10.906	12.876	45.574	54.336	41.394	48.436	23.130	44.446
	III	0.260	0.000	0.000	0.000	0.093	0.478	4.377	18.786	15.344	7.016	3.283	1.446
1993	I	2.733	1.880		4.975	13.654	27.796	84.814	131.490	136.304	73.298	26.758	20.245
	II	1.446	0.880		5.165	10.389	28.492	65.842	92.448	75.038	62.028	19.418	7.933
	III	0.808	0.669		0.539	1.809	5.030	10.218	29.707	29.952	13.246	8.090	7.317
1994	I	5.079	3.062	2.432	1.998	7.578	20.657	57.558	93.477	53.821	10.073	4.182	2.492
	II	2.542	1.620	0.995	1.404	9.754	13.117	35.769	51.111	52.285	8.316	2.186	1.299
	III	1.620	0.995	0.803	0.534	0.369	2.664	6.187	24.929	6.482	2.303	1.095	0.621
1995	I				4.199	9.160			45.265	39.087	12.958	6.497	4.421
	II				6.334	16.137			38.167	27.557	14.344	4.482	3.809
	III				0.898	1.196			9.430	5.462	2.303	1.196	0.369
1996	I		0.919	2.164	5.082			51.162	51.800	42.148	19.745	7.378	7.411
	II		0.621	2.303	11.915			44.195	35.985	35.336	16.319	6.631	4.075
	III		0.219	0.152	0.621			4.075	12.427	6.040	3.677	1.842	2.070
1997	I	6.469	5.028	5.040	6.638	8.612	11.033	39.887	49.830	28.850	16.099	18.113	13.760
	II	2.787	3.289	3.289	10.080	15.775	9.430	42.389	51.346	34.690	20.811	28.377	22.166

	III	2.186	1.730	1.404	1.196	1.620	0.000	5.177	0.000	6.040	2.664	2.303	2.542
1998	I	2.894	2.019	1.339	1.120	1.672	2.187	24.305	61.297	64.303	57.890	8.711	5.466
	II	3.259	4.361	0.580	3.041	2.349	4.099	31.681	133.519	111.320	107.373	9.816	2.349
	III	0.543	0.408	0.408	0.186	0.186	0.227	2.632	7.949	3.972	2.349	2.259	1.689
1999	I	5.029	3.316	2.537	2.211	3.028	7.956	34.442	51.625	28.838	22.221	6.158	3.115
	II	2.731	1.616	1.158	0.887	1.841	24.680	50.002	48.200	38.237	29.227	5.210	1.544
	III	1.689	1.158	0.791	0.838	0.838	1.158	3.259	9.167	4.634	3.372	1.279	0.746
2000	I	1.367	0.727	0.456	1.130	1.288	5.464	25.667	48.138				
	II	0.660	0.378	0.249	1.408	1.044	5.669	30.550	63.784				
	III	0.378	0.227	0.135	0.168	0.168	0.378	4.099	6.490				
2001	I	0.914	0.250	-	-	-	-	33.934	66.312	19.280	5.579	1.899	0.550
	II	0.507	0.168	-	-	-	-	56.306	59.633	32.372	5.360	1.408	0.322
	III	0.186	0.071	-	-	-	-	3.972	5.513	1.841	1.044	0.378	0.135
2002	I	0.789	0.177	0.610	0.447	0.122							0.550
	II	1.689	0.120	0.838	0.660	0.249							0.322
	III	0.093	0.036	0.052	0.019	0.019							0.135
2003	I	4.257	3.169	4.625	3.362	0.860	8.615	51.581	91.761	28.152	4.523	21.437	26.944
	II	2.349	2.171	3.041	3.149	0.938	21.308	88.941	170.994	27.939	9.167	14.036	14.036
	III	1.408	1.044	1.218	0.746	0.000	0.000	1.764	2.731	2.535	0.272	0.168	2.349
2004	I	6.082	3.002	4.390	6.417	5.217	14.565	42.862	62.809	34.564	21.380	10.369	7.694
	II	5.210	1.279	2.171	5.828	5.062	28.364	54.365	128.005	64.493	12.958	5.062	3.372
	III	1.218	1.044	1.218	1.475	1.158	1.544	4.634	7.197	5.828	4.361	3.149	2.535
2005	I	5.985	4.155	6.909	4.779	5.359	7.677	45.646	65.791	89.665	23.049	10.411	7.445
	II	2.731	1.920	5.062	3.149	3.604	8.749	25.471	134.639	168.401	26.278	5.360	3.259
	III	1.920	1.544	1.544	1.544	1.343	1.343	8.749	9.167	10.039	5.062	3.149	2.535

Where

I. MONTHLY RUNOFF IN MILLION M^3 II. MAXIMUM DISCHARGE IN M^3/sec III. MINIMUM DISCHARGE IN M^3/sec	MP = MOMENTARY PEAK IN M^3/sec MMD = MAX.MEAN DAILY IN M^3/sec

Appendix B: Generator product details

Table B.1. Generator product details for Fujian Mindu Motor Co., Ltd

Packing	Wooden Case
Model NO.	STC
Standard:	CE, ISO9001
Productivity	10000sets Per Month
Unit Price/Payment	FOB Fuzhou USD20/Set-USD500/Set
HS Code	8501610000
Trademark	Mindu or OEM
Origin	Fuan City ,Fujian Province,China
Min. Order	10sets
Transportation	The Same as Contract
Output Power	3kw-100kw
Rated Voltage	430v/230v
Frequency	50Hz/60Hz
Speed	1500rpm/1800rpm
Insulation Class	B
Export Markets	North& South America, Eastern Europe, Southeast& Eastern Asia, Africa, Oceania, Mid East, Western Europe

Product Description

Three phase AC synchronous generators; Alternators

1. Main specification

   3kw-100kw 50Hz/60Hz
   230v/400v, or 220/380v
   Insulation class: B

2. Optional
   - ❖ Output choice: 80%out put power or 100% out put power are both available, depends on the customer's requirements
   - ❖ Materials choice: 100%cooper wire, 100% aluminum, or 50% cooper wire&50% aluminum.
   - ❖ Terminal switch box choice: Internal wiring connecting or outside wiring connecting.
3. Usage

The ST generators are to be used in town, the countryside, mountainous and pasture lands as a electric power source for lighting purpose. It can also be as a reserved power source for emergent case.

4. Advantage

- Steady speed.
- Without any starting devices the generator can directly start an unload squirrel cage induction motor.
- When the revolution slip of prime mover is 3% and load varies in the range of 0-100% cosj1.0-0.8 they give good constant voltage.

Table B.2. Product description for Fujian Mindu Motor Co., Ltd generators

Type	Output(kw)	Output(kva)	Current (A)	Rated voltage	Pole Number	Speed (r.p.m)
STC-3	3	3.8	5.4	400	4	1500
STC-5	5	6.3	9			
STC-7.5	7.5	9.4	13.5			
STC-10	10	12.5	18.1			
STC-12	12	15	21.7			
ST-15	15	18.8	27.1			
STC-20	20	25	36.1			
STC-30	30	37.5	54.1			
STC-40	40	50	72.2			
STC-50	50	62.5	90.2			
STC-64	64	80	115.6			
STC-75	75	93.8	135.5			

Quick Details

Table B.3. Quick detail for Marelli generators

Condition	New
Output Type	AC Three Phase
Brand Name	Marelli
Place of Origin	Italy

Packaging & Delivery

Packaging Detail: Depending on the size of the alternator. Common packed on skid with plastic cover.

Delivery Detail: Ex-stock or 16 ~ 18 work weeks depending on model.

Table B.4. Product description for Marelli generators

Description	Specification
Prime Mover	Shaft Alternators PTO / PTI / PTH mode
Duty	Continuous
Frame Size	160, 200, 225, 250, 315, 355, 400, 450, 500, 560, and 630
Model No.	iSA4, iSB4, iSC4, iMA4, iMB4, iLA4 and iLB4.
No. of Poles	4
Gen. Output [kVA]	14.8 to 3735
Gen. Output [kW]	Upon request
Voltage [V]	380 to 690
Frequency [Hz]	50 / 60
Speed [rpm]	1500 / 1800
Power factor	0,8
Ins. Class/Temp. Class	H/F
Method of cooling	IC01 with SOLAR inlet filter. IC81With double pipe air to fresh water heat exchanger.
Over speed	1,2 x nominal speed
Protection degree	IP23 / IP55 upon request
Amb. Temp[^oC]	50
Bearings	Ball or Roller Sleeve bearing from 400 to 700 frame sizes are available on request.
Mounting	B3 - Single or Double bearing
Classification	ABS, DNV, RINA, BV, LR etc. upon request.
Standards	IEC 600334-1, CEI EN 60034-1, BS4999-5000, VDE 0530, NF 51-100, 111, OVEM-10, NEMA MG1.22, UL 1446, 1004B and "CE" mark.

Appendix C: Penstock diameter selection graphs

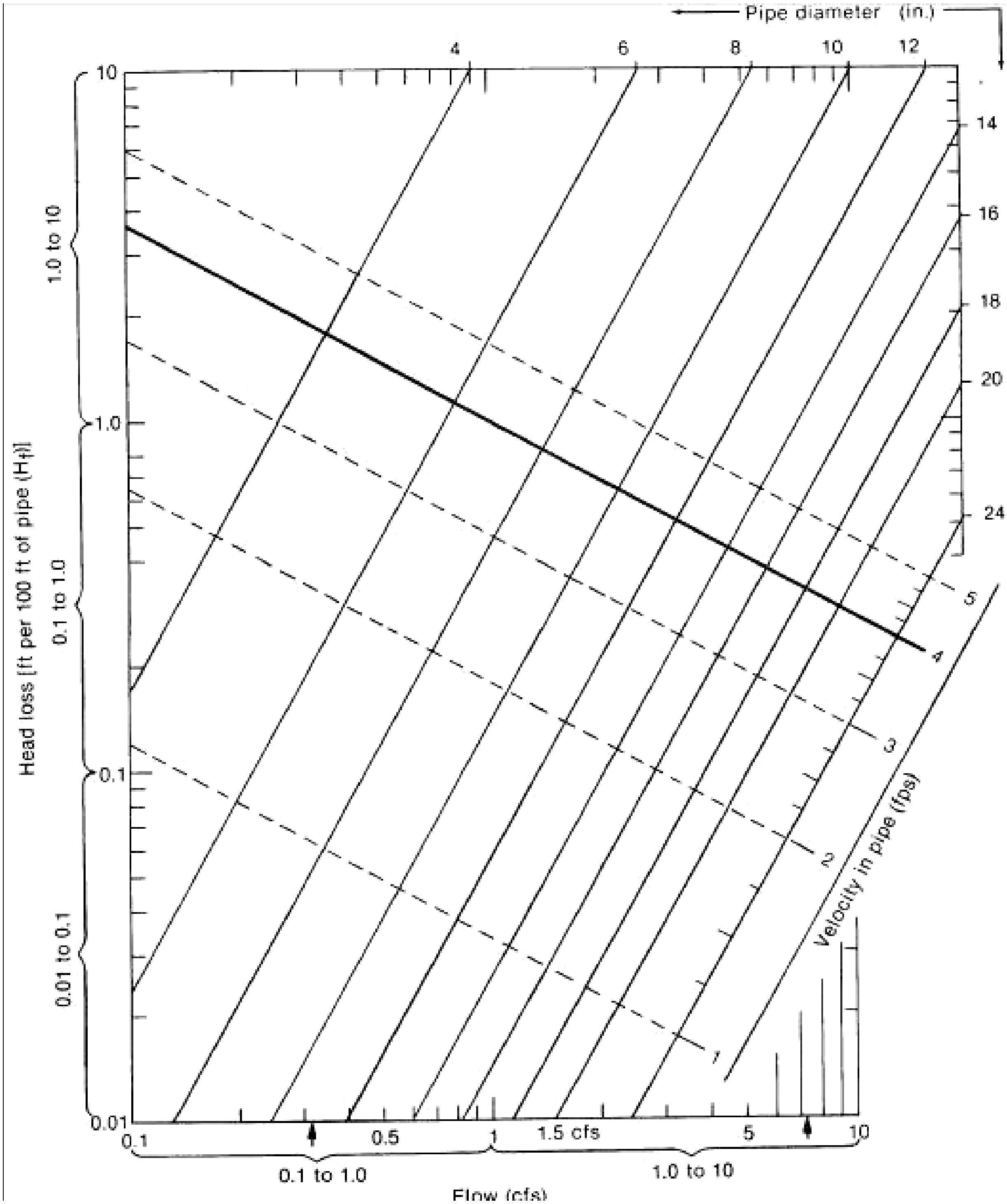

Figure C.1. Penstock diameter selection graph1

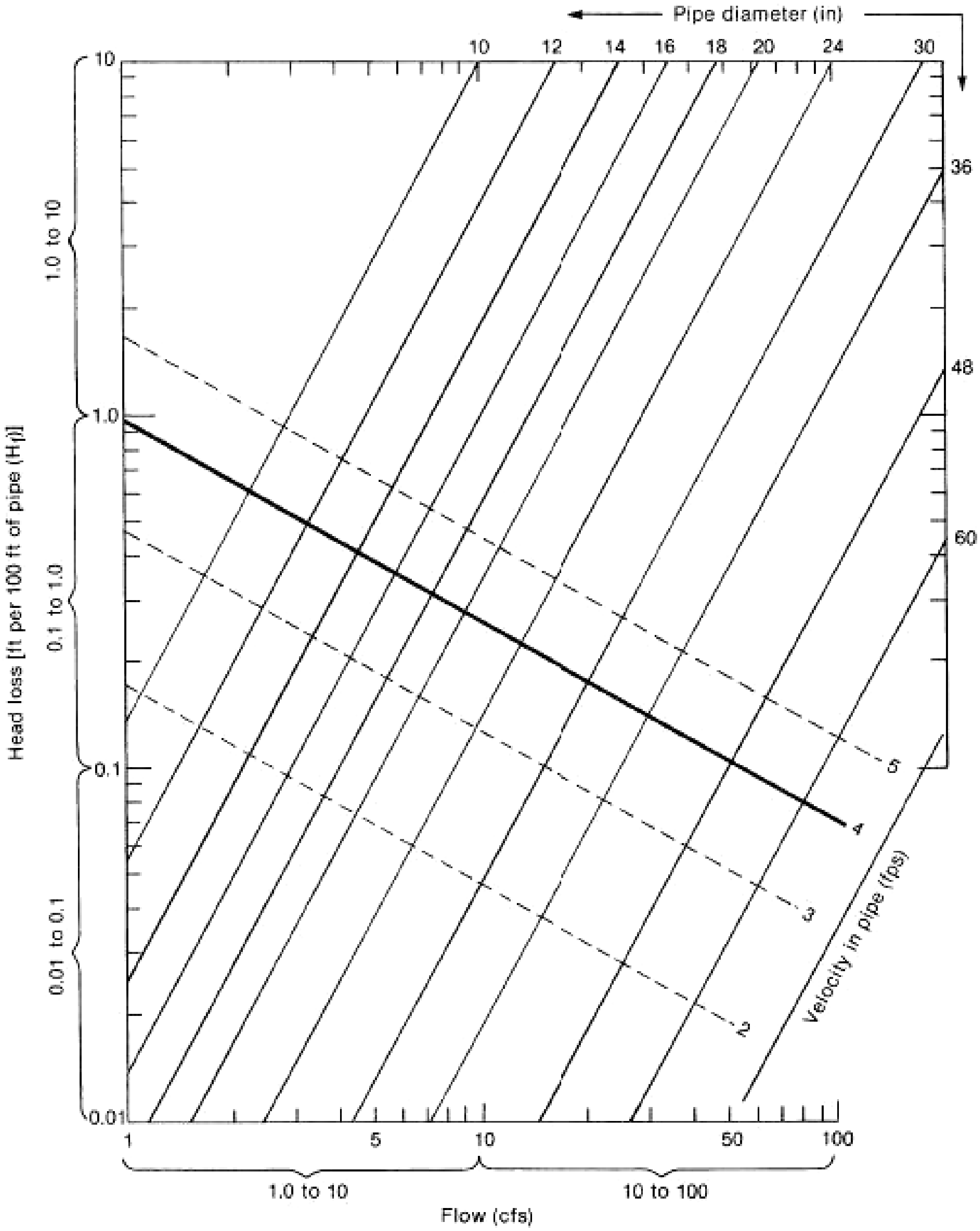

Figure C.2. Penstock diameter selection grapgh2

Table C.3. Penstock material sizes and their variability

Pipe size →	4 in.			6 in.			8 in.			10 in.			12 in.		
Pipe material ↓	t_W	P_R	S_A	t_W	P_R	S_A	t_W	P_R	S_A	t_W	P_R	S_A	t_W	P_R	S_A
Steel	0.24	1400	57	0.28	1200	56	0.25	800	54	0.25	640	52	0.25	540	51
							0.32	1100	56	0.37	1030	55	1.38	890	54
													0.41	1000	55
PVC (polyvinyl chloride)	0.11	100	12	0.16	100	12	0.21	100	12	0.26	100	12	0.31	100	12
	0.17	160	15	0.25	160	15	0.33	160	15	0.41	160	15	0.49	160	15
	0.26	250	18	0.39	250	18	0.51	250	18	0.63	250	18	0.75	250	18
PE (polyethylene)	0.265	100	9	0.39	100	9	0.51	100	9	0.63	100	9	0.75	100	9
	0.41	160	11	0.60	160	11	0.78	160	11	0.98	160	11	1.16	160	11
	0.62	250	13	0.91	250	13	1.18	250	13	1.47	250	13	1.75	250	13
A-C (transite—asbestos cement)	0.32	100	41	0.46	100	41	0.56	100	40	0.62	100	39	0.72	100	38
	0.41	150	44	0.53	150	43	0.63	150	41	0.83	150	42	0.96	150	41
	0.41	200	44	0.66	200	45	0.81	200	45	1.02	200	45	1.16	200	44
FRP (fiber reinforced epoxy)	0.07	225	17	1.11	250	16	0.14	225	16	0.18	225	15	0.21	225	15

Pipe size →	14 in.			16 in.			18 in.			20 in.			24 in.[a]		
Pipe material ↓	t_W	P_R	S_A	t_W	P_R	S_A	t_W	P_R	S_A	t_W	P_R	S_A	t_W	P_R	S_A
Steel	0.25	490	50	0.25	430	49	0.25	380	48	0.25	340	47	0.25	280	45
	0.38	810	54	0.38	710	52	0.38	630	52	0.38	570	51	0.38	470	
	0.44	970	55	0.50	990	55	0.50	880	54	0.50	790	53	0.50	660	
										0.59	950	54	0.69	940	54
PVC (polyvinyl chloride)	← PVC pressure pipe not available →														
PE (polyethylene)	0.82	100	9	0.94	100	9	1.06	100	9	1.12	100	9	1.41	100	9
	1.27	160	11	1.46	160	11	1.64	160	11	1.81	160	11			
	1.92	250	13												
A-C (transite—asbestos cement)	0.74	100	38	0.83	100	37	0.95	100	37	1.06	100	37	1.24	100	
	1.11	150	41	1.23	150	41	1.47	150	42	1.64	150	42	1.98	150	
	1.38	200	44	1.57	200	44	2.09	200	45	1.98	200	45	2.81	200	45
FRP (fiber reinforced epoxy)	0.26	225	15	0.29	225	15									

DECLARATION

I, the undersigned, declare that this thesis is my original work, has not been presented for a degree in this or other universities, all sources of materials used for this thesis work have been fully acknowledged.

All examiners‟ comments are duly incorporated.

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Place: Addis Ababa Institute of Technology, Addis Ababa University, Addis Ababa

Date of Submission:_____________, 2011

This thesis has been submitted for examination with my approval as a university advisor.

Dr.Getachew Bekele Signature:_____________

Advisor