Technical Specifications of Micro Hydropower System Design and its Implementation

Feasibility Analysis and Design of Lamaya Khola Micro Hydro Power Plant

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Abstract:
The purpose of this thesis is to first conduct a literature review regarding the technical specifications and design parametres required to design a working Micro Hydro Power System MHS (Micro Hydropower System). After review of the theory and principles of Micro Hydro System design; these principles are applied to the real case of Lamaya Khola Micro Hydro Project in Pangrang Village Development Committee (VDC) of Nepal. The field data required to design the civil components of the micro hydro project were derived from secondary data sources such as the study conducted by the village development committe as well as other independent project surveys. The micro hydro designed in this thesis was of "run-of-the river" type. Similarly, system components designed in this thesis are intake structure, headrace canal to divert the water from the source, forebay tank, sedimentation basin and the penstock assembly. Owing to the complexity and lengthy process of designing all of the system components; only these specific civil structures are designed in this study. Design of other powerhouse and distribution components is beyond the scope of this study and so are merely selected based on the design criteria from the literature. Despite these shortcomings, the system designed is plausible, applicable and principally sound in the real life case of Lamaya Khola Micro Hydro Project in Nepal. Wherever possible, the system components that are designed are also illustrated with AutoCad. The contribution of this thesis is its practical implementation of the principle of micro hydro system design in a real life situation and specific considerations in the case of Nepal, which is topographically very different from majority of the other countries.

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ACRONYMS AND ABBREVIATIONS

HPP	Hydro Power Plant
Kg	Kilogram
Km	Kilometre
kW	Kilowatt
l	Litre
m	Metre
MHP	Micro Hydro Project
MHS	Micro Hydropower System
MW	Megawatt
s	Seconds
PPA	Power Purchase Agreements
VDC	Village Development Committee

1 INTRODUCTION

1.1 Overview of Hydropower Systems and their Classification

In a hydropower system, the energy present in water is converted into mechanical or electrical energy by the use of hydropower plant. Generic hydro power systems can be categorized in many different ways. Some of the methods of classification are based on how the electricity is generated by the plant, what kind of grid system is utilized for the distribution of electricity, the type of load capacity and the type of storage used by the system. (Pandey, 2006)

1.1.1 Power generation capacity

Although various categorizations exist based on different locality and nationality, the generally accepted classification of hydro power plants based on the ability to generate power is provided in Table 1.

Table 1: Common Classification of Hydro Power Plants (HPP) based on capacity generation (Bhattarai, 2005)

Power Generation Capacity (Watts)	Type of Hydro Power Plant
<100 kW	Micro
100-1000 kW	Mini
1MW-10 MW	Small
10MW-300 MW	Medium
>300 MW	Large

Generally, hydropower plants (HPP) generating less than 100kW of electricity are termed as micro HPP; those generating 100 to 1000 kW are termed as mini HPP; those anywhere between 1 MW to 10 MW are termed as small HPP; between 10 MW to 300 MW are termed as medium HPP and those with the power generating greater than 300

MW are termed as large HPP. Although variations in definitions exist; this is the most commonly accepted definition (Bhattarai, 2005).

1.1.2 Types of Storage

Based on the type of storage used in hydro power plants they can be classified into storage type or "run-of-the- river" type. The major difference between these types of hydro power plants is that in the former, a dam is constructed to act as reservoir of water sources and has the ability to continuously supply the water in undulating manner. However in the latter, "run-of-the river" type, it is constructed by directing the water source to the turbine and the water source may vary according to seasons. Storage type are used generally in small to large hydropower plants whereas "run-of-the river" type are more common in micro, mini and small hydro power plants (Bhattarai, 2005). In this study, “run-of-the-river type” of micro hydro plant is designed at a later stage.

1.1.3 Types of Grid System

Based on the type of grid system, hydro power plants can be classified into local grid and extensive grid systems (Hydraulic Energy Program et al., 2004) . In the local grid system, the electricity is generated and distributed only for the small locality, without use of any sophisticated electromechanical distribution systems. In contrast, in an extensive grid system, the electricity generated by the HPP (Hydro Power Plant) is loaded in a form of extensive grid such as the national grid system. Generally larger HPP (Hydro Power Plant) are of these types but in this study, focus is paid to the local grid system. Similarly, based on the load of the distribution system HPP (Hydro Power Plant) have been classified into base load plant and peak load plant but it is beyond the scope of this study; as the system designed in this thesis does not consider these factors.

1.2 General Principle of Micro Hydro Power system (MHS)

Micro hydro power plants are designed to generate electrical or mechanical power based on the demand for energy of the surrounding locality. In a typical MHS (Micro Hydropower System) the water from the source is diverted by weir through an opening intake into a canal (Fox, 2004) . A settling basin might sometimes be used to sediment the foreign particles from the water. The canal is designed along the contours of the landscape available so as to preserve the elevation of the diverted water. The water then enters the fore-bay tank and passes through the penstock pipes which are connected at a lower elevation level to the turbine. The turning shaft of the turbine is then used to operate and generate electricity (Simoes, 2004). The machinery or appliances which are energized by the hydro scheme are called the load. A typical MHS (Micro Hydropower System) layout is provided in Figure 1. The detailed description of the principal components will be given at a later stage.

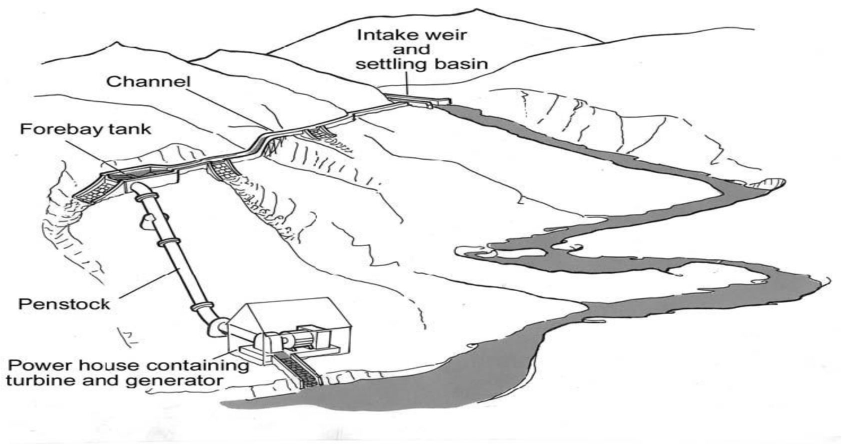

Figure 1: A typical MHS (Micro Hydropower System) configuration (Pandey B. , 2006)

1.3 Micro Hydro Systems

The focus of this study is to design a Micro Hydro Power System (MHS) at a later stage. Therefore, after classifying the generic hydro power systems in section 1.1, this section will be focused on describing different types of MHS (Micro Hydropower System). The factors that affect the choice of MHS (Micro Hydropower System) are the needed capacity from the project, the anticipated demand for power from the locality and also the profile of the project. The major factor that needs to be assessed beforehand are whether the MHS (Micro Hydropower System) will be connected to some grid system for delivering electricity or just be a standalone system which is not connected to any electrical grid system. When the MHS (Micro Hydropower System) is planned to be at a remote site, it could either be battery based or AC direct system. However, if it is to be connected to the electrical grid then AC direct systems are more appropriate.

(Hydraulic Energy Program et al., 2004) The final choice will ultimately be influenced by many other factors which are going to be explained in the following section.

1.3.1 AC direct Systems

This type of MHS (Micro Hydropower System) is designed to supply the load directly, as it does not use battery as storage. This is the most common system that can be found in normal use and is most suitable for grid connected sites and remote standalone sites. The smallest available fully integrated system of this kind is 200W which can work with the head (this concept will also be explained later) as low as 1 meter (Hydraulic Energy Program et al., 2004). When the larger unites are required due to capacity or site profile, they are manufactured and assembled accordingly.

1.3.2 Grid Connected

When there is already a grid supply available it is possible to still install a system and obtain electrical power from both the MHS (Micro Hydropower System) and the grid supply. It is possible to supply the excess power generated from MHS (Micro Hydropower System) to the grid through “net metering” (Greacen, 2004). A single meter measures the electricity purchased from the utility and turns backward when the small power producer feeds electricity into the grid. The net-meter measurement determines the amount of electricity charged to the user. Net-metering programs are in various stages of development in British Columbia, Alberta, Manitoba and Ontario. Each utility has its own policy for grid connections.

1.3.3 Hybrid Systems

Hybrid system is a system where different sources of power are used to generate electricity. It could include any combinations of wind, photovoltaic system and MHS (Micro Hydropower System) There are several advantages of hybrid systems over single type of system, because it is possible to offset the low peak period of one source by high peak period of the alternate source. For example, when wind and photovoltaic sources are installed together, wind speeds might be low in summer but then sun shines brightest and longest at the same period of time, making it possible to generate power when it is required (Fennell, 2011).

1.4 General Status of Micro Hydro Power Plants in Nepal

Since the purpose of this study is to provide the theoretical background of MHS (Micro Hydropower System) and later utilize this knowledge to design a MHS (Micro Hydropower System) system in a locality in Nepal, it is relevant to provide reader with the general information on MHS (Micro Hydropower System) status in Nepal. To the time of writing this thesis, forty-six small hydropower plants, have been established including six plants under construction. Most of the plants are in operation in isolated mode, and only a few of them are connected to the national grid system.

As of June 2007 a total of 33 PPAs (Power Purchase Agreements) for Small Hydro power projects have been signed with a total installed capacity of 128 MW. Out of these, nine projects with total installed capacity of 25 MW are in operation and 6 projects with total installed capacity of 12 MW are under construction. Now, a total of 36 applications for PPA are under various stages of study. Power evacuation has emerged as the most important issue; impending speedy conclusion of PPA. The detailed current status of micro hydro projects in Nepal is provided in Appendix 2. (Bhattarai, 2005)

1.5 Research Description

1.5.1 Research Objectives and Questions

The objective of the study is to analyze the factors that are essential in designing a micro hydro system and to utilize this information to practically design a MHS (Micro Hydropower System) in a specific case of Nepal, which will be described in the method section. The focus of the study is in designing technical components of the MHS (Micro Hydropower System) rather than the other factors such as the distribution of electricity after the completion of the project and other social, financial and environmental implication. Therefore the research questions could be formulated as follows:

- What are the technical specifications of MHS (Micro Hydropower System)?

- How can these specifications help to practically design a MHS (Micro Hydropower System) in a real life situation?
- What are the difficulties in practical implementation?

1.5.2 Research Design

Theoretical background at the beginning of the study will provide information regarding the principle that is used to generate electricity by using the water from the water source. The principle components of the MHS (Micro Hydropower System) are also described to facilitate the understanding of the system components and to explain how MHP (Micro Hydro Project) works in an interconnected manner. The technical specifications or design parameters that are required to design MHS (Micro Hydropower System) will be also derived from literature review and explored further; because these are the most relevant information that are conducive to the design and implementation of MHS (Micro Hydropower System) in a practical context.

The method section of the study starts with the general description of the locality where the MHS (Micro Hydropower System) is to be designed. Since it was not possible to conduct on-field survey in the exact locality, the relevant data already published by the organization (Pangrang Village Development Committee) responsible for constructing the MHS (Micro Hydropower System), will be utilized as secondary source of data in designing the MHS (Micro Hydropower System) in the site. The data published by Pangrang V.D.C., includes several parameters that are very essential for the design of MHS (Micro Hydropower System) which contains such factors as measurement of head, flow of water, design discharge and so on. All of these concepts are explained while describing the method section.

Finally, based on the theoretical principles of MHS (Micro Hydropower System) and design parameters, coupled with onsite survey data and other calculations; a practical MHS (Micro Hydropower System) will be designed and implemented. Wherever possible, using the dimensions derived from the calculations, the components will be designed and shown figuratively using AutoCAD software. Lastly, the experience in system design in the specific context will lead to recommendations and implications for future system design in a similar context.

The general research design or the structure of the study as described in this section are summarized and illustrated in Figure 2.

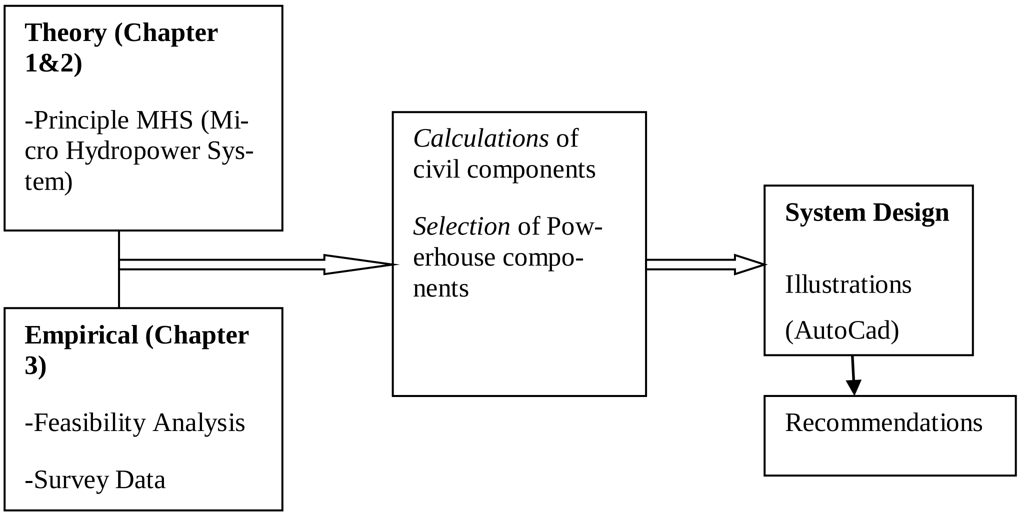

Figure 2: Research Design of the study

1.5.3 Limitations of the Study

The research is limited in several fronts. Firstly, the focus of the study is on MHS (Micro Hydropower System) rather than small, medium or large HPP each of which require totally different approaches to system design. Secondly, although the investment and design of MHS (Micro Hydropower System) is primarily for generation of electricity for consumption, the study is focused only on the technical specification and design of MHS (Micro Hydropower System) rather than designing network for electricity distribution which again requires a totally different perspective in system design. Thirdly, the actual implementation process of MHS (Micro Hydropower System) is inevitably affected by the local political, legal, social and economic environment, which is not the focus of this study; rather it is concerned more with the technical design and implementation of design in a certain context. Often the choice of components and geography are limited by financial considerations, although they might not be implemented with a strict financial rationale, therefore consideration of financial factors in choices of components and geography are omitted from this study. The study is further limited because it is beyond the scope of a small study such as this to explain all the components that are required to develop MHP (Micro Hydro Project). Additionally, apart from the system components to be discussed in the theory section, there could be additional civil components dependent on the geography of the location, which is beyond the scope of this study.

2 LITERATURE REVIEW

2.1 Principal Components of a Simplified MHS (Micro Hydro-power System)

A typical MHS (Micro Hydropower System) is arranged as depicted in Figure 2.

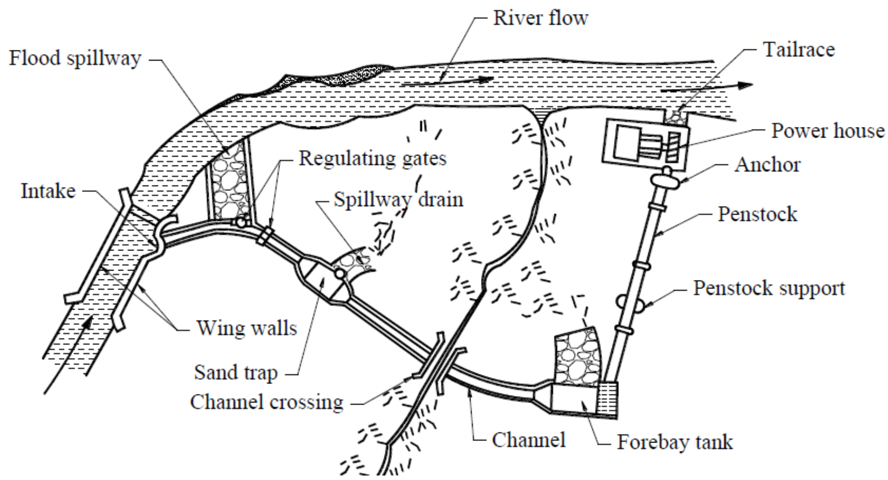

Figure 3: General Layout of the MHP (Micro Hydro Project) and its principal components (Pandey B. , 2006)

The principal components that are used in the MHS (Micro Hydropower System) could be further classified into civil components, powerhouse components and transmission and distribution networks. All of these components are again elaborated in the following sections.

2.1.1 Civil Work Components

This section describes the components of the typical MHS (Micro Hydropower System) that could be classified as civil works components. The civil components described in this section are those major components such as the intake, headrace canal, de- Sanding basin, spillway, forebay tank, penstock pipes and tailrace (BPC Hydroconsult, 2006).

- Intake

  Intake is the primary means of conveyance of water from the source of water in required quantity towards the waterways of HPP (Hydro Power Project). Intake could be of side intake type or the bottom intake type. Usually, trash racks have to be placed at the intake which acts as the filter to prevent large water born objects to enter the waterway of the MHP (Micro Hydro Project) (Harper, December 2011).

- Headrace Canal

  Once the water enters through the intake, the headrace canal conveys the water to the forebay. Sometimes, pipes can also be used in place of the canals. The materials to be used in constructing the canal depends upon the geographical condition of the site and other obvious factors such as the availability of labor and materials. Most usual types of canal are built from combination of cement and mortar, only soil, mixture of stone and mud, mixture of stone masonry with cement and other different types of possible combinations (Pandey V. , 2011). When pipes are used, they are generally of HDPE (High Density Polythene) types. The length of the headrace canal can be anywhere from few meters to over a kilometer long. The most important thing to consider while constructing head race canal is to make the slope of the canal only slightly elevated because higher slope can lead to higher velocity of water which can then cause erosion in the headrace canal surface.

- Settling Basin

  Specially, in the case of Nepal, rivers generally carry high amount of sediments due to erosion activities in hills and mountains. In order to reduce the sediment density, which has negative impact to other components of the MHS (Micro Hydropower System) de-sanding basins are used to capture sediments by letting the particles settle by reducing the speed of the water and clearing them out before they enter the canal. Therefore, they are usually built at the head of the canal. They are equipped with gate valves for flushing the settled undesirable sediments. De sanding basin is capable of settling particles above 0.2-0.3 mm of size (Harvey, Micro Hydro Design Manual, 1983). Figure 3 shows the typical de-sanding basin.

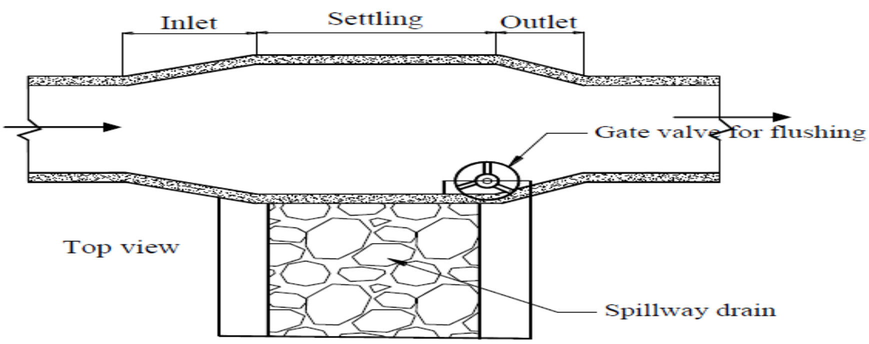

Figure 4: A typical De-Sanding Basin (Harvey, Micro Hydro Design Manual, 1983)

- Spillway

  Specially in the case of Nepal, where flooding of water source is typical, spillways need to be designed to remove the excess water due to floods, in order to minimize the adverse effects to the other components of the MHS (Micro Hydropower System). Spillways are often constructed in de-sanding basin and the forebay, from which the excess water is safely diverted to the water source. Figure 4 shows the typical method of construction of the spillway

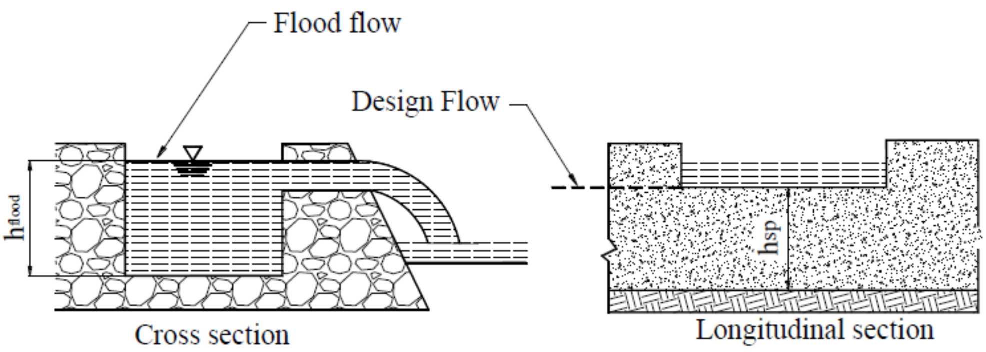

Figure 5: A typical spillway in MHS (Micro Hydropower System) (Pandey B. , 2006)

- Forebay Tank

  Forebay tank is basically a pool at the end of headrace canal from which the penstock pipe draws the water. The main purpose of the forebay is to reduce entry of air into the penstock pipe, which in turn could cause cavitation (explosion of the trapped air bublles under high pressure) of both penstock pipes and the turbine (Masters, 2004). It is also necessary to determine the water level at the forebay because operational head of the microhydro power plant is determined through this factor. A forebay again requires two sets of additional construction. As the water speed is lowered at the forebay, it can cause sedimentation of particles, which requires the construction of spillway as mentioned before. Similarly, installation of trash racks to filter the fine sediments might be required before the water from the forebay gets inside the penstock pipes. Figure 5 illustrates a typical forebay tank in MHS (Micro Hydropower System).

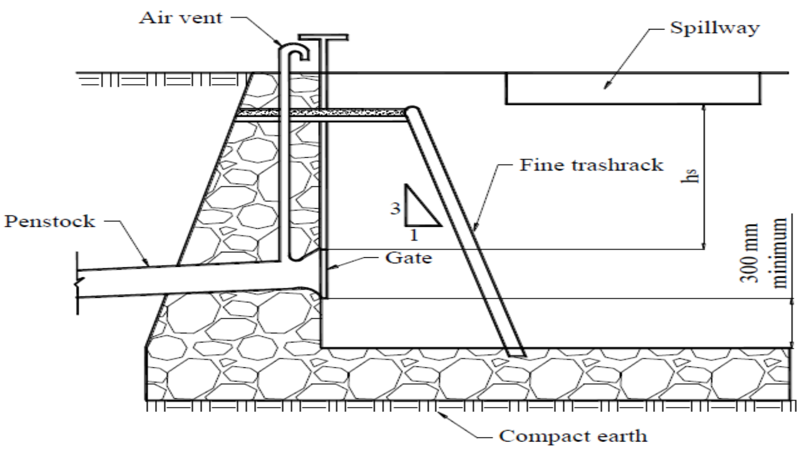

Figure 6: Design of a Typical Forebay in MHS (Micro Hydropower System)

(Sanchez & Rodriguez, June 2011)

- Penstock Pipes

Penstock pipes are basically close conduct pipes that helps to convey the water from the forebay tank to the turbine. The materials used in penstock are usually steel, HDPE (High Density Polythene) and increasingly PVC (Poly Vinyl Chloride). It is one of the most important components of the MHS (Micro Hydropower System) because it is at this point that the potential energy of the water is converted into kinetic energy. The velocity of water at the penstock is typically 3m/s and is often located at a slope over 45 degrees (Sanchez & Rodriguez, June 2011). Due to the risk of contraction and expansion of penstock pipes due to fluctuation in seasonal temperature, sliding type of expansion joints are placed between two consecutive pipe lengths. Anchor block, which is basically a mass of concrete fixed into the ground, is used to restrain the penstock from movement in undesirable directions. Figure 6 shows the configuration of penstock pipes in typical MHS (Micro Hydropower System).

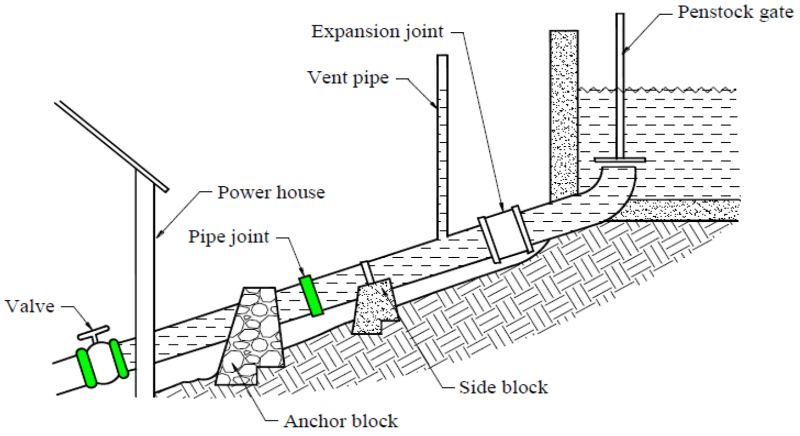

Figure 7: Components of the penstock assembly (Pandey B. , 2006)

- Tailrace

  Tailrace is very similar to headrace canal described previously in this section. The only difference with that of the headrace canal is that it is situated at the end of the civil components and is used to convey the water back to the source after use in the micro hydro plant.

2.1.2 Powerhouse Components

It is at this stage that the conversion of mechanical energy of water into electrical energy takes place. Basically, powerhouse consists of electro-mechanical equipment such as turbines, generator and drive systems which will be explored further in the following sections

- Turbine

  In a MHS (Micro Hydropower System) hydraulic turbine is the primary component which converts the energy of the flowing water into mechanical energy through the rotation of the runner. The choice of particular turbine depends upon technical parameters such as design head and discharge at which the turbine is to operate as well as other practical considerations such as the availability and cost of maintenance personnel. The optimum speed of the turbine is the particular speed of its rotor at which the turbine performs its best. The turbine needs to operate at this optimum speed in order to get maximum possible output at all loading conditions. Based on its functionality, turbines have been generally differentiated into two groups: impulse turbine and reaction turbine. (Simoes, 2004)

Under impulse turbine; pelton, turgo and cross-flow turbine are included. They are different from the reaction turbine in that their rotors are not submerged in the water but are allowed to rotate freely in the atmospheric pressure (Khatri & Uprety, 2002). The bucket that is mounted on the periphery of the runner is affected by the impulse force of the high velocity water which rotates the runner and shaft of the turbine. When the water passes through its free jet type nozzle, the pressure energy in the water is converted into kinetic energy.

Reaction turbine are different from impulse turbine in that the turbine's rotor is submerged in water and the water acting on the wheel is greater than the atmospheric pressure. It derives its name partly because it runs by the reaction force of the exiting water. At the outlet, a draft tube is fitted in the turbine. The runner utilizes both the potential as well as kinetic energy of the water (Fennell, 2011).

- Generators

  Although this study is not overly concerned with the selection, and uses of generators in the MHS (Micro Hydropower System) it is, however, relevant to describe the basic types of generators and how they are integrated in the MHS (Micro Hydropower System). There are basically two types of generators in use for hydroelectricity generation; either synchronous or induction generators. Synchronous generators are the primary types of generators which are used extensively in large scale power generation. When the power output levels are generally low (less than10 MW), induction generators are extensively used. Induction generators are also the preferred type of generators in MHP (Micro Hydro Project) because they can operate at variable speeds with constant frequency, are available cheaply and requires less maintenance than the synchronous generators. Both of these generators have the possibility to be used connected to the grid or just standalone operation (Upadhayay, 2009).

- Drive systems

  The main purpose of the drive systems is to transmit the power from turbine to the generators at a stable voltage and frequency at a required direction and required speed. Like any normal drive systems, in a MHS (Micro Hydropower System) also, drive systems comprise of generator shaft, turbine shaft, bearings, couplings, gearboxes and belts and pulleys. The different types of drive systems common in MHS (Micro Hydropower System) are direct drive, “V” or wedge belts and pulleys, timing belt and sprocket pulley and gearbox drive systems. A direct drive system is one in which the turbine shaft is connected directly to the generator shaft. In contrast, “V” or wedge belts and pulleys are the most commonly used type of drive systems in MHS (Micro Hydropower System). However, in very small systems (less than 3 kW) where efficiency is critical, timing belt and sprocket pulley are commonly used. Gearboxes are suitable in large machine where drive belts are not efficient. Due to high maintenance and alignment costs of gearboxes, they are less frequently used in MHS (Micro Hydropower System). Figure 7 shows a typical direct coupled drive system.

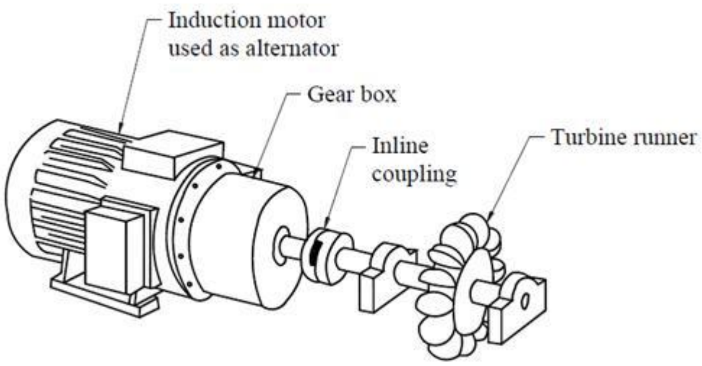

Figure 8: Direct Coupled Drive System (Chitrakar, 2004)

- Electrical Load Controllers

  All MHS (Micro Hydropower System) will have to have switchgear in order to separate the power flow when necessary and also to control the electrical power flow. There are several different kinds of switches used in an MHS (Micro Hydropower System) such as isolators which are manually operated, switch fuses which additionally can provide fuse for current limiting, MCCB (Molded case circuit breakers) which are used for protection from over current or short circuits and so on. The choice of electronic load controller is largely dependent upon the type of generator installed in MHS (Micro Hydropower System). For instance, when the induction generator is used in the MHS (Micro Hydropower System) it is necessary to install Induction generator controllers (IGC). Additionally, transmission network are also a major component of the MHS (Micro Hydropower System) if the output is fed into some other grid system; however, detailed description of electrical load controller is beyond the scope of this thesis as it is concerned primarily with the design of the civil works components.

2.2 Design Parameters of a MHS (Micro Hydropower System)

In this section, the design parameters that are needed to design a MHS (Micro Hydropower Systems) are reviewed. These parameters will be utilized at a later stage while designing the case micro hydro system.

2.2.1 Head Measurement

The head is defined as the vertical height in meters from the level where the water enters the penstock to the level where the water leaves the turbine housing. Altimeter is used to calculate the "head" during the field survey.

2.2.2 Measuring Water Flow Rate

The first step in determining the hydro power potential of a water source is to measure the flow rate. It has been defined as the quantity of water flowing past a point at a given time. There are several methods available to measure water flow rate such as salt dilution method, bucket method, weir method and so on. However, in mountainous region such as that of Nepal salt-dilution method of determining the head is the most common one. The salt dilution method is especially appropriate in this case because most of the water source used for the MHS (Micro Hydropower System) is small flow stream. Salt dilution method is thought to be accurate and quick in cases of shallow mountain streams such as in our case.

In this method, a pre-determined weight of pure dry salt is dissolved in a bucket of water. This water is then poured into the stream in certain predetermined location. Conductivity reading is then taken after every 5-10 seconds by immersing the conductivity meter 30-50 meters downstream near to the bed and center of the stream. The reading will show rise, reach a peak and then again return to normal over a period of time. Based on these the flow of the water (Q) is calculated and is given by:

Q (m^3/s) =mass of the salt (kg)/ [conversion factor (kg/m^3/ohm-litre) x area under the curve (ohm-l)] (Monition, 1984)

Here, the mass of the salt is known previously, the value of the conversion depends also upon the temperature and is given explicitly on the manual of the conductivity meter, and the curve referred to above is generated by plotting the change in conductivity with time. Based on this method, the flow (Q) of water source can be determined.

2.2.3 Measuring Potential Power and Energy

According to the Bernoulli energy equation, (Fox, 2004) energy in water is stored in terms of pressure energy, velocity energy and elevation energy. It can be further stated as:

Power (Energy/sec) = Pressure energy/sec + velocity energy/ sec + elevation energy/sec or,

$$P = \frac{p}{\rho g} + \frac{v^2}{2g} + z$$

Where,

P = electrical or mechanical power produced, W

ρ = density of water, kg/m^3

g = acceleration due to gravity, m/s^2

z = elevation of the point above the reference point

v = velocity energy of the water

The potential power that can be generated from the micro hydro power plant is often calculated from the survey of the site (Pandey B. , 2006).The difference in the energy is converted into useable energy by the hydropower plant when there is a difference between energy of the water. Therefore,

$$P = \frac{p}{\rho g} + \frac{v^2}{2g} + z$$

Or,

$P= (p/\rho g+v^2/2g+z)_{intake} - (p/\rho g + v^2/2g+z)_{exit}$

However, there will be some loss of power while the available water energy is converted by the hydropower plant. The actual power that can be generated from the given source of water is thus,

$$P=\rho.g.H.Q.\eta \qquad (1.1)$$

Where,

P = electrical or mechanical power produced, W

ρ = density of water, kg/m^3

g = acceleration due to gravity, m/s^2

H = elevation head of water, m

Q = flow rate of water, m^3/s

η = overall efficiency of MHs (Micro Hydropower system

It can be seen clearly from the equation that the power generated by the water available depends upon the flow rate of the water, elevation head (elevation difference between intake and exit of water), and gravitation force, density of water and efficiency of the hydropower system. The goal of the hydro power is to convert the available water energy into mechanical or electrical energy.

2.2.4 Construction of Intake Weir

Especially in situations like that of Nepal, it is very important that the water that enters through the intake is optimum both during the high river flow (monsoon) or the low river flow (summer) seasons. The weir may be of natural or of artificial construction. One important design parameter for the construction of intake weir is that its height should be kept at minimum but enough to channel the required flow of water. Similarly, most often natural construction such as boulders created intake are most appropriate as they minimize the entry of river borne materials and help to keep the level of the water flow at optimum.

There are several steps to construction of the intake weir. First of all, it is necessary to design the orifice for the side intake. Side intake is the most common type of structure used to divert the river flow towards the headrace channel. If the orifice is to be submerged then the design discharge or the flow of water needed for the MHS (Micro Hydropower System) from the side intake is calculated by the following equation (BPC Hydroconsult, 2006):

$$Q = A \cdot V = A \cdot C \cdot \sqrt{2 \cdot g \left(h_r - h_h \right)} \tag{2. 1}$$

Where,

Q = discharge through the orifice (m^3/s)

V = velocity through the orifice (m/s)

A = area of the orifice (m^2)

$h_r - h_h$ = difference between the river and the headrace canal water levels

C = coefficient of discharge of the orifice

The value of “C” or coefficient of discharge varies according to the materials used to develop the structure of the intake. Since, in this study the probable material used for the structure is going to be masonry orifice, the value of “C” is given to be 0.6. It can usually range from 0.6 to 0.8, when the structure is very meticulously finished. $[h_r - h_h]$ usually varies according to the discharge in the river. Similarly; the limit of velocity that is to enter from the side intake in masonry orifice is given as 3 m/s. For MHS (Micro Hydropower System) the recommended velocity is somewhere between 1.0 - 1.5 m/s (Monition, 1984).

2.2.5 Headrace Canal Design

The canal dimensions and cross section while designing the headrace canal are governed by various factor such as capacity, velocity of the water, slope of the side, head loss and seepage and the type of sediment disposition in the canal. The recommended side slopes while designing the headrace canal and the maximum headrace canal velocities are given in table 2. Since the type of canal that is feasible in Nepal is mostly stone masonry type, only the parameters for masonry canal are given in the table.

Table 2: Recommended side slopes, maximum headrace canals velocities and roughness coefficients of materials (BPC Hydroconsult, 2006)

Material used in the Canal	Side Slope(N=h/v)	Maximum recommended velocity for canals (V)	
		>0.3m depth	>1m depth
Stone masonry with mud mortar	0.5-1.0	1.0	1.0
Stone masonry with cement mortar	0-1.5	1.5	1.5
Roughness Coefficients for Masonry Canals			
Masonry Canals	Brickwork	Roughness coefficient "n"=0.015	
	Normal Masonry with cement mortar	*0.017*	
	Coarse rubble masonry	0.020	

The first step in designing a headrace canal is to choose the type of canal depending upon the conditions of the site where the MHP (Micro Hydro Project) is to be designed. As mentioned before, the design parameters for the MHS (Micro Hydropower System) to be designed in this case is limited to masonry types whose recommended parameters were given in Table 2. Based on this table, a suitable type of velocity (V) for a type of canal is chosen. The roughness coefficient (n) can also be determined from the Table 2 (same table) (BPC Hydroconsult, 2006). After the selection of appropriate type of canal structure, the next step is to calculate the cross sectional area of the headrace canal given by:

$A = Q / V$ (where Q is the design flow, as described section 2.2.2.) (3.1)

After determining the cross section area, side slope N is determined which is the ratio of the horizontal length of the headrace canal divided by the height of the side wall, that is

N= h/v (it can also be determined from Table 2. After all these parameters are determined the next step is to calculate the optimum height of the canal (H) by:

$$\chi = 2\sqrt{1 + N^2} - 2\cdot N \tag{3.2}$$

$$H = \sqrt{\frac{A}{(\chi + N)}} \tag{3.3}$$

Where χ is the factor used to optimize the shape of the canal. If the optimum canal shape is not possible given the condition of the site, other factors such as the width or the height of the canal should be chosen first based on the site conditions and then other dimensions calculated based on that measurement.

Followed by calculation of the optimum width of the canal bed (B) by:

$$B = \chi\cdot H \tag{3.4}$$

And finally the optimum width of the top of the canal (T) by:

$$T = B + (2\cdot H\cdot N) \tag{3. 5}$$

In order to make sure that the water flows in a stable and uniform manner, the velocity of the water should be less than 80% of the critical velocity V_c which is given by:

$$V_c = \sqrt{\frac{(A\cdot g)}{T}} \tag{3. 6}$$

As given before, "A" refers to cross sectional area and "T" refers to optimum width of the top of the canal. In case the canal velocity is greater than 0.8V, the calculations should be repeated in a hit and trial manner with lower velocity until it satisfies the condition. Based on all these information, the wetted perimeter (P) of the headrace canal can be calculated by using the following equation;

$$P = B + 2hr\cdot\sqrt{1 + N^2} \tag{3.7}$$

Where in special case of the rectangular canal,

$$p = B + 2H \tag{3.8}$$

This is followed by the calculation of the hydraulic radius (R) by;

$$R = \frac{A}{P} \tag{3.9}$$

The only dimension now left to be calculated is the slope (S) of the headrace canal which is given by Manning's equation. The manning equation is the most commonly used equation to calculate open channel flows. It simulates water flows in channels where the water is open to atmosphere and is not flowing under any other forms of pressure (Pandey B. , 2006). The Manning's equation can be stated as,

$$S=(n.V/R^{0.667})^{2} \tag{3.10}$$

With this all the dimension needed for the design of the headrace canal are known. However if the slope of the canal varies depending upon different sections of the canal it is necessary then to calculate the head loss for all of these sections and to sum them up. Head loss in the head race canal can be calculated by;

$$\text{Head Loss} = L\times S \tag{3.11}$$

Where L is the length of the canal section and S is given by Manning's equation (Fox, 2004) which can be rewritten as,

$$Q = \frac{\left[\left(BH + NH^2\right)^{\frac{5}{3}} \cdot \sqrt{S}\right]}{\left[\left[n\cdot\left[B + 2H\cdot\sqrt{1 + N^2}\right]\right]^{\frac{2}{3}}\right]} \tag{3.12}$$

However, in order to allow for the uncertainties in the design of the headrace canal it is necessary to allow for a "free board" of 300 mm if the water flow is less than 500 l/s and 400 mm of freeboard for water flow is in between 500 - 1000 l/s (liter/second). It is also necessary to consider the size of the particle that can possibly be carried in the headrace canal which is undesirable. The maximum particle size in diameter that can be transported in the head race canal can be calculated by,

$$d = 11\ RS \tag{3.13}$$

Where, "R" is the hydraulic radius as given in equation 3.9 before and S is again given by the Manning's equation given in equation 3.10.

2.2.6 Spillway Design Parameters

The dimension of the spillway are given by,

$$L_{spillway}=(Q_{flood}-Q_{design}) / C_w(h_{flood}-h_{sp})^{1.5} \tag{4.1}$$

Where,

$L_{spillway}$ = length of the spillway (m)

Q_{flood} = flood flow via intake (m^3/s)

$Q_{desig\ n}$= design flow in headrace canal (m^3/s)

h_{flood} = height of the flood level in the canal (m)

h_{sp} = height of the spillway crest from canal bed (m)

$h_{overtop} = h_{flood} - h_{sp}$

In addition to this parameter a coefficient while determining the spillway profile C_w needs to be determined. Literature (ENTEC AG, March 2001) agrees that for a MHP (Micro Hydro Project) broad round edged profile where $C_w = 1.6$ is easy and reliable to be constructed. Similarly the value used for C_w is 1.6. Therefore, the value taken while designing spillway for the case project is also 1.6.

The basic process for designing the spillway is to first calculate the intake during floods. Then the maximum height of the water level in the canal during a flood should be calculated followed by the dimensions for the spillway crest.

2.2.7 Construction of Settling Basin

For the construction of the settling basin, the first step is to choose a suitable width of the basin (W). Rule of the thumb dictates that the width of the settling basin should be two to five times larger than that of the headrace canal trying to make it as bigger as possible depending upon the available width in the MHP (Micro Hydro Project) location (Pandey B. , 2006). After determination of the width, the next process is to calculate the length of settling basin ($L_{settling}$) by using the equation:

$$L_{settling} = 2 \times Q / (W \times V_{vertical}) \quad (5.1)$$

Where,

Q = design flow (m^3/s)

$V_{vertical}$ = fall velocity (For the settling particles of 0.3 mm diameter the fall velocity is taken as 0.03 m/s)

By this equation the length of the settling basin is determined, but it is very important to check at this time that the length of the settling basin is around four to ten times its width. After determination of the length of the settling basin, it is necessary to calculate the silt load (S_{load})of the settling basin, which is given by:

$$S_{load} = Q \times T \times C \quad (5.2)$$

where,

S_{load} = silt load (kg)

Q = discharge (m^3/s)

T = silt emptying frequency in seconds. In MHP (Micro Hydro Project), 12 hours or 43,200 seconds is used (Pandey B. , 2006).

C = silt concentration of incoming flow (kg/m^3) [If there are no silt concentration data, 0.5 kg/m^3 can be safely used (Pandey B. , 2006).

After determination of these dimensions it is necessary to finally calculate the volume of the silt load by:

$$VO_{silt} = \frac{S_{load}}{\left| S_{density} \cdot P_{factor} \right|} \qquad (5.3)$$

Where,

VO_{silt} = volume of silt stored in basin

$S_{density}$ = density of silt (2.600 kg/m^3 is generally used)

P_{factor} = packing factor of sediments submerged in water (50% is generally used)

After calculating the volume of the silt it is then necessary to calculate the average depth required for the settling basin ($D_{collection}$) which can be given by:

$$D_{collection} = VO_{silt} / (L_{settling} \times W) \qquad (5.4)$$

2.2.8 Design of Forebay Tank

Having discussed the construction of the settling basin it should be noted here that the construction of the forebay tank is very much similar to the design of the settling basin. The only difference between the construction of the forebay tank and the settling basin is that the forebay tank is connected to the penstock pipes.

While constructing the forebay tank, the first step is to calculate the submergence head which is the depth of water above the crown of the penstock pipe. It should be carefully designed because if the submergence head is too small air can enter into the pipe causing variations in the penstock flow as well as causing explosion of penstock pipe due to entry of unwanted air in the pipes. (ENTEC AG, March 2001)The basic rule of the thumb while calculating the submergence head is given by the equation:

$$H_s \geq 1.5\ V^2/2g \qquad (6.1)$$

Where V=velocity in the penstock

The next step is to calculate the storage depth. It is generally recommended to be 300 mm or equal to the penstock pipe diameter. Since, in Nepalese condition, where the forebay tank is cleaned manually the minimal size and structure of forebay tank should allow a normal person to enter and clean the tank. Therefore, the minimum clear width recommended is 1 meter, and so designed, that 15 seconds of design flow, is stored in the tank above minimum submergence head. A gate valve is often situated at the entrance of the penstock which allows the water flow in the penstock pipe to close for maintenance work in the turbine (Khatri & Uprety, 2002).However, rapid closure of valve can cause vacuum inside the pipe causing its collapse. To prevent such a situation, air vent are usually placed in the forebay tank which let the air to enter the air vent rather than the penstock. The dimension of the air vent is given by the equation:

$$d^2_{airvent} = Q \checkmark [(F/E)\ (D/t_{effective})^3] \quad (6.2)$$

Where,

$d^2_{airvent}$ = internal diameter of air vent (mm)

Q = maximum flow of water through turbine (l/s)

E = Young's modulus for the penstock (N/mm^2)

$t_{effective}$ = effective penstock wall thickness at upper end (mm)

F = safety factor (which is generally considered to be 5 for buried penstock pipe and 10 for the exposed pipe)

2.2.9 Factors to be considered in Penstock Selection and Design

The factors that need to be considered in designing and selecting the penstock material are briefly described in this section. The most important factor to be considered while designing the penstock pipe is the material to be used as a penstock. Usually mild steel and HDPE pipes are used in MHP (Micro Hydro Project). There are several factors to be considered when selecting material to be used in the penstock pipe. Table 3 illustrates the possibilities of using different kinds of material based on various factors. The more the number of "stars" the more favorable is the material type under different characteristics. For example, if friction loss was the major concern in selection, uPVC type of penstock would clearly be the first choice.

Table 3: The differences in Penstock Pipe Material

Material	Friction Loss	Weight	Corrosion	Cost	Jointing	Pressure
Mild Steel	***	***	***	****	****	*****
uPVC	*****	*****	****	****	****	****
Concrete	*	*	*****	***	***	*
Ductile Iron	****	*	****	**	*****	****

After selecting the material for the penstock pipe, it is necessary to determine its diameter. The most important design parameter in this selection is that the velocity of the water should be in between 2.5 m/s to 3.5 m/s. If the velocity is lower or higher it can cause loss in the power output and thus be uneconomical in the longer run. The equation for determining the diameter of the pipe (Fox, 2004) is given by:

$$d_{pipe} = \sqrt{\left(\frac{4 \cdot Q}{\pi \cdot V}\right)} \qquad (7.1)$$

Where,

d_{pipe} = inside diameter of the pipe (m)

Q = design flow (m^3/s)

V = average velocity in the pipe (m/s)

After selecting the material and the diameter of the penstock pipe it is necessary to calculate the head loss in the pipe length which is given as;

Total head loss = major head loss (h_f) + minor head loss (H_{minor}) where,

Major Head loss (h_f) (Fox, 2004) is equal to:

$$h_f = \frac{f \cdot L \cdot V^2}{2 \cdot g \cdot d_{pipe}} \qquad (7.2)$$

And,

$$H_{minor} = V^2 \left(K_{entrance} + K_{bend} + K_{contraction} + K_{valve}\right)/2g \qquad (7.\ 3)$$

Where,

F = friction factor for pipe material, dimension less

L = length of pipe in meters

V = Average velocity inside pipe, m/s

d_{pipe} = the inside pipe diameter in meters

KS = Coefficients for pipe shape geometry (dimension less)

After selection of the material, calculation of the diameter and head loss it is also necessary to calculate the thickness of the penstock pipe. The thickness of the pipe depends on the pipe diameter, the material and the type of turbine selected. Since, the most likely turbine that are selected in MHP (Micro Hydro Project) in cases of Nepal are pelton turbine, the equation for calculating the pressure wave velocity, a

$$a = \frac{1400}{\sqrt{\left(1 + \frac{2,1 \cdot 10^9 \cdot d}{E \cdot T}\right)}} \qquad (7.4)$$

Where,

E = the value of Young´s Modulus (Fox, 2004) for mild steel is 210 X 10^9 N/m^2 and for HDPE is 0.2 - 0.8 x 10^9 N/m^2

d = pipe diameter in mm

T = the wall thickness in mm

Then, the velocity V in the penstock is given by:

$$V = \frac{4 \cdot Q}{\pi \cdot d^2} \qquad (7.5)$$

And therefore the surge head (h_{surge}) is

$$h_{surge} = \frac{a \cdot V}{n \cdot g} \qquad (7.6)$$

Total head (h_{total}) = h_{gross} + h_{surge}

Additionally, the critical time (T) also needs to be calculated via following equation:

T_c = (2L)/a (7.7

Where, Tc = the critical time in seconds

L = the length of penstock in m

a = wave velocity

As a precaution it is also necessary to calculate the safety factor (SF) of the penstock pipes. The equation of the safety factor (SF) is given by:

$$SF = \left(t_{effective} \cdot S \right) \cdot \left(5 \cdot h_{total} \cdot 10^3 \cdot d \right) \qquad (7.8)$$

Where,

S = the ultimate tensile strength of the pipe material in N/m^2

d = the internal diameter of the pipe in mm

After all these calculations, the optimum thickness of the penstock pipe can be determined.

2.2.10 Selection of Turbines and its Components

The parameters that help in the choice of turbine are tabulated below in table 4. It is primarily the head measurement that determines the selection of a suitable turbine for a particular MHS (Micro Hydropower System). (Hydraulic Energy Program et al., 2004). For example, in cases where the head measurement is more than 50 meters, pelton or turgo types of turbines are chosen over others. Similarly, when the head measurement is in between 10 meters and 50 meters, cross-flow, turgo or multijet pelton types of turbine are preferred. In cases where the head measurement is lower than 10 meters, cross-flow turbine is preferred. The selection for reaction type of turbines is also made in a similar way, and these criteria are summarized in table 4.

Table 4: Factors affecting the selection of the Turbine for MHS (Micro Hydropower System)

Head Classification	Type of Turbine	
	Impulse	Reaction
High (>50m)	Pelton, Turgo	
Medium (10-50m)	Cross flow, Turgo, Multi-jet Pelton	Francis (spiral case)
Low (<10m)	Cross flow	Propeller, Kaplan, Francis (Open Flume)

2.2.11 Generators

Synchronous generators are used in most MHP (Micro Hydro Project) because it has the ability to establish its own operating voltage and maintain frequency while it is operating in a remote location. (Harper, December 2011)

3 METHODS

3.1 Description of the Case

Lamaya Khola MHP (Micro Hydro Project) project was approved by the Pangrang Village Development Committee (VDC) of Nepal on May 2010, after a detailed feasibility research taking into account all the relevant technical, economic and social factors. The project gets its name from Lamaya Khola River, the water source for the MHP (Micro Hydro Project), which is a perennial source of water with sufficient discharge also in the dry season. Pangrang VDC lies in the western part of Nepal, in the mountainous regions where the elevation of the proposed site for the MHP (Micro Hydro Project) is above 1300 mean sea level.

Before discussing the design of primary components of the MHP (Micro Hydro Project), it is necessary first to discuss the preliminary data available through the feasibility survey. The survey team appointed by Pangrang Village Development Committee made detailed engineering survey at around summer of 2010. Detailed measurements were carried out to locate the best position for intake, headrace canal, forebay, powerhouse, tailrace and the spillway. Some of the important parameters that were measured were published by Pangrang VDC and have been used in this study as secondary source of data. Some of those important survey parameters are discussed in this section.

The report published by Pangrang V.D.C. on Lamaya Khola River shows that the flow of water measured on 12th of May, 2010 was 0.204 m^3/s. However, as the report explains, design discharge was taken to be 0.154 m^3/s, after considering that 15% of the calculated lowest flow downstream releases due to environmental reasons and 10% of the discharge were lost through evaporation or seepage. The survey team used salt dilution method to calculate the water flow and to derive the design discharge from this data. The detailed results of the mean monthly flow of Lamaya Khola River, is shown in Table 5

Table 5: Mean Monthly Flow of Lamaya Khola River (Pangrang Village Development Committee, 2010)

Month	Flow at River, Liter/ Second
January	535.44
February	402.69
March	300.91
April	221.26
May	201.34
June	604.34
July	2480.30
August	3084.34
September	2212.58
October	1442.60
November	1006.72
December	736.79
Annual Average	1102.42

Pangrang VDC also conducted the survey about the energy consumption pattern in Pangrang VDC and the estimated demand for electricity, so as to determine the power output of the MHP (Micro Hydro Project), which would be able to satisfy this demand. The detailed of the energy consumption pattern are quite much in detail but the part of the energy consumption pattern which shows the estimated power demand to be 25 kW is given in Appendix 3.

3.2 Feasibility Analysis

The detailed feasibility study was carried out by independent consultants through fieldwork. The power to be generated for the MHP (Micro Hydro Project) is determined largely based on the demand of the local community (Pangrang Village Development Committee, 2010). Survey was carried out by Pangrang Village Development Committed in 2010 to collect information regarding the demand for power in the locality and the villager's willingness to pay for the electricity supplied. In the demand survey the headcount of the villagers according to households and rural commercial demand such as for offices; schools and so on were also calculated. The survey also included the technical and socio economic issues important for the project but as highlighted in the limitations of this study, those factors are beyond the scope of this study. The feasibility analysis also included the survey regarding hydrological, geological and topographical information needed prior to the actual design of the system components. Based on these data, appropriate location for water intake, headrace canal, forebay and other powerhouse and electrical transmission system were carried out. Site survey also included the flow measurement of Lamaya Khola River, determination of head needed for the required power output which was taken to be 25 kW. Similarly, after desirable locations for the civil structures were determined by the survey, the “head” available and the distance between these various civil structures were also measured.

The determination of head is very important steps in the design of MHS (Micro Hydropower System). The gross head was measured to be 30 meters by on field design survey by the use of altimeter. (Pangrang Village Development Committee, 2010). There are several methods of determining the water flow of the river, but in this case the site surveyors used the salt-dilution method of determining the head. The salt dilution method was especially appropriate in this case because Lamaya Khola River is a small flow stream. It is also thought to be accurate and quick for shallow mountain streams such as in our case. The summary of the data available from the feasibility analysis is provided in table 6.

Table 6: Summary of feasibility analysis data (Source: Published Report from Pangrang Village Development Committee)

Features	Preliminary Survey data
Gross Head	30 m
Measured Flow and Date	0.204 m^3/s (May 2010)
Least Flow	0.20134 m^3/s
Design Flood Level	0.6 m above normal water level
Water Level	0.8 m
Design Discharge	0.154 m^3/s
River Source	Lamaya Khola
Power	25000Watts/ 25 kW
No of households demanded	259 households
Length of Headrace Canal	1071m (Stone Masonry with cement mortar)
Generator	50 kVA,3 Phases, Synchronous, Brushless
Load Controller	ELC 25 kW with Ballast tank 30 kW
Penstock Type and Length	Mild Steel Type, 66.0 m
Type of Turbine	Peloton 38 kW shaft output
Powerhouse Dimensions	5 m x 4 m x 2.7 m

3.3 Design of Primary Components

Based on this primary survey, as a secondary data source and the design parameters discussed in the theoretical overview, these information has been utilized in the design of various system components of the MHP (Micro Hydro Project).

3.3.1 Design of orifice for side intake

It is known from the feasibility analysis that the design flow of Lamaya Khola river, Q = 0.154 m^3/s. It has also been determined from survey that normal water level in the river, h_r = 0.8 m and the design flood level is about 0.6 m above the normal level water. Therefore it can be calculated that the design flood level,

h_f = 0.8 m + 0.6 m = **1.4 m.** (This is necessary in order limit the excess of flows during the flood season, especially around the monsoon time.)

For the design purpose the velocity of water to pass through the orifice is taken as, V = 1.3 m/s. This value was so taken because for MHS (Micro Hydropower System) the recommended velocity through the orifice during normal flow is 1.0 - 1.5 m/s). Based on this premise, it is possible now to calculate the, Area of orifice, (A)

= Q x V = 0.154/ 1.3 = **0. 12 m^2**

The water level at the headrace canal or h_h is chosen to be 0.5 m. It is taken as such in order to submerge the orifice under the headrace canal. It is also known from the previous discussion of design parameters for side intake that;

$$Q = A \cdot V = A \cdot C \cdot \sqrt{2 \cdot g \left(h_r - h_h \right)} \qquad (2.1)$$

Now, letting **C = 0.6** (This value was taken because the orifice is planned to be of the roughly finished masonry type, whose value is normally taken to be 0.6, from table 2)

$$Q = 0.12 \times 0.6\sqrt{\ 2 \times 9.81(0.8 - 0.5)}$$

= 0.174 m^3/s

= **174 l/s**

Since it is known from the survey that $Q_{required}$ = 154 l/s and the calculations of side intake shows that it can deliver 174 l/s , the design is considered acceptable during the normal seasons.

However it is also necessary to consider, whether this design will work during the monsoon season when the water floods by taking into consideration the value of discharge through the orifice during flood flow (Q_{flood}).

It is known from the discussion of the design parameters that,

$$Q = A \cdot V = A \cdot C \cdot \sqrt{2 \cdot g \left(h_r - h_h \right)} \qquad (2.1)$$

Letting C=0.6 (for roughly finished masonry)

$Q_{flood} = 0.12 \times 0.6\sqrt{2} \times 9.81(1.4 - 0.5)$ = 302.55 l/s

This value will be important at the later stage when designing the spillway since the orifice is only designed to take 154 l/s

3.3.2 Design of Headrace Canal

The canal type chosen was stone masonry with the cement mortar where the design discharge from the survey data is calculated to be **$Q = 0.154\ m^3/s$**. The choice of this material was made because it is recommended that this type of material be used where the type of the soil is porous such as in the case of Lamaya Khola MHP (Micro Hydro Project) construction site. Choosing other materials such as earthen or stone mud canals had the risk of leading to water seepage through the canal surface that could have caused landslides in the surrounding area. (Pangrang Village Development Committee, 2010)

From table 2 which is reproduced below, the side slope and roughness coefficients of different types of headrace canal are given as:

<table>
<tr><td rowspan="2">Material used in the Canal</td><td rowspan="2">Side Slope(N=h/v)</td><td colspan="2">Maximum recommended velocity for canals (V)</td></tr>
<tr><td><0.3m depth</td><td><1m depth</td></tr>
<tr><td>Stone masonry with mud mortar</td><td>0.5-1.0</td><td>1.0</td><td>1.0</td></tr>
<tr><td>Stone masonry with cement mortar</td><td>0-1.5</td><td>1.5</td><td>1.5</td></tr>
<tr><td colspan="4">Roughness Coefficients for Masonry Canals</td></tr>
<tr><td rowspan="3">Masonry Canals</td><td>Brickwork</td><td colspan="2">Roughness coefficient “n”=0.015</td></tr>
<tr><td>Normal Masonry with cement mortar</td><td colspan="2">0.017</td></tr>
<tr><td>Coarse rubble masonry</td><td colspan="2">0.020</td></tr>
</table>

From the table, the roughness coefficient and the side slope of the canal can be easily determined as: roughness coefficient of normal masonry with cement mortar (n)

n = 0.017

Similarly for this type of canal the side slope (N) is selected to be=0.5

N = h/v

= 1/2

= **0.5**.

With this information and given that (Q) = 0.154 m^3/s, it is now possible to calculate the

Cross sectional area of the headrace canal,

A = Q/V (3.1)

= 0.154/0.9 = **0.171 m^2** (Since the maximum recommended velocity for stone masonry with cement mortar is 1.5 m/s, the velocity is arbitrarily taken to be 0.9 m/s)

Now the next objective is to calculate the optimum height of the canal (H), width of the canal bed (W) and the width of the canal top (T). For that, it is first necessary to find χ which is also the factor used to optimize the canal shape and is given by;

$x = 2\sqrt{(1+N^2)} - 2N$ (3.2)

$x = 2\sqrt{(1+0.5^2)} - 2 \times 0.5$

$\boldsymbol{x} =$ **1.236** (It has no unit because it is a coefficient to optimize the canal shape)

With this information it is possible to calculate the water depth in the canal (H) as follows:

- ⇨ $H = \sqrt{A/(x+N)}$
  (3.3)

$H = \sqrt{0.171/(1.236+0.5)}$

H= 0. 314 meters

With this information it is possible to calculate the bed width of the headrace canal (B) as follows:

$B = H \times x$ (3.4)

$B = 0.314 \times 1.236$

B = 0.387 meters

Now in order to calculate the top width of the design water level (T),

⇨ $T = B + (2HN)$ (3.5)

$T = 0.39 + (2 \times 0.31 \times 0.5)$

T = 0.7 meters

It is known from the discussion of design condition that it is necessary to check if

$V < 0.8\, V_c$

because in order to ensure that the water flows in a stable and uniform flow in the head-race canal the velocity of water must be 80% less than the critical velocity where critical velocity ($V_{c)}$ is,

⇨ $V_c = \sqrt{[Ag/T]}$ (3. 6)

$= \sqrt{[0.171 \times 9.8/ 0.7]}$ $A=0.171\ m^2$

V_C = **1.55 m/s**

$0.8V_c = 0.8\ m \times 1.55 =$ **1.23**

Therefore the velocity of the water (0.9 m/s) in the headrace canal is less than 80% the critical velocity (1.23 m/s). Therefore it can be considered that the design of this head-race canal is acceptable.

After determining that the design is acceptable, and calculating the internal canal dimensions now it is necessary to calculate the wetted perimeter of the headrace canal which is given by:

$P = B+2\times H \times \sqrt{(1+N^2)}$ (3.7)

$P = 0.387+2\times 0.314\times \sqrt{(1+0.5^2)}$

P = 1. 08 meters

After this it is also necessary to calculate the hydraulic radius "R"

To do this we know that

$R = A/P$ (3.9)

Where, (A) is the cross sectional area of the headrace canal and (P) is the wetted perimeter which was calculated beforehand. Therefore, hydraulic radius "R"

= 0.171 / 1.06

R = 0.161meters

Now the final dimension "bed slope" "S" needs to be calculated in order to design the headrace canal and this is given by the Manning's equation. Manning's equation actually relates the flow and velocity of water but it can be applied in this situation as follows.

$S = [nV / R^{0.667}]^2$ (3.10)

$= [0.017\times0.9 / 0.161^{0.667}]^2$

$S = 2.675\times 10^{-3}$

S = 0.002675

S = 0. 0028= (1:374)

This slope value indicates that in a 1 m of drop in 374 m of horizontal canal length. Now it is necessary to calculate the head loss in the headrace canal which is given by:

Head Loss = Lx S (3.11)

We know from the survey conducted that the length determined for the headrace canal is 1071 meters (refer to table 5 for results from the survey) and we calculated that slope is 0.0028. Therefore,

Head Loss = 1071 x 0.002675 = **2.86 meters**

As the actual slope of the canal can be different depending upon, it is important now to calculate the head loss for each of the parts and to add them up repeatedly for different velocities. In this case, the optimum height for the canal bed can be determined by hit and trial method in Manning's equation for the value of H or,

$Q = [(BH + NH^2)^{5/3} \sqrt{S}] / [n \{B+ 2H \sqrt{(1+N^2)}^{2/3}]$ (3.12)

Where, it has been already calculated, bed width of the headrace canal (B) as 0.387 m, N, the slope of the canal of cement mortar type (N) chosen as 0.5, the roughness coefficient of same canal (n) as 0.017 and finally design flow (Q) as 0.154 m^3/s.

- ⇨ $0.154 = [(0.39H + 0.5H^2)^{5/3} \sqrt{0.0028}] / [0.017 \{0.039+ 2H \sqrt{(1+0.5^2)}^{2/3}]$

By trial and error method the equation is balance when H~0.38 m.
Therefore, in this case, the flood flow occupies 50% of freeboard, and therefore the head of the spillway or $h_{overtop}$ will be 100 mm.

Now it is necessary to check for the size of the largest particle that can travel through the canal. This is necessary because beyond a certain size of particle, it is not desirable that they would pass through the canal,

$$d = 11RS \tag{3.1}$$

= 11 x 0.161 x 0.002675 = 0.0474 m = **4.74 mm**

The particle larger than 4.74 mm would settle in this headrace canal. Therefore, to avoid deposition upstream of the settling basin, the gravel trap must be designed to remove all particles greater than 4.74 mm.

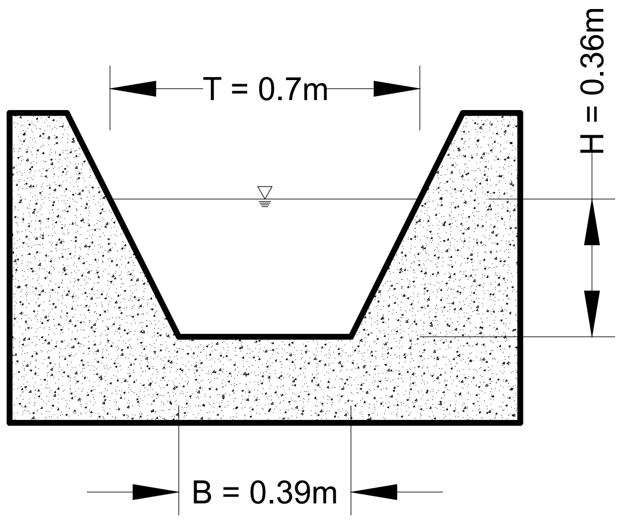

Figure 9: Representation of the headrace canal (AutoCAD)

Figure 9 illustrates the design of the headrace canal in autoCAD by putting in the above calculated dimensions.

3.3.3 Design of spillway

It was already pointed out in the section discussing design parameters that the design of the spillway can be made with calculations of its different dimensions.
The dimension of the spillway are given by,

$L_{spillway}=(Q_{flood}-Q_{design}) / C_w(h_{flood}-h_{sp})^{1.5}$ (Pandey V. , 2011) (4.1)

Where,

$L_{spillway}$ = length of the spillway (m)

Q_{flood} = flood flow via intake (m^3/s)

Q_{design} = design flow in headrace canal (m^3/s)

h_{flood} = height of the flood level in the canal (m)

h_{sp} = height of the spillway crest from canal bed (m)

$h_{overtop} = h_{flood}-h_{sp}$

It is known by calculating dimensions of the stone masonry with cement mortar head-race canal that it can let the design flow of water (Q_{design}) of 0.174 m^3/s during normal season. Similarly, during the flood season the flow of water (Q_{flood}) might reach to 0.302 m^3/s. This was determined when the orifice for side intake was calculated. By design of convention $H_{overtop}$ is taken as 100 mm. It is also known that the height of the flood level in the canal is 1.4 meters. This value calculated previously during the design of orifice for the side intake. The only other value needed in the equation to determine the length of the spillway is C_w or the coefficient for a road crested weir with round edg-es. This is generally used for the construction of MHP (Micro Hydro Project) and its value is considered to be 1.6 (Pandey B. , 2006).

For the design of the spillway it is first necessary to consider what would be the length required if the design flow was 0 m^3/s,

Because the material for spillway was chosen as crested weir with round edges profile,

$C_w = 1.6$

$Q_{flood} = 0.302\ m^3/s$

$Q_{design} = 0\ m^3/s$

$h_{overtop}$ ($h_{flood} - h_{sp}$) = 100 mm by convention,

In this case the length of the spillway would be,

$L_{spillway} = (Q_{flood} - Qdesign) / C_w (h_{flood} - h_{sp})^{1.5}$

= (0.302-0) / 1.6 (0.1)1. 5

= 0.302/1.6 x 0.0316=0.302/0.051=5.921= **6 m**

However, it is also necessary to calculate the length of the spillway considering the actual flow of water that enters through the orifice and the head race canal. In that case,

$C_w = 1.6$

$Q_{flood} = 0.302\ m^3/s$

$Q_{design} = 0.154\ m^3/s$

$h_{overtop}\ (h_{flood} - h_{sp}) = 100$ mm by convention,

In this case the length of the spillway would be,

$L_{spillway} = (Q_{flood} - Qdesign) / C_w\ (h_{flood} - h_{sp})^{1.5}$

$= (0.302\text{-}0.154) / 1.6\ (0.1)^{1.5}$

$= 0.148/1.6 \times 0.0316 = 0.148/0.051 = 5.921 =$ **3 meters**

In order to satisfy both of these conditions, it is desirable that the spillway of length **6 m** be constructed.

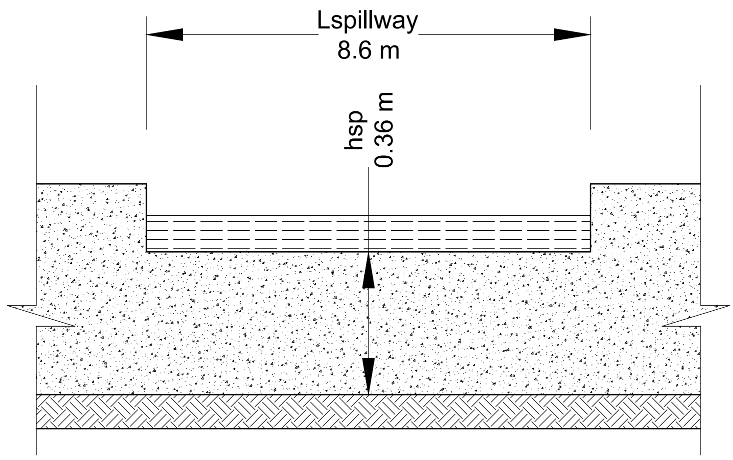

Figure 10: Longitudinal section of the designed spillway (AutoCAD)

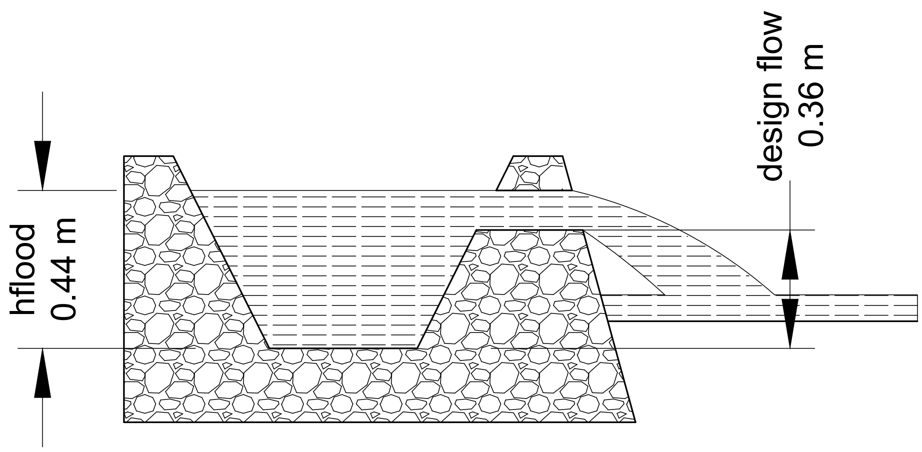

Figure 11: Cross section of the designed spillway (AutoCAD)

Figure 10 and figure 11 represent the longitudinal view and the cross sectional view of the spillway designed by putting the calculated dimensions in autoCAD.

3.3.4 Design of the settling basin

For the design of settling basin, the first step is to choose arbitrarily the suitable width of the settling basin. The rule of thumb says that it should be two to five times the width of the headrace canal; we know that the width of the headrace canal from calculations before is 0.387m. The width of the settling basin (W) was chosen as 1.5 m which is about four times the width of the headrace canal and is therefore allowed, So we have W=1.5 m, (Q or Q_{gross}) or the design flow is 0.154 m^3/s. It is known that in order to calculate the settling length ($L_{settling}$) of the settling basin the following equation is given,

$L_{settling} = 2 \times Q / (W \times V_{vertical})$ (5.1)

$V_{vertical}$ refers to the fall velocity taken as 0.03 m/s for example, for the value of particle of the size 0.3 mm. The same value will be taken in this case also,

The value (W) is arbitrarily chosen to be 1.5 m; Q_{gross} = 0.154 m^3/s and $V_{vertical}$ = 0.03 m/s

With this, following information can be obtained;

$L_{settling} = 2 \times Q / (W \times V_{vertical})$

$= (2 \times 0.154) / (1.5 \times 0.03) =$ **6. 84 m**

Therefore the length of the settling basin is 6.84 m. As the design parameter showed, the length of the settling basin should be four to 10 times of its width.

Here length = 6.84 m which is 6.84 / l.5 = 4.6; which is almost five times the width. Hence, the design is acceptable.

The next step is to calculate the expected silt load, S_{load} of the settling basin which is given by $S_{load} = Q \times T \times C$ (5.2)

This is quite straightforward because according to the recommended design parameters, T = silt emptying frequency in seconds = 12 hours = 12 x 60 x 60 = 43,200 seconds and C or the silt concentration of the incoming flow was given as 0.5 kg/m^3, therefore:

$S_{load} = Q \times T \times C = 0.154 \times 43200 \times 0.5 =$ **3326, 4 kg**

Now it is necessary to calculate the volume of the silt load, which is given by

$VO_{silt} = S_{load} / (S_{density} \times P_{factor})$ (5.3)

$S_{density}$ = density of silt, and it has been recommended that in absence of reliable data the safe parameter is 2600 kg/m^3, which will be used in this case also;

P_{factor} = packing factor of sediments submerged in water = 0.5(50%) as given. Therefore the volume of the silt load now is given as,

$VO_{silt} = S_{load} / (S_{density} \times P_{factor}) = 3326 / (2600\times 0.5) = \mathbf{2.55\ m^3}$

After these it is now important to calculate the average collection of the depth required in the settling basin which is given as:

$D_{collection} = VO_{silt} / (L_{settling} \times W)$ (5.4)

All the other parameters has been already calculated, therefore;

$D_{collection} = VO_{silt} / (L_{settling} \times W)$

$= 2.55 / (6.84\times 1.5)$

$= \mathbf{0.26\ m}$

Practically, as a safety factor in this design of settling basin, some extra volume is available to tapering. Here, it is also found out that the depth of the settling basin $D_{settling}$ is equal to the channel depth. Therefore, to avoid the turbulence, the entrance and the exit should be needed. However, the rule of the thumb for this design is to make them equal to the length in the basin width or in this case 2 m. Based on these dimensions, settling basin was designed in autoCAD software as represented in figure 12 and figure 13 successively. Figure 12 represents the longitudinal view of the settling basin whereas figure 13 represents the "top view" of the settling basin.

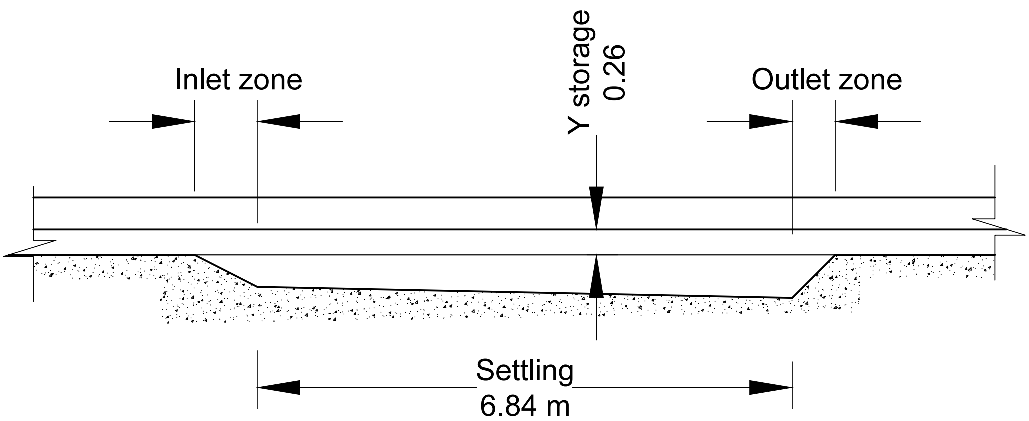

Figure 12: Longitudinal view of the settling basin (autoCAD)

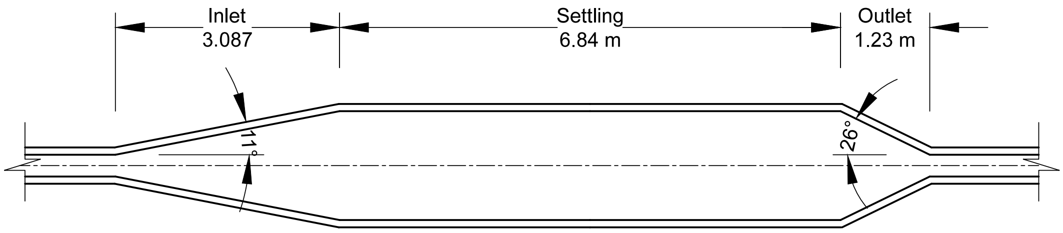

Figure 13: Top view of the settling basin (AutoCAD)

3.3.5 Design of penstock pipe

In the design of the penstock pipe, the first step is to choose the material for the penstock pipe. For that, the best choice in Nepalese condition is thought to be of mild steel, as they can withstand high pressure, are cheap to get and in case where several joints are required they are the easiest to manage (Chitrakar, 2004). As discussed already in the design parameters, the table 3 below which is reproduced again, showed the comparison of different materials:

Material	Friction Loss	Weight	Corrosion	Cost	Jointing	Pressure
Mild Steel	***	***	***	****	****	*****
uPVC	*****	*****	****	****	****	****
Concrete	*	*	*****	***	***	*
Ductile Iron	****	*	****	**	*****	****

After selecting the material for the penstock pipe, which was chosen as mild steel, it is necessary to determine its diameter. The most important design parameter in this selection is that the velocity of the water should be in between 2.5 m/s and 3.5 m/s. If the velocity is lower or higher it can cause loss in the power output and thus be uneconomical in the longer run. The equation for determining the diameter of the pipe is given by:

$$d_{pipe} = \sqrt{\left(\frac{4 \cdot Q}{\pi \cdot V}\right)} \qquad \text{(Fox, 2004)} \tag{7.1}$$

Where,

d_{pipe} = inside diameter of the pipe (m)

Q = design flow (m^3/s)

V = average velocity in the pipe (m/s)

In this specific case, since the pipe is a long set type, V= 2.7 m/s was chosen to minimize the head loss. (From design parameters it was recommended that velocity of water V be somewhere between 2.5m/s and 3.5m/s to optimize the velocity)

$d_{pipe} = \sqrt{[4Q/\pi V]} = [\sqrt{4 \times 0.154 / \pi \times 2.7}] = 0.28$ m = **280 mm**

After selecting the material and the diameter of the penstock pipe it is necessary to calculate the head loss in the pipe length which is given as;

Total head loss = major head loss (h_f) + minor head loss ($H_{minor)}$ where;

Major Head loss (h_f) (Fox, 2004)

$$h_f = \frac{f \cdot L \cdot V^2}{2 \cdot g \cdot d_{pipe}} \tag{7.2}$$

Where,

F = friction factor for pipe material, dimension less

L = length of pipe in meters

V = Average velocity inside pipe, m/s

d_{pipe}= the inside pipe diameter, meters

Now in this situation; f = 0.0014 (from Moody Chart: Provided in Appendix 4), Length chosen for the pipe was 66 m from the survey (table 5). Average velocity inside pipe is chosen as 2.7 m/s and the diameter of the pipe from calculation were found out as 270 mm or 0.27 m, therefore;

Major head loss $h_f = (fLV^2)/ (2g\ d_{pipe})$

$= [0.0014\times 66 \times 2.7^2]/ [2\times 0.27\times 9.8] =$ **0.1272 m**

Now in order to calculate the total head loss it is also necessary to calculate the minor head loss which is given by;

Minor head loss $(h_{minor)} = V^2\ (K_{entrance} + K_{bend} + Kcontraction + K_{valve})/2g$ (7.3)

Since the material for the penstock is mild steel this is also a pipe with seven vertical bends. Therefore, (reference here)

$K_{entrance} = 0.2$

$K_{contraction} = 0$ (not available in this case)

$K_{valve} = 0$ (not available in this case)

$K_{bend} = 0.34$ for θ 23°,

$K_{bend} = 0.20$ for θ 40°,

$K_{bend} = 0.06$ for θ 12°,

$K_{bend} = 0.18$ for θ 37°,

$K_{bend} = 0.04$ for θ 8°,

$K_{bend} = 0.13$ for θ 26°,

$K_{bend} = 0.04$ for θ 80°

Minor head losses, $h_{minor} = [V^2/2g] \times (K_{entrance} + K_{bend} + K_{contraction} + K_{valve})$ (7.3)

$= [2.72 / 2\times 9.8] \times (0.2 + 0.34 + 0.20 + 0.06 + 0.18 + 0.04 + 0.13 + 0.04)$

= **0.45 m**

Now this the total head loss can be calculated = 0.44 m + 0.272 m = **0.722 m**

% head loss = 0.5698 / 30 = 0.02% <5 %. This shows that the diameter of the penstock could be actually made a smaller.

After selection of the material, calculation of the diameter and head loss it is also necessary to calculate the thickness of the penstock pipe. The thickness of the pipe depends on the pipe diameter, the material and the type of turbine selected. Since, the most likely turbine that are selected in MHP (Micro Hydro Project) in cases of Nepal are pelton turbine, the equation for calculating the pressure wave velocity (Fox, 2004), “a”

$$a = \frac{1400}{\sqrt{\left(1 + \frac{2,1\cdot 10^9\cdot d}{E\cdot T}\right)}} \tag{7.4}$$

Where,

E = the value of Young´s Modulus for mild steel is 210 X 10^9 N/m^2

d = pipe diameter in mm, in this case 280 mm

T = the wall thickness in mm

Now, in this situation, first it is necessary to calculate the thickness required at the downstream end of the penstock

($h_{static} = h_{gross}$ = 30 m (from the survey data)

d = 280 mm

In this case also, the hit and trial method of solution should be aimed for. While trying for T = 5 mm and when E= 210 × 109 N/m^2)

- ⇨ $a = 1400 / \sqrt{[1 + (2.1 \times 10^9 \times d / E \times T)]}$
- ⇨ $1400 / \sqrt{[1 + (2.1 \times 10^9 \times 0.28 / 210 \times 10^9 \text{ x } 5 \times 10^{-3})]}$
- ⇨ **1399.99 m/s**

But then it is also known that, Q = 0.154 m^3/s (from survey data) and diameter as 280 mm or 0.28 m

$V = 4\times Q / \pi\times d^2$ (7.5)

$= 4 \times 0.154 / \pi\times 0.28^2$ = **2.50 m**/s. Therefore the actual velocity of the water in the penstock pipe would be 2.5 m/s

$h_{surge} = a \times V / (n \times g)$ (7.6)

= 1399. 99× 2.5 / 2 × 9.8 = **178** m

$h_{total} = h_{gross} + h_{surge}$ = 30 + 178 = **208** m

$t_{effective}$ = [5/ 1.1× 1.2] -1.0 = **2.78** mm

In order to calculate the safety factor (SF) of the penstock pipe following equation can be used;

$SF = (t_{effectiv} \times S) / (5 \times h_{total} \times 10^3 \times d)$ (7.8)

$= 2.78 \times 350\times 10^6 / 5 \times 208 \times 10^3 \times 280$ = **3.341**

2.5 < SF (3. 341) < 3.5

The safety factor condition shows that the design of penstock pipe assembly is acceptable. Although, it was not possible to design and show penstock pope to scale; the representative figure was designed in autoCAD and is presented in figure 14.

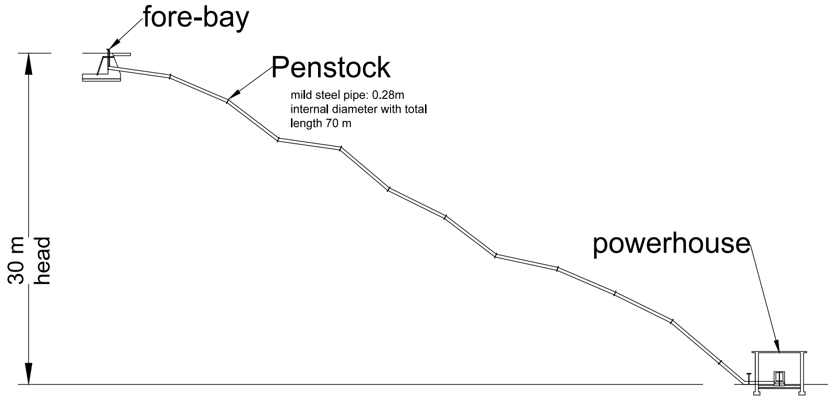

Figure 14: Design of the penstock pipe (AutoCAD)

Similarly, although calculating the dimensions of numerous anchor blocks to support the penstock assembly was not feasible, the representative design of an anchor block from autoCAD is shown in figure 15.

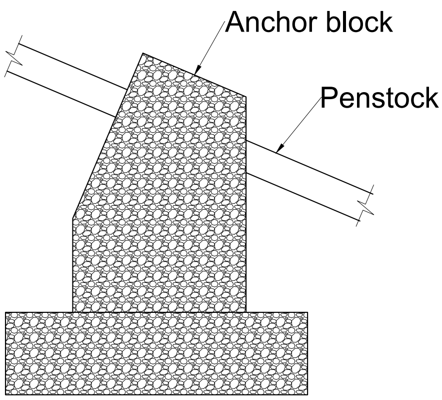

Figure 15: Anchor blocks designed to support penstock pipes in vertical and horizontal bends to support the structure (AutoCAD)

3.3.6 Design of the forebay tank

The forebay tank is similar to the settling basin except that the outlet transition is replaced by a trash rack and the forebay tank has an opening to penstock pipes. The most important element to be calculated in the design of the forebay tank is the submergence head. The submergence head or the depth of water above penstock pipe, should fulfill the criteria (Submergence head) $h_s \geq 1.5\ V^2/2g$ (6.1)

Where, V refers to the velocity of water in the penstock, which in this case is 2.5 m/s;

Therefore,

$h_s \geq 1.5\ V^2/2g$

$h_s \geq 1.5 \times 2.5^2/2 \times 9.8$

$\mathbf{h_s \geq 0.48\ m}$

In other words, the submergence head of the forebay tank should be 0.48 meters.

Similarly while designing the forebay tank it is also necessary to construct the diameter of the air vent or $d_{airvent}$ which is given as,

$d^2_{airvent} = Q\ \sqrt{[(F/E)\ (D / teffective)^3]}$ (6.2)

Where,

$d^2_{airvent}$ = internal diameter of air vent in mm

It is already known from the survey that the maximum flow of water through turbine is 0.154 m^3/s, "E" is the Young's modulus for the penstock material, D is the diameter of the penstock and "F" is the safety factor. In this case, it has been chosen as 10 because our design consists of exposed pipes. Therefore,

$d^2_{airvent} = 154\ \sqrt{[(10/210000)\ (280/\ 2.78)^3]}$

= **32.77 mm.** The diameter of the air vent to be constructed is therefore 32 mm.

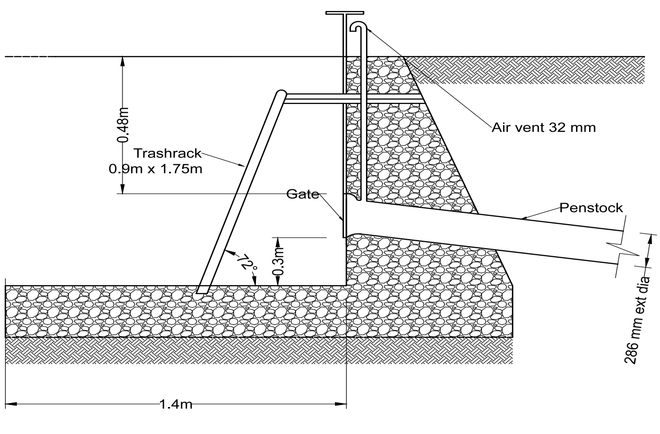

Figure 16: Design of the forebay tank with trash rack and air vent shown (AutoCAD)

3.3.7 Selection of Turbine and basic design of other powerhouse components

Since, the gross head of the MHS (Micro Hydropower System) is 30 m and the design discharge is 0.1554 m^3/s, the appropriate turbine for this situation proposed is pelton flow turbine, with efficiency of 70-75% and rated capacity of more than 25 kW (most probably 38 kW). Similarly, the basic dimensions of the powerhouse were already calculated in the preliminary survey. Based on these dimensions, the design of physical powerhouse in autoCAD is given in figure 17.

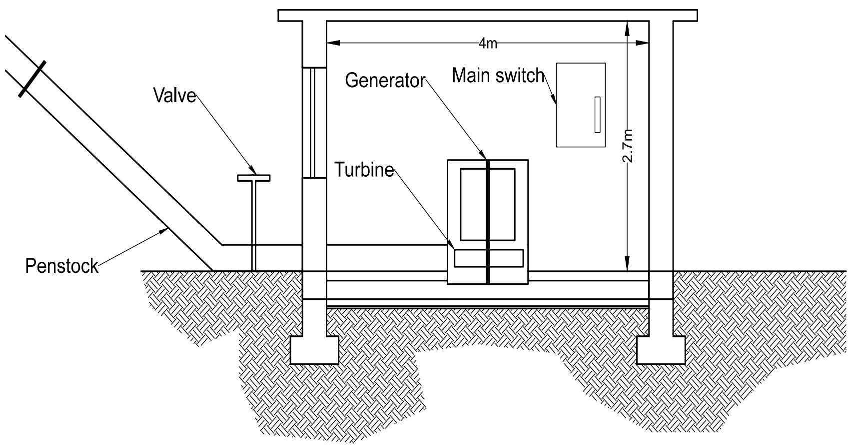

Figure 17: Design of powerhouse components from AutoCAD

4 RESULTS

Based on the field survey data, the study dealt with designing the major civil components for the Lamaya Micro Hydro project in Pangrang Village Development Committee Nepal. Based on the design parameters, the calculations carried out helped to determine critical dimensions of the civil components of the Lamaya Khola Micro Hydro Project. The critical dimensions of various components are summarized in Table 7.

Table 7: Summary of the critical dimensions of civil components of Lamaya Khola MHP (Micro Hydro Project)

Components	**Critical Dimensions**
Dimensions of Orifice for side Intake	Design flood level = 1.4 m
	Area of Orifice = 0.12 m^2
	Delivery discharge = 0.174 m^3/s Or 174 l/s
	Flood discharge = 302.55 l/s
Dimensions of Headrace Canal	Cross sectional area = 0.171 m^2
	Optimum Canal Height = 0.314 m
	Canal bed width = 0.387 m
	Canal top width = 0.7 m
	Critical Velocity = 1.55 m/s
	Wetted Perimeter = 1.08 m
	Head Loss = 2.86 m
	Hydraulic Radius = 0.161 m
	Canal Bed Slope = 0.0028 (1: 374m)
	Size of largest particle = 4.74 mm
Dimensions of Settling Basin	Length = 6.84 m
	Expected silt load = 3326.4 kg
	Volume of Silt Load = 2.55 m^3
	Average collection depth = 0.26 m
Spillway	Length = 6 m
Dimensions of penstock assembly	Material = Mild steel pipe 280 mm MS ID
	Length = **70 m**
	Pipe diameter = 260 mm
	Total head loss = 0.722 m
Forebay	Submergence head $\geq$0.48
	Diameter of air vent = 32.77 mm

5 CONCLUSIONS

Although, this thesis was about designing the system components for a MHP (Micro Hydro Project), majority of the system components designed included only civil work components which comprised of the side intake, headrace canal, forebay, sedimentation tank, and spillway and penstock assembly. However, the thesis would have gone beyond its limitation in scope and length if the design parameters and design process for other civil components such as the physical powerhouse have been added. In addition to civil components, a MHP (Micro Hydro Project) also consists of powerhouse components such as generator, turbine and so on and other components related to electrical distribution. These were not included in this thesis.

Similarly, the practical design of various components that were conducted in this thesis also led to the realization that the design of the system components is very much determined by the location specific factors. From the very beginning, the MHP (Micro Hydro Project) designed was constrained to being "run of the river" type, because the river source, Lamaya Khola, is situated in a mountainous topographical region. Similarly, in the design of spillway, headrace canal and forebay tank, the choice of materials were already determined by their availability and local topographical conditions. For example, the choice of stone masonry with cement mortar type of canal for the headrace was considered because in the topographically hilly reason, mud mortar type, for example would have led to seepage of water from the canal and so would have caused landslide in the longer run which is not considered desirable. Similarly, the choice of mild steel for the penstock and the type of turbine selected were also largely selected based on the norm of the region. Altogether the study showed that construction of MHP (Micro Hydro Project) was feasible in the project site and there were no major problems apparent at least at the design stage of the micro hydro project.

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APPENDICES

APPENDIX 1: Glossary of Formulas used

1) Power (P)

$$P=\rho.g.H.Q.\eta \qquad (1)$$

2) Construction of Intake Weir

$$Q = A\cdot V = A\cdot C\cdot\sqrt{2\cdot g\left(h_r - h_h\right)} \qquad (2)$$

3) Construction of Headrace Canal

$$A= Q/V \qquad (3.1)$$

$$\chi = 2\sqrt{\left(1 + N^2\right)} - 2\cdot N \qquad (3.2)$$

$$H = \sqrt{\frac{A}{(\chi + N)}} \qquad (3.3)$$

$$B = \chi\cdot H \qquad (3.4)$$

$$T = B + (2\cdot H\cdot N) \qquad (3.5)$$

$$V_c = \sqrt{\frac{(A\cdot g)}{T}} \qquad (3.6)$$

$$P = B + 2hr\cdot\sqrt{1 + N^2} \qquad (3.7)$$

$$p = B + 2H \qquad (3.8)$$

$$R = \frac{A}{P} \qquad (3.9)$$

$$S=(n.V/R^{0.667})^{2} \qquad (3.10)$$

$$\text{Head Loss} = L\times S \qquad (3.\ 11)$$

$$Q = \frac{\left[\left(BH + NH^2\right)^{\frac{5}{3}} \cdot \sqrt{S}\right]}{\left[\left[n\cdot\left[B + 2H\cdot\sqrt{\left(1 + N^2\right)}\right]\right]\right]^{\frac{2}{3}}} \tag{3.12}$$

$$d = 11\ RS \tag{3.13}$$

4) Design of Spillway

$L_{spillway}=(Q_{flood}-Q_{design}) / C_w(h_{flood}-h_{sp})^{1.5}$ (4.1)

5) Design of Settling Basin

$L_{settling} = 2\times Q/ (W\times V_{vertical})$ (5.1)

$S_{load} = Q.\ T.\ C$ (5.2)

$$VO_{silt} = \frac{S_{load}}{\left(S_{density}\cdot P_{factor}\right)} \tag{5.3}$$

$D_{collection} = VO_{silt}/ (L_{settling}\times W)$ (5.4)

6) Design of Forebay Tank

$H_s \geq 1.5\ V^2/2g$ (6.1)

$D_{airvent} = Q \checkmark[(F/E)\ (D/t_{effective})^3]$ (6.2)

7) Penstock Design

$$d_{pipe} = \sqrt{\left(\frac{4\cdot Q}{\pi\cdot V}\right)} \tag{7.1}$$

Total head loss=major head loss (h_f) + minor head loss (H_{minor}) where;

$$h_f = \frac{f\cdot L\cdot V^2}{2\cdot g\cdot d_{pipe}} \tag{7. 2}$$

$H_{minor}=V^2(K_{entrance}+K_{bend}+K_{contraction}+K_{valve})/2g$ (7.3)

$$a = \frac{1400}{\sqrt{\left(1 + \frac{2,1\cdot 10^9\cdot d}{E\cdot T}\right)}} \tag{7.4}$$

$$V = \frac{4\cdot Q}{\pi\cdot d^2} \tag{7.5}$$

$$h_{surge} = \frac{a\cdot V}{n\cdot g} \tag{7.6}$$

$$Tc = (2L)/a \quad (7.7)$$

$$SF = \left| t_{effective} S \right| \cdot \left| 5 \cdot h_{total} \cdot 10^3 \cdot d \right| \quad (7.8)$$

APPENDIX 2: Existing Small Hydropower Plants in Nepal

Table – 1 Existing Small Hydropower Plants*

S. No	Name of the Plants	District	Capacity (kW)	Year in Operation	Remarks
1	Pharping SHP	Kathmandu	500	1911	Out of Service
2	Sundarijal SHP	Kathmandu	640	1935	Grid Connected
3	Panauti SHP	Kavrepalanchowk	2400	1965	Grid Connected
4	Phewa SHP	Kaski	1088	1967	Grid Connected
5	Dhankuta SHP	Dhankuta	240	1971	Grid Connected
6	Surkhet SHP	Surkhet	345	1977	Grid Connected
7	Phidim SHP	Panchthar	240	1981	Isolated, DHQ
8	Tinao SHP	Rupandehi	1000	1978	Grid Connected
9	Baglung SHP	Baglung	200	1981	Grid Connected
10	Doti SHP	Doti	200	1981	Isolated, DHQ
11	Jumla SHP	Jumla	240	1982	Isolated, DHQ
12	Jomsom SHP	Mustang	240	1982	Isolated, DHQ
13	Seti SHP	Kaski	1500	1985	Grid Connected
14	Salleri–Chialsa SHP+	Solukhumbu	400	1986	Isolated, DHQ
15	Darchula SHP	Darchula	300	1992	Isolated, DHQ
16	Taplejung SHP	Taplejung	125	1988	Isolated, DHQ
17	Tehrathum SHP	Tehrathum	100	1988	Isolated, DHQ
18	Bhojpur SHP	Bhojpur	250	1989	Isolated, DHQ
19	Khandbari SHP	Sankhuwasabha	250	1989	Isolated, DHQ
20	Bajhang SHP	Bajhang	200	1989	Isolated, DHQ
21	Chaurjhari SHP	Rukum	150	1989	Isolated, DHQ
22	Serpodaha SHP	Rukum	200	1989	Isolated, DHQ
23	Okhaldhunga SHP	Okhaldhunga	125	1990	Isolated, DHQ
24	Bajura SHP	Bajura	200	1990	Isolated, DHQ
25	Arughat SHP	Gorkha	150	1990	Isolated, Villages
26	Surnayagad SHP	Baitadi	200	1991	Isolated, DHQ
27	Rupal Gad SHP	Dadeldhura	100	1991	Isolated, Villages
28	Tatopani SHP	Myagdi	2000	1991	Grid Connected
29	Andhi Khola SHP**	Syangja	5100	1991	Grid Connected
30	Namche SHP+	Solukhumbu	600	1993	Isolated, Villages
31	Achham SHP	Achham	400	1995	Isolated, DHQ
32	Kalikot SHP	Kalikot	500	1999	Isolated, DHQ
33	Dolpa SHP	Dolpa	200	1999	Isolated, DHQ
34	Syange SHP**	Lamjung	183	2001	Grid Connected
35	Indrawati Khola SHP**	Rasuwa	7500	2002	Grid Connected
36	Piluwa Khola SHP**	Sankhuwasabha	3000	2003	Grid Connected
37	Heldung SHP	Humla	500	-	Under Construction
38	Gam Gad SHP	Mugu	400	-	Under Construction
39	Puwa Khola SHP	Ilam	6200	2000	Grid Connected
40	Rairang Khola SHP**	Dhading	500	2004	Grid Connected
41	Sunkoshi Khola SHP**	Sindhupalchok	2500	2005	Grid Connected
42	Chaku Khola SHP**	Sindhupalchok	1500	2005	Grid Connected
43	Khudi Khola SHP**	Lamjung	4000	2006	Grid Connected
44	Sisne Khola SHP**	Palpa	700	-	Under Construction
45	Baramchi Khola SHP**	Sindhupalchowk	1000	2006	Grid Connected
46	Thopal Khola SHP**	Dhading	1640	-	Under Construction
		Total	**49,906**		

* - Previously called SHP Plants of less than 100 kW are not included in this list

+ - Local Company (Community). ** - Private IPPs

APPENDIX 3: Expected Load Demand Pattern in Pangrang VDC (Village Development Committee)

Load	4.00 am – 7.00 am	7.00 am – 10.00 am	10.00 am – 12.00 pm	12.00 pm –4.00 pm	4.00 pm – 6.00 pm	6.00 pm – 11.00 pm	11.00 pm – 4.00 am	Remarks
HH lightening	25					25		9 hrs/day
Agro-processing		8 kW		8 kW				10 hrs/day, 312 days/year operation
Rural carpentry		3 kW		3 kW				10 hrs/day, 312 days/year operation
Computer		2 kW						8 hrs/day, 312 days/year operation
Bakery		5 kW						5 hrs/day, 300 days/year operation
High Vision Hall			3 kW					6 hrs/day, 288 days/year operation
Total	**25 kW**	**18kW**	**10kW**	**11 kW**	**11 kW**	**25 kW**	**5 kW**	

Note: The agro-processing units and other end use should be run at different times during the day without overloading the plant.

APPENDIX 4: Moody Chart (Fox, 2004)