Addis Ababa University
School of Graduate Studies

Optimization of Small Hydropower in
The Abbay basin

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Description of abbreviations

EEPCO	Ethiopian electric power corporation
Acres	Acres international
ICS	Interconnected system of power supply
TCE	Tropics Consulting engineers
USBR	United states bureau of reclamation
WAPCOS	Water resources development master plan for Ethiopia
MME	Ministry of Mines and Energy
EACE	Ethiopian Association of Civil Engineering
MINOS	Modular In-core Nonlinear optimization
GRG	Generalized Reduced Gradient

Abstract

Although Ethiopia possesses a huge hydropower potential, the energy problem in the country is enormous. A great majority of the population in Ethiopia are dependent on traditional resources to fulfill their energy needs. Due to the poor energy supply, the people not only have to put up with an unattractive living environment but also suffer the consequences of climatic change caused by the associated environmental impacts. Numerous villages do not get electricity supply due to the scattered nature of the settlements. Therefore, the energy policy should give enough attention to decentralized systems in which small hydropower potential plays a significant role. This study considers some of the small hydropower sites in the Abbay basin and that the selection of optimum sites or the optimum level of development at the sites can be undertaken with the application of non-linear optimization techniques. The optimization has been conducted on the basis of four theoretical small hydropower systems established in the Abbay basin. Among these four, the two systems consisted of two hydropower alternatives each and the other two consisted of three hydropower alternatives each. In this thesis, Microsoft Excel 2000 solver, which uses the Generalized Reduced Gradient(GRG2) algorithm for optimizing nonlinear problems has been used. Using this optimization technique, the level of development of each of the hydropower sites has been determined. The method can also be used to select the optimum small hydropower site(s) from a number of alternatives. Further study for optimization can be conducted on renewable hybrid systems combining a decentralized energy sources for rural electrification.

2 Introduction

2.1 Current energy problem in Ethiopia

In Ethiopia, power cut-offs are frequent, access to electricity and modern fossil-based fuels is poor and unreliable, and the energy infrastructure suffers from chronic lack of investment. This is because Ethiopia possesses poor resources in terms of indigenous fuel and also the general economic situation of the country is very low.

In Ethiopia, the scarcity of energy becomes greater due to the high rate of population growth and weak energy supply system. The pace of energy supply has been outstripped by that of energy demand.

The generally backward infrastructural development in Ethiopia radically limits the pace of economic growth. Particularly the rural communities are adversely affected. This has a negative impact on the overall economy of the nation as the rural communities are the basis for agricultural development, which is the back bone of the economy of the nation.

Out of many infrastructural facilities that the small communities lack, the absence of appropriate form of energy is the most obvious one. Lack of energy has a detrimental effect on the growth of the nation since energy is the driving engine of economic development. In line with this, one of the primary objectives of the energy policy of the government of Ethiopia has been to ensure a reliable supply of energy at the right time and at an affordable price, particularly to support the agricultural development led industrialization strategy (MME 1994).

Ethiopia is endowed with huge hydropower potential, which has not been properly exploited. About 95% of the energy consumption in the country is based on firewood and fire wood derivatives. A total of about 5% of the population mainly in bigger cities have access to electricity (Acres-EEPCo2000). This has very big pressure on the environment which faces a process of continuous degradation and wide and severe deforestation. Unless improvement in the energy supply is made, the fire wood usage in Ethiopia is growing due to increase in population size. The overuse of firewood partly results in recurrent drought.

Investments in hydropower in Ethiopia have concentrated so far on large hydro dams. Many utilities prefer large centralized investments because they are considered easier to manage.

The Ethiopian electric power corporation (EEPCO) is the government utility in Ethiopia that monopolized until recently the electric energy generation and distribution in the country and concentrates highly on larger schemes .Because of its restricted budget, it adopts the criteria of cost effectiveness in its decision making to interconnect any town to the national grid. And this criterion favors only big load centers which have the necessary economic level to pay for electric supply.

Due to the mentioned facts, the current status of energy supply in Ethiopia is quite bleak. EEPCO is not only unable to cater for the needs of small communities but also has problem with the supply to the larger towns which may be solved to some extent due to the completion of Gilgel Gibe hydroelectric project.

2.2 Objective of the thesis

The objectives of this thesis are:

- To optimize small scale hydropower in the Abbay basin.
- To decide optimum site(s) for development and indicate the optimum level of development at the selected sites with optimum power supply.

3 The Abbay river Basin

3.1 Significance of the Abbay basin

The Abbay basin is the most important river basin in Ethiopia. It accounts for almost 20% of Ethiopia's land area, 50% of its total average annual runoff, and 25% of its population. The Abbay River itself has an average annual run off of about 49BCM.The Abbay basin covers a total area of 196700Km^2. The Abbay river basin has an irrigation potential of 978000ha and hydropower potential of 70036GWH/yr from which only 1.2% is utilized.

3.2 General description

The Abbay river rises in the center of the catchment develops its course in a clockwise spiral in a deep gorge collecting tributaries along its 922 km length from Lake Tana to the Sudan border. The elevation of the basin ranges from 490 masl at the Sudan border to 4230 masl at the summit of Mount Guna.

Annual rainfall varies between about 800mm to 2200mm, with a mean of about 1420mm.

The basin is predominantly rural in character with an economy dominated by rain fed subsistence agriculture.

3.3 Population and the settlement pattern

The basins population at the time of the 1994 census was 14.2 million, with an average density of about 71 persons per km^2 and an urbanization rate of less than 10%. The population growth rising in numbers at about 350000 per year.

Both the population density and growth rates are relatively high in Gojjam and south Gondar. They are lower in Shewa and Wello reflecting land degradation and less favorable agricultural conditions. Population growth is particularly low in North and South Wello. In contrast, growth rates in North Gondar and Benshangul Gumuz are high, reflecting the migration patterns.

Despite 47% of the population being under 15, household size is small, with houses often distributed across the farming area rather than concentrated in villages. The settlement pattern and the low rate of urbanization are associated with an ill-developed infrastructure and low level of services, housing standards are poor, and access to water supply and electric power are below the national average (Main Report April 1999)

3.4 Land and water resources

3.4.1 Physiography and Geology

3.4.1.1 Location

Abbay basin is located in the center and west of Ethiopia. It is approximately between latitude $7^0 45'$ and $12^0 46'$ north and longitude $34^0 06'$ and $40^0 00'$east being generally rectangular in shape, and extending about 400 km from north to south and about 550km from east to west(fig 2.4-1).

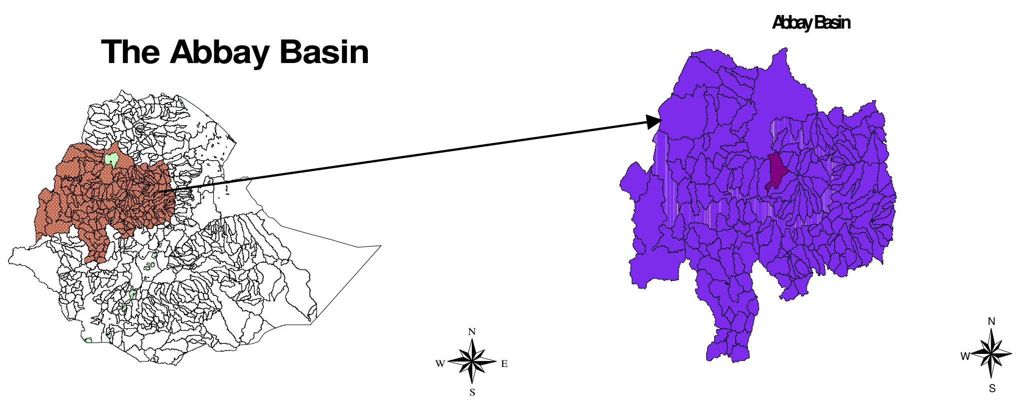


Figure 3.4-1 The Abbay Basin

3.4.1.2 Physiography

The Abbay river basin contains a mixed topography of high mountains, rolling ridges, flat grass land areas and meandering streams. After leaving Lake Tana the Abbay River enters a deep canyon (the Abbay gorge) formed within the underlying Precambrian metamorphic and sedimentary rocks. The canyon forms a natural barrier to communication and together with numerous deeply entrenched tributary water courses restricts the extension of ground water reservoirs and their storage capacity.

The distribution of elevation classes within the basin is indicated in the Table 2-1.

Table 3-1 distribution of elevation classes in Abbay basin

contour interval	Area(km^2)	Percentage of basin
500-1000	33918	17
1000-1500	42045	21
1500-2000	53859	27
2000-2500	40179	20
2500-3000	23534	12
3000-3500	5464	3
3500-4000	793	-
>4000	19	-
total	199,812	100

Source (Abbay basin Master Plan)

3.4.1.3 Geology

Three main geological units have been distinguished.

i) Precambrian metamorphic rocks (32-35%) of the basin in the west.

ii) Mesozoic sedimentary formation (10-11%) of the basin

iii) Thick tertiary and quaternary volcanic formations cover (50-56%) in the north, center and east of the basin.

The Abbay basin has been widely influenced by the Precambrian tectonic activity that led to the formation of the rift system.

3.5 Climate

The climate of the Abbay basin is dominated by two factors: It's near equatorial location and an altitude ranging from 590 amsl to more than 4000m. The influence of these factors determines the rich variety of local climates, ranging from hot and desert like along the Sudan border, to temperate on the high plateau, & cold on the mountain peaks.

There are three recognized seasons:

- The main rainy season (kiremt) lasts generally June to September. About 70-90% of total rain occurs during this season.
- The dry season (bega) lasts from October to January.
- The minor rainy season (belg) lasts from February to May.

Mean annual rainfall over the basin is about 1400mm with mean evapotranspiration about 1300mm.

Four readily identified areas can be described according to their climatic conditions, two with relatively high rainfall and two with relatively low rainfall.

- A southern area covering much of east and west Wellega, Jimma and Illubabor zones (region-4). This area is characterized with relatively high rainfall (1400-2200mm) and long wet season which provides good conditions for agriculture.
- A central area covering most of west Gojjam (region-3) as well as parts of neighboring zones of region 3 and 6. This also has relatively high rainfall (1400-2200mm) but there is a more pronounced seasonal pattern.
- An area covering most of the eastern part of the basin including north and south Wello, eastern parts of Gojjam and south Gondar(all in region 3), and much of north and north west Shewa (region 4), but excluding small areas in the mountains. This area is characterized by relatively low annual rain fall (<1200mm) distributed in both rainy seasons according to a bimodal pattern. It represents the main drought-prone area in the basin.

- A west and North West area covering north Gondar and much of region 6. This has relatively low rainfall (<1200mm) falling predominantly in the main rainy season, and high average rates of evapotranspiration.

3.6 Water resources and land cover

3.6.1 Surface water resource

The water resource of the basin is dominated by the Abbay river which rises in the center of the catchment and develops its course in a clockwise spiral.

At the border, average annual discharge is 49.4BCM (billion cubic meter), with the low flow month (April) contributing to less than 2.5% of the flow for high flow month (August).

3.6.2 Sediment

The annual sediment discharge of the basin is between 130Million tones/yr (as assessed from sedimentation in rosaries reservoir) and 335Mt/yr as estimated by the consultant based on field sediment measurements. The higher measurement is equivalent to an annual sediment yield from the total catchment area of 1700t/km^2.

3.6.3 Land cover

Almost the entire high land area is now under farm land. Further, nearly all cultivation (about 90%) occurs in the high lands. This is a strong indicator of the problems faced by small holders in extending cultivation in to the low lands.

4 Hydropower demand and potential assessment in the Abbay basin

4.1 Energy demand survey

Evaluation of energy demand is one of the challenging aspects of small hydropower study in rural electrification programs in developing countries. Data is scarcely available and survey methods are bound with difficulties (FAO 1996). This is partly because of the problem of quantifying the diverse sources of energy which people in rural areas utilize. Commonly, poor households and communities rely on diverse sources of energy, using one fuel for heating, another for cooking or lighting, and another for agricultural or other productive activities (Zelalem-2002). Unreliable supplies have made rural households dependent on diverse sources of energy.

The task of estimating the load growth factor in newly electrified rural areas is as equally challenging as estimating the demand. This is due to the absence of a reliable socio-economic database for the estimation of energy consumption levels and the trend in the growth of consumption, especially of the communities and districts are still at initial stage of electrification. There are different forecasting methods and each of the forecasting methods uses a different approach to determine electricity demand during a specific year in a particular place.

Forecast of electrical demand for all existing and potential service areas is required for

1. Layout and design of distribution networks
2. Load flow studies
3. Economic and financial evaluation of the networks
4. Determination of the amount and timing of secondary energy and capacity available for export.

For purposes of forecast, the supply area may be divided in to two primary elements

1) Currently supplied demand centers
2) Currently unsupplied demand centers

In this thesis work, the currently unsupplied centers focused on. Energy demand forecasting for these centers will be considered in the next sections.

4.1.1 Forecast for currently unsupplied centers

The majority of towns in Ethiopia have no access to electric energy. Therefore; there exists no record of electric consumption on which to base a forecast of electrical demand. In this study, some of these towns found in the Abbay basin (within the Amahara region) are selected as target area.

Factors identified to have an effect on amount of electricity consumed (demand estimation) are:

1) Demographic characteristics, 2) economic characteristics, 3) prices of electricity and 4) non economic characteristics such as weather and development policies.

4.1.1.1 Number of residential connections

1) **Population**-the first step is to estimate the total population at each community over an assumed study period. The population growth rate of 4.07% per year is considered as per the 1994 census

2) **Occupants per house hold** -the population forecast is converted to the total number of potential connections by dividing the population by the average number of the occupants per house. An overall average household size of 4.63 persons per house hold can be used (Zelalem-2000).

3) **Market penetration-**Data for 31 electrified urban centers in Ethiopia indicates that approximately 35% of the house holds are connected to the system. The penetration rate does not include an allowance for the observed practice of connecting several houses to a single meter. For a variety of reasons, some house holds in a given community will not be connected to the supply system. This is perhaps due to insufficient house hold income, or the distance of the house holds relative to the town center does not justify the connection (Acres-1995).

An analysis of existing data for the EEPCo system was undertaken by Acres international to estimate an approximate upper bound on the number of potential connections. This upper bound represents the maximum connection level. Based on the above analysis, this upper bound has been estimated to be 35%. In addition not all house holds will be connected within the first years of electrification. Thus an initial connection rate was determined and a growth curve over 10 years to the maximum connection rate was developed based on the available data (Acres-1995). The initial penetration rate adopted is 10%. This level is assumed to grow gradually at a rate of 15%.

4.1.1.2 Average residential consumption

For conversion of potential connections to electricity forecasts, average residential consumption is required.

Average residential consumption per residential connection in six urban centers (Addis Alem, Bonga, Ginchi, , Sendafa, Tefki, and Yirga chefe) with population less than 10,000 was 280kwh/yr(Acres-1995). This residential demand was determined based on past sales data (Acres-1995). In addition, residential consumption was determined based on common house hold appliances, based on house hold expenditure on energy for which conversion was made by EEPCo (Zelalem-2002) and based on correlation with gross domestic product to help compare the results of the different approaches. Finally the average consumption of 280kwh/yr as determined by Acres-1995 has been assumed for residential demand, (Zelalem-2000), which is also used for the present thesis.

Over the period 1978-1990, consumption per connection in the 6 towns displayed an annual growth rate of 5 %(Acres-1995). Therefore; this growth rate associated with the average consumption of 280kwh/yr have been used in the forecast.

4.1.1.3 Average non-residential consumption

Consumption patterns in population centers with electrical supply were reviewed to determine the relation between residential and non-residential use.

Non residential use includes commercial, street lighting and small industrial activity. No large industrial activity is assumed for the unsupplied towns. Non residential consumption is estimated at 123kwh/yr for small commercial, 40kwh/yr for street lighting and 522kwh/yr for small industries. Total non residential consumption is 685 kwh/yr and the associated annual growth rate is 8.25% based on the analysis of the six urban centers (Acres-1995).

The provision of energy demand aims to satisfy the energy consumption of all sectors fully. However, use of the total demand will exaggerate the installation capacity of isolated power plants. It is expected that the industrial demand is likely to arise several years after energy supply has commenced. When the demand becomes large enough to absorb the industrial demand, the load centers can be economically connected to the ICS (Zelalem-2000) .Based on this, reasoning, the non residential consumption is taken to be 163kwh/yr excluding the industrial demand.

4.1.1.4 Generation requirements

The forecasts which are expressed as sales forecasts are converted to generation requirements by the application of a loss rate of 12 %(i.e. losses are 12% of generation). An overall system load factor of 45% is used to convert the energy forecasts to their corresponding peak demand requirements. This system load factor is assumed to grow from 35%-45% within the operation period of 20 years. This low level is taken because effective utilization of the power capacity is unlikely for newly electrified load centers.

4.1.1.5 Demand projection

The projection of the various demands for the load centers is determined according to the flow chart in fig3.1-1.The projected demands indicate greater peak value and it is the arithmetic sum of the projected values of the three demand types, i.e.residential,street lighting and commercial. The actual peak values assume reduced magnitude if the load is staggered according to the daily load variation in fig 3.1-2

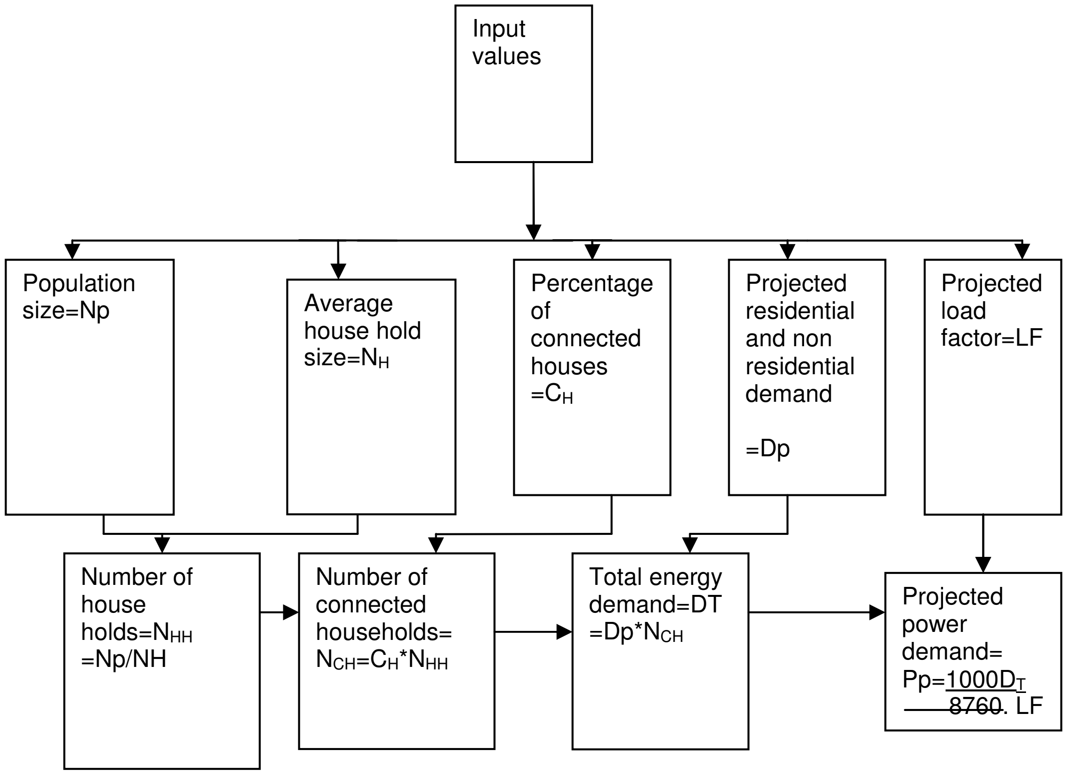


Figure 4.1-1 flow chart of demand projection

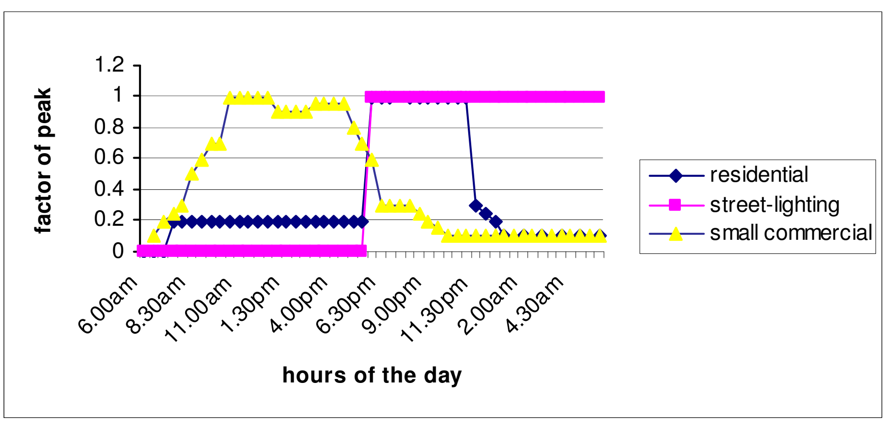


Figure 4.1-2 daily load variation pattern (after TCE 1998)

Therefore, a reasonable value of peak demand is obtained by applying the pattern of daily load variation to the first three demand items.

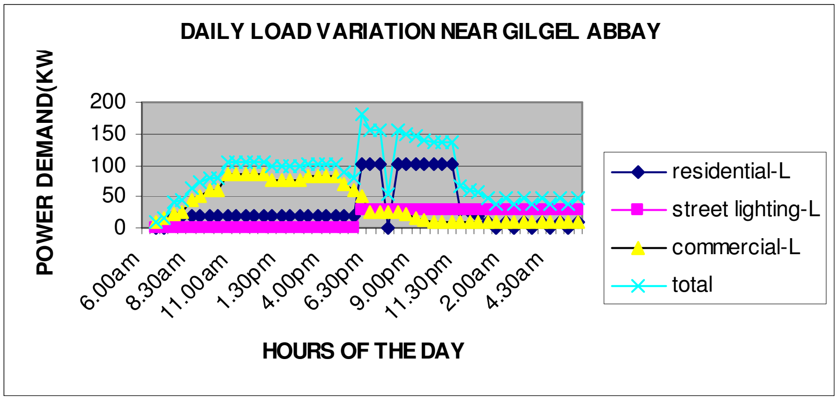


Figure 4.1-3 daily load variation near Gil Gel Abbay

4.1.1.6 Demand centers within the Abbay basin

The study is based on the demand centers which are listed by the central Statistical Authority's June 1998 report as not electrified towns and compiled by EEPCo by including other towns. The demand centers are out of the 300 towns proposed to be electrified during 2001-2005.Unfortunately there is no yet an official schedule or accepted method to prioritize the connections of these proposed towns (Acres-2000).EEPCo's planning office has been communicated to get information on criteria for rural electrification, but there is no criteria set for this purpose. There are numerous small villages in the basin which suffer the setbacks of energy shortage. However, it would be beyond the scope of this thesis to establish an energy distribution system which covers all these small villages.

For solving the energy shortage of small villages, a combination of all forms of renewable energy could be considered and the problem could be solved. Fig 3.1-4 shows selected demand centers and small hydropower sites used for the thesis. (The geographic coordinates are given in the appendix).

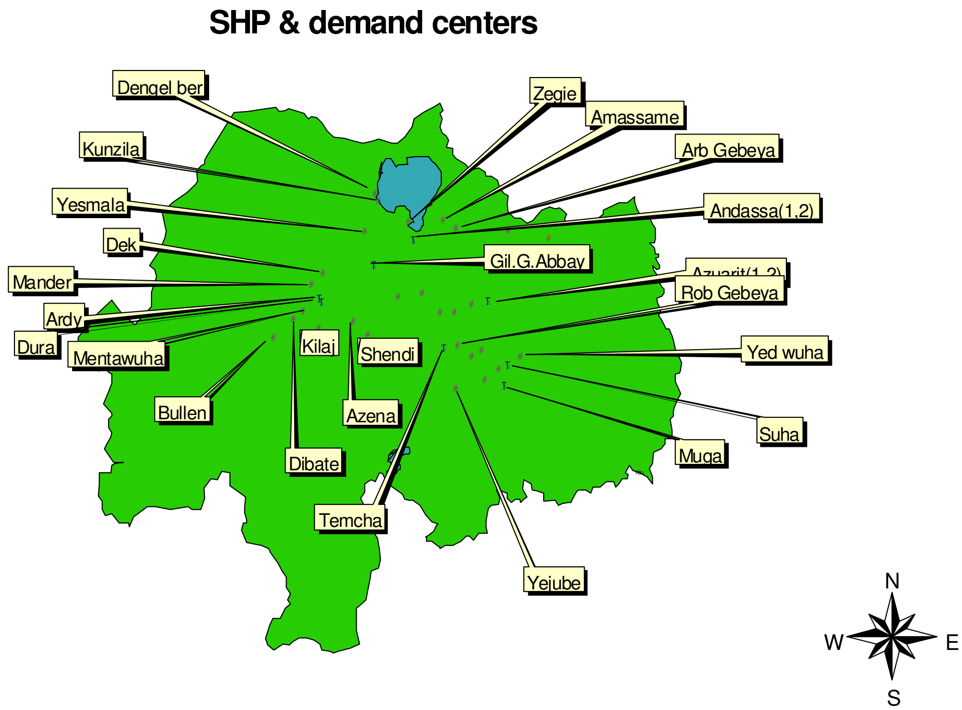


Figure 4.1-4 SHP sites and demand centers

4.1.1.7 Quantification of demand for rural load centers

The procedure described above is applied to the data of selected load centers to estimate their energy demand. The load for identified sites is tabulated below.

For quantifying demand for rural electrification, the approach used by EEPCo has been also applied and resulted in less demand as compared to the above method. In addition, no information was obtained upon which EEPCo method has been based. Therefore; the method described above is used for this thesis. The quantified demand using the two approaches is included in the appendix for comparison.

Table 4-1 Demand centers near sites and corresponding power demand

s-no	Rural town	Power demand	Power demand(adjusted)	s-no	Rural town	Power demand	Power demand(adjusted)
1	Merawi	1202.61	1009.37				
2	Durbete	1007.23	845.38	24	Kuy	280.11	235.10
3	Dengel ber	181.78	152.57	25	Yebkile	137.72	115.60
4	Kunzila	433.65	363.97	26	Wejel	142	125
5	Arb Gebeya	346.58	290.89	27	Lumame	716.35	601.25
6	Mekane yesus	1197.30	1004.91	28	Yejub	485.47	407.47
7	Arb gebeya	396.85	333.08	29	Rob gebeya	59.47	49.91
8	Ammasane	318.47	267.29	30	Sire	994.40	834.62
9	Tis abbay	547.67	459.66	31	Yetmen	142	125
10	Merto lem	822.60	690.42	32	Arb gebeya	87.33	73.29
11	GindeWoin	512.04	429.76	33	Arjo gudatu	382.08	320.69
12	Sede	174.26	146.26	34	Garlo	69.58	58.39
13	Diguatsion	112.98	94.83	35	Guyi	349.95	293.72
14	Feres bet	219.74	184.43	36	Kotu	119.85	100.59
15	Gebeze mariam	260.16	218.36	37	Aliyu	204.97	172.64
16	Gishe Abbay	253.81	213.86	38	Sheno	700.68	588.09
17	Debre zeit	314.71	264.14	39	Mendita	326.76	274.26
18	Dibbate	377.29	316.66	40	Deneba	346.58	290.90
19	Menta Wuha	611.02	512.84	41	Bullen	422.90	354.95
20	Kilaj	298.51	250.55	42	Shendi	696.66	584.72
21	Mander	187.61	157.46	43	Azena	293.85	246.63
22	Dek	203.41	170.73	44	Yesmala	398.54	334.50
23	Yed Wuha	216.11	181.39	45	Ellias	465.13	390.39

Note: Both power demand and adjusted Power demand are in KW (kilo watt)

4.2 Hydrologic analysis

The purpose of hydrologic evaluations is to provide a value or values of stream discharge that can be used in selecting the size of the power plant units and to determine annual energy production. This implies that site specific hydrologic data are needed to give the time variation of stream discharge. Rarely is a stream gage located at the desired hydropower site.

For estimates of potential energy that might be developed at a specific site it is convenient to use the average flow value for the year as the flow value in the power equation. Using an average annual flow value and an average value for topographic head in the power equation and multiplying the results by the total number of hours in a year gives the total energy available in KWHs(J.J.Fritz-1984).

The common method of describing the flow available at a site is through the use of flow-duration curve. A flow-duration curve is a graphical representation of the average time availability of flow. The curve is a plot of flow versus the percent of time that particular flow is equaled or exceeded. The curve is computed from sequential list of flows that are representative of flows available for power production at a particular point of interest in a stream. The curve is a very useful tool in hydrologic analysis in general and especially useful for hydropower studies. In hydropower analyses the flow-duration curve can be used to determine estimated power and energy from a proposed hydropower installation. The flow-duration curve provides information on low flows which are necessary to design storage capacity and /or select appropriate turbines. Therefore, a careful determination of flow duration curves is an essential part of the hydropower system development.

Normally, design is made by reading flow values directly from the flow duration curve. It is also possible to fit an equation to the flow duration curve using least square techniques. That way, the flow duration curve can help in the analysis of different installation capacities at a particular site. Such analysis is necessary in the course of optimization to judge the viability of the site at different installation capacities. Either an exponential decay type or polynomial type of equation can be fitted to the curve. Hence, the energy corresponding to a chosen installation capacity can be calculated by applying integration methods.

4.2.1 Computation of flow duration curve

The two basic methods of computing flow-duration curves at gauged points are the ranked flow and class interval techniques.

In the ranked flow-duration technique the time series of flows is rank-ordered according to the magnitude of flow. The rank-ordered values are then assigned order numbers. The order numbers are then divided by the total number in the record and multiplied by 100-representing the percentage of time that a particular mean flow has been equaled or exceeded during the period of record analyzed.

The class interval technique is slightly different, in that each of the time series flow values are categorized into class intervals. These classes of flows range from highest to lowest value of flow in the time series. A tally is made of number of flows in each class and the number of values greater than each class can be determined. The number of values greater than each class is divided by the total number of flows to get the percent exceedance. This percent exceedance is plotted versus the upper class interval to get the flow-duration curve values. This technique is usually faster than the rank ordering technique, especially where the time series of flow is lengthy.

For this thesis the ranked flow technique is used as it gives more correct results than the second method which averages out extreme events.

4.2.2 Gauges in the Abbay Basin

The collection of discharge data for the flow duration analysis is very important. There are about 122 gauge stations for Abbay river basin. In this thesis, discharge data has been transferred to the hydropower sites identified within the basin, from the near by gauging stations. The discharge data is obtained from ministry of water resources, hydrology department.

The geographic distributions of the gauging stations within the basin are shown in fig3.2-1

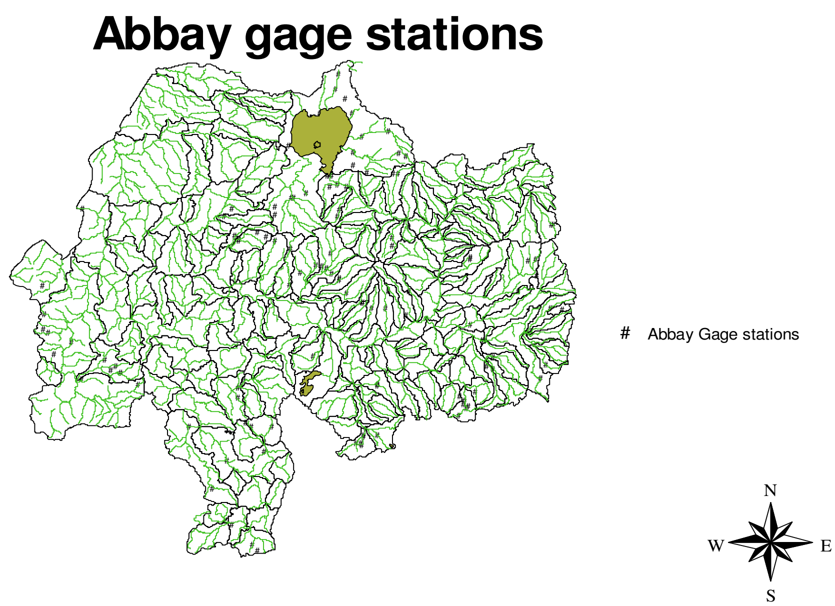


Figure 4.2-1 Abbay basin Gauging Stations

4.2.3 Development of flow Duration curves

4.2.3.1 Length of flow Records

Thirty years of historical stream flow data is generally considered to be the minimum necessary to assure statistical reliability (US corps of engineers Manual). However, for many sites, considerably less than 30 years is available. Most of the gauging stations on small rivers have got historical data less than the minimum required i.e., 30 years. For sites with shorter records, correlation and regression techniques can be applied to extend a period of record. For this thesis, near by stations with sufficient concurrent records on monthly basis have been selected and the trend line has been fitted. This equation has been used to fill the missed data and applied to extend the data where necessary.

4.2.3.2 Transfer of flow duration curve

Although a flow-duration curve is usually available for the gauging stations in a region, it is unusual for one of these gauges to be located precisely at the hydropower site of interest. There is, however, often a gauge located on the same river or a down stream river with a drainage area (DA) containing the site's watershed. The data from one or more gauges may then be adjusted to represent that of the site. Equation 3.2-1 is the common type of relation that is used to estimate flow-duration at a site (Gulliver and Roger-1991).

$$Q_{site} = \left[\frac{DA_{site}}{DA_{gauge}} \right]^n Q_{gauge} \qquad \text{Equation 4.2-1}$$

Where DA_{site}-drainage area of the power plant site

DA_{gauge}-drainage area of the gauge

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

Q_{gauge}-discharge at gauge(m^3/s)

n –a parameter typically varies between 0.6 and 1.2.

When the value of n in equation 3.2-1 is set equal to 1, the above relationship turns out to be a case of data transfer by proportion of the areas. Usually, the proximity of the site to the gauge is used as a basis for the choice of the value of n. Some guide lines for the choice of n are as follows.

1. If the drainage area (DA) site is within 20% of the DA of gauge ($0.8 \le \frac{DA_{site}}{DA_{gauge}} \le 1.2$), use n=1. The estimated discharge at the site will probably be within 10% of the actual discharge, which is normally sufficient.

2. If the DA site is within 50% of the DA gauge, consider whether the data of the two gauges (up stream and down stream gauges) can be combined. In addition, when a weighted average between upstream and downstream gauges is possible, the following linear interpolation (equation 3.2-2) may be applied for a site lying between upstream and down stream gauges (Gulliver and Roger-1991).

$$Q_{site} = \frac{(DA_{gauge1} - DA_{site})Q_{gauge1} + (DA_{site} - DA_{gauge2})Q_{gauge2}}{DA_{gauge1} - DA_{gauge2}} \qquad \text{Equation 4.2-2}$$

The daily flow data from the two gauges should be used to compile a new set of daily flow data for the site. A flow-duration curve is compiled from the new flow data.

For this case, comparing watersheds may be helpful because Q_{site} may be off by 30%.

If there is a partial discharge station near the site, it can give an indication of the proper value of n. The ratio of partial discharge to gauge discharge on the same day versus gauge discharge is plotted. The average of these values may be used to estimate n.

3. If the DA site is only within 80% of the DA gauge, the recommendation is to do everything listed above if possible. In addition, when discharge versus DA is plotted for all gauges in the watershed basin, value of n may be identified by these data. Then one may ask whether the drainage area is relatively wet or dry.

Table 4-2 sites and their features

s-no	River Name	$DA_{site}(Km^2)$	$DA_{gage}(Km^2)$	Head(m)	Length(Km)*	Gradient of river bed(%)	Altitiude(masl)
1	G-Abbay	1118	1664	6.0	66.5	13.5	1900
2	Andassa(1)	401	573(nr-Bahir Dar)	5.4	65.7	11.4	1900
3	Andassa(2)	401	573	11.6	65.9	11.6	1900
4	Andassa(3)	401	573	11.6	68.2	11.8	1900
5	Teme	119	156.3(nr-Mota)	76.0	35.5	18.3	2300
6	Azuari(1)	191	209(nr-Mota)	5.0	40.3	19.6	2400
7	Azuari(2)	191	209	76.0	40.4	19.7	2400
8	Andassa-br	298	573	11.6	27.9	20.4	2000
9	Suha	488	359(nr-Bichena)	92.1	49.8	11.1	2300
10	Muga	524	375(nr-Dejen)	47.3	79.4	15.8	2200
11	Temecha	400	406(nr-Dembecha)	14.0	38.2	25.4	2030
12	Fettam(3)	720	282(nr-Tilili)	58.0	89.1	12.3	1600
13	Fettam(2)	305	282(nr-Tilili)	18.0	48.5	10.5	2100
14	Fettam(1)	216	282(nr-Tilili)	25.0	28.0	6.43	2400
15	Ardy	251	219(nr-Metekel)	14.5	52.8	16.2	1550
16	Dura	619	539(nr-Metekel)	26.0	74.8	14.0	1550

*-length of the river from upstream to site

4.2.3.3 Comparison of flow-duration curves from monthly and daily data

A flow-duration plotted by monthly average flows has a tendency towards a higher design discharge that some times produces quite large errors. This is because the monthly average used will mask the within-month variation. Consequently, it is necessary to obtain an average daily flow series, as far as possible, in order to build a daily flow duration curve (Tong Zheng Wang Hai Ding-1996).

Ramsahoye 1982 considers the determination of the average daily flow-duration curve as the unquestionably most important hydrological input for the dependent power, energy, and economic studies of a hydropower project. Specially when it is desired to provide investors and lenders with certainty, conservative estimate of stream flow is considered appropriate, Eichenberg 1999. However, the analysis based on monthly data is useful to make preliminary insight in to the design.

For this thesis, it was intended to use daily flow. But obtaining the average daily series has been impossible because it is classified information .Hence; monthly data has been used to plot a flow-duration curve.

4.2.3.4 Flow Duration Curves for Selected small Hydropower sites

For each of the selected hydropower sites, flow duration analysis is made using the methods mentioned above and an appropriate equation to the flow duration curve is fitted so that quick estimate of the flow corresponding to any given percentage of exceedance can be made. This is because optimization is carried out by considering different alternative hydropower developments with different power capacities.

One of the important advantages of representing the flow-duration curve with a mathematical equation is to simplify the calculation of the annual energy generation levels by integration methods at any specified level of exceedance. Moreover, the power-duration curve can easily be derived from the flow-duration curve for known head. Fig 3.2-2 shows the flow duration curve for one of the selected sites.

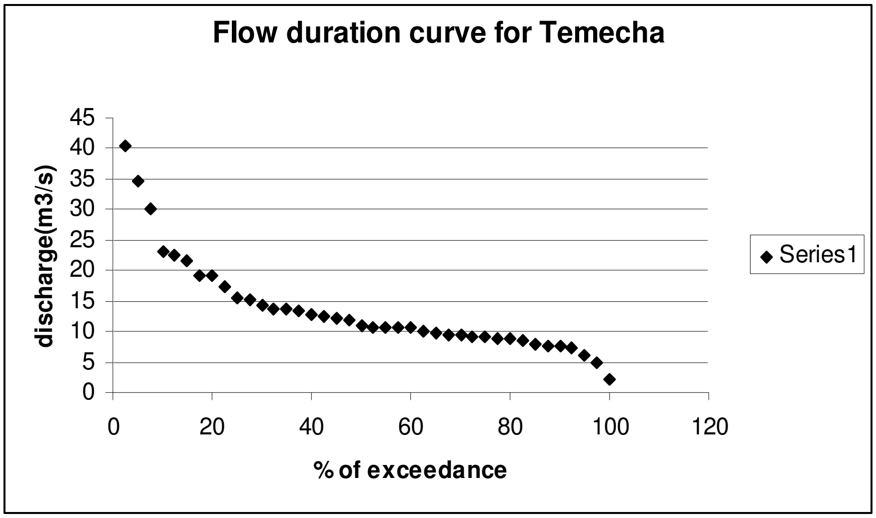


Figure 4.2-2 Flow duration curve for Temcha River

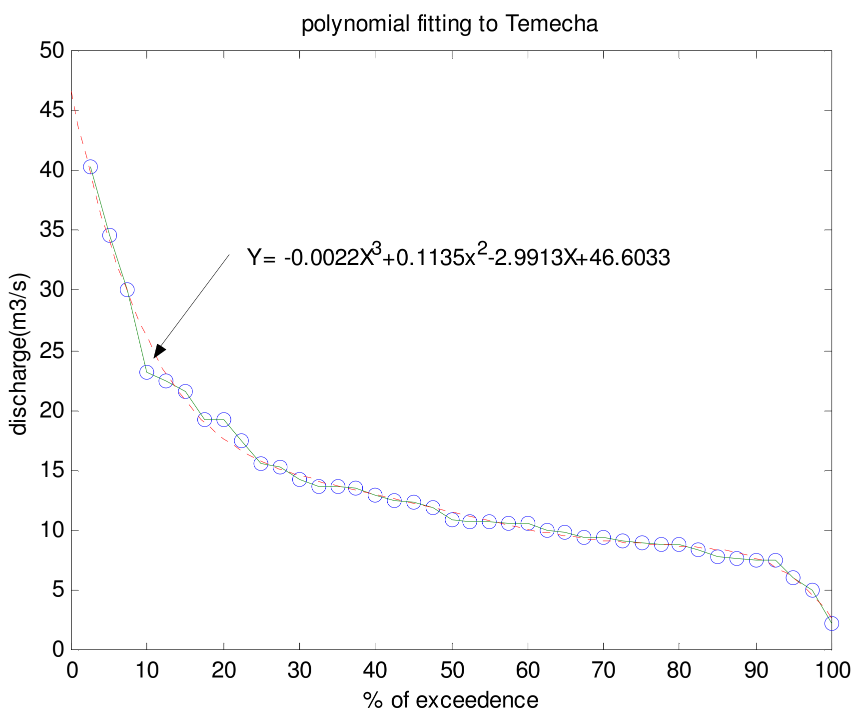


Figure 4.2-3 Polynomial fitting to Temcha.

4.2.3.5 Summary of Dependable Flows at the various sites

The hydrological analysis helps the determination of the dependable flows at the various sites which will be used to identify the limits of power potential at the alternative sites. These will serve the purpose of providing the constraints of the small hydropower system for the optimization. For site identified, the dependable flow for 90%, 85%, and 80% are determined and shown in table 3-3

Table 4-3 summary of dependable flows at various sites

S-No	Site name	Q90	Q85	Q80
1	G-Abbay	34.51	36.27	37.23
2	Andassa(1)	4.33	4.66	5.03
3	Andassa(2)	4.33	4.66	5.03
4	Andassa(3)	4.33	4.66	5.03
5	Teme	1.13	1.255	1.306
6	Azuari(1)	1.644	1.99	2.47
7	Azuari(2)	1.644	1.99	2.47
8	Suha	2.61	2.92	3.25
9	muga	4.75	5.15	5.55
10	Temecha	7.49	7.76	8.2
11	Fettam(3)	22.56	23.278	23.7
12	Fettam(2)	7.918	8.174	8.318
13	Fettam(1)	5.86	6.048	6.16
14	Ardy	4.44	4.73	4.9
15	Dura	20.3	20.47	21.41
16	Fato	6.44	6.64	6.77
17	Belo	2.51	3.14	3.71

4.3 Hydropower potential assessment in the Abbay basin

This section bases itself on information from section 3.2 and on site specific topographical information to establish the limits of hydropower potential at each site. These limits serve as the constraints for the hydropower system. Before estimating the site specific potential, some country-wide potential is introduced below.

4.3.1 The Hydropower Potential of Ethiopia

Ethiopia has a vast hydropower potential, which is estimated to be about 15,000 - 30,000 MW. So far very little percentage (less than 2%) of the vast potential has been harnessed. In order to develop this vast potential of power several projects have been initiated to generate more and more hydroelectric power. Some 300 hydropower plant sites in the whole river basins of the country with a total technical power potential of 159,300 Gwh/year have been identified. Out of these potential sites, more than 100 are large scale (more than 60 MW) and the rest are small (less than 40 MW) and medium scale (40-60 MW) hydropower plant sites.

Table 4-4 Hydropower Potential of Ethiopia

Name of River Basin	Number of Potential Sites					
	Small Scale 40 MW	Medium Scale 40-60 MW	Large Scale > 60 MW	Total	Technical Hydropower Potential (GWH/year)	Percentage Share of the Total %
Abbay	74	11	44	129	78,800	48.9
Rift Valley Lakes	7	-	1	8	800	0.5
Awash	33	2	-	35	4,500	2.8
Omo – Gibe	4	-	16	20	35,000	22.7
Genale – Dawa	18	4	9	31	9,300	5.8
Wabi Shebelle	9	4	3	16	5,400	3.4
Baro Akabo	17	3	21	41	18,900	11.7
Tekeze – Angereb	11	1	8	20	6,000	4.2
Total	173	25	100	300	159,300	

Source (EACE bulletin vol1,no1,1998-by Solomon Seyoum Hailu)

4.3.1.1 Large Scale Hydropower Projects

The favorable sites for Large Scale Hydropower Development Scheme within the river basins of Ethiopia number more than 100 and are fairly distributed throughout the width and breadth of the country (Solomon-1998). As the development of these schemes requires huge investment, they are not in the priority list by the government (Solomon-1998).

4.3.1.2 Medium Scale Hydropower Projects

The promising and candidate sites for the development of Medium Scale Hydropower Development number more than 20(Solomon-1998). From these potential sites three in Tekeze, three in Gojeb and one in the Blue Nile Basin had been selected for studies (Solomon-1998). One of the sites in Tekeze is presently under construction and the construction of one of the sites in Gojeb is expected to start the next fiscal year (EEPCo). In addition a consortium led by Lahmeyer has completed feasibility studies of several projects. They are Halele, Chemoga-Yeda stage 1 and Beles. In 1999, Norplan/Norconsult completed four pre-feasibility studies of major hydroelectric developments. Baro, Genale and Geba were found to be attractive (Acres-2000main report).

4.3.1.3 Small Scale Hydropower Projects

The potential sites for small Scale Hydropower development are immense in number. The development of these potentials needs to be given special attention and encouraged along with the Medium Scale Hydropower Schemes especially in the rural areas of the country. Ways and means should, therefore, be sought and facilitated in harnessing small hydropower resources in Ethiopia even if it is not encompassed within the top priority lists. These are areas where private participation should be fully supported and encouraged in developing these untapped resources without any limitations.

4.3.2 Grid connection option to load centers

As far as the grid connection option as alternative to isolated small hydropower system and diesel is concerned, to be considered as a source of supply to small demand centers within the basin, it must be designed starting either from an existing substation with surplus capacity or from a substation to be specially constructed and fed by an existing high voltage transmission line. The feasibility of options depends on the distance between the town or settlement to be supplied and the available capacities of existing lines and substations. The investment costs for the transmission line itself do mostly not depend on the capacity to be transmitted, because, generally, a standard type of wiring is applied which is oversized and thus also copes with stronger load increases. If a 132KV line passes nearby but no substation is available, theoretically, a 132/33Kv step-down transformer would be needed, whereby the transformer should be of two-winding, three phase type. In addition, one 33KV outgoing feeder bay is required in the substation in order to control and protect the supply system through the use of 33KV circuit breaker, lightning arrestors, etc. the construction of additional substation explicitly, for the supply of a relatively minor load center requiring less than 300KW is quite expensive and not competitive with the MHP or diesel genset option.(Hedi Feibel-2003) has shown that the cost estimation for the connection to an existing grid, more precisely to an existing substation with excess capacity, is competitive to an MHP system for distances of up to about 25-35km. According to her study the rough investment cost for connection to an existing grid/substation, diesel genset and MHP system has been determined and was found to be 1742USD/KW, 1294USD/KW and 1689USD/KW respectively. If neither transmission line nor substations are available close to the project area, which is the most probable case in rural Ethiopia, the costs for an additional transmission line and a substation have to be considered. In 1994, it was found that very little room for expansion exists from 15KV supply stations (Hedi Feibel-2003). The feeders emanating from these stations have generally been extended to the limits of acceptable voltage drops, and, in most cases, further expansion would require additional line exists and express feeders with very large conductor size. Thus most cases necessitate substation construction or even extension of 66KV, 132KV or 230KV transmission lines, this makes grid connection option uncompetitive with small hydropower. Fig 3.3-1 shows the committed and existing transmission lines in the Abbay basin, developed as new theme during the thesis work.

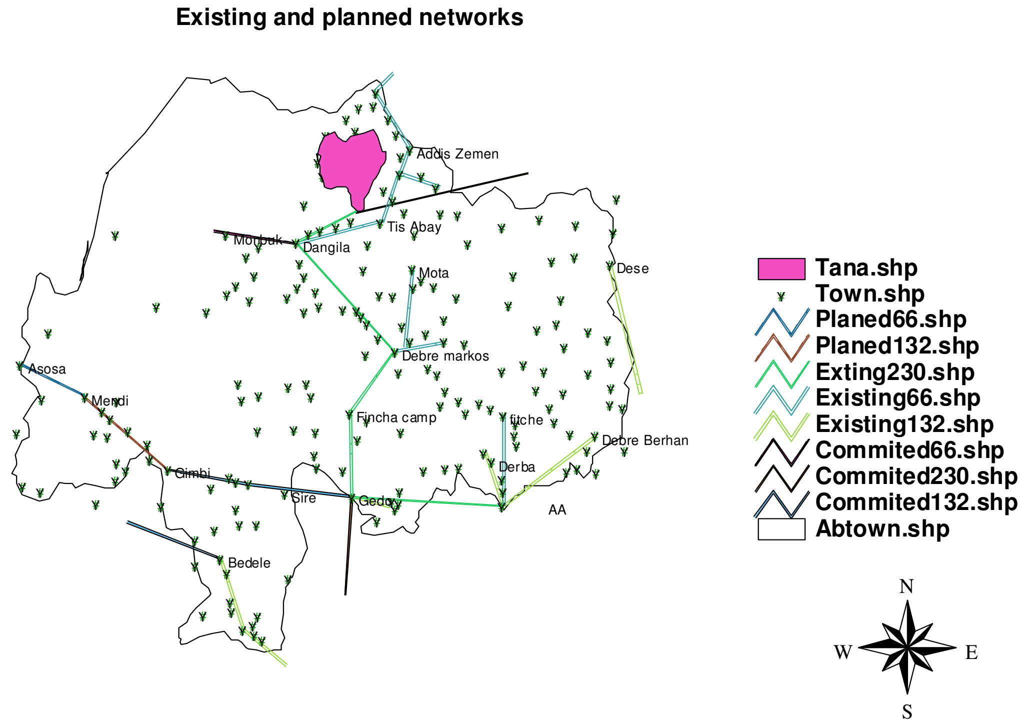


Figure 4.3-1.Transmission line in Abbay basin

4.3.3 Expected energy supply and demand balances

There are some optimistic signs that indicate the energy supply can improve in Ethiopia. Beginning in 2003, projects are already in hand to provide a suitable balance for the years immediately a head. These include the Gilgel Gibe hydroelectric power development currently inaugurated and the Tekeze and Gojeb hydroelectric projects which are under construction and the Aluto Langano geothermal scheme. With the completion of the projects under construction and the Gilgel Gibe the demand-supply balance is shown in table 3.5.

Table 4-5demand supply balance in Ethiopia

	Short term(to 2003)		Medium term(to 2012)		Long term (to 2025)	
	Energy (GWh)	Power (MW)	Energy (GWh)	Power (MW)	Energy (Gwh)	Power (MW)
Forecast demand						
-Moderate scenario	2065	487	3597	848	7517	1772
-Target scenario	2065	528	3892	918	9903	2335
Existing supply & capability	2659	654	4044	981	4044	981
Projected needs						
-moderate scenario	Nil	Nil	Nil	Nil	3473	791
-target scenario	Nil	Nil	Nil	Nil	5859	1354

Source (Acres-2000-main report)

4.3.4 Hydropower potential assessment in Abbay Basin

The Abbay basin accounts for the major share of the country's irrigation and hydropower potential. Hydropower potential in the Abbay basin is little developed (only 1.2% is utilized out of 55GWH/yr potential). Previous studies on small scale hydropower indicate that up to 7000MW exists within the basin.

In Abbay basin, previous studies have been conducted at preliminary level mainly by USBR and at less extent WAPCOS study. In addition, BECOM in association with BRGM and ISL and Acres international have conducted detail studies. For this thesis, the small scale hydropower sites studied by the study team from china have been considered.

From table 3.4 above it can be seen that the Abbay basin shares 48.9% of the total hydropower potential. Some of the sites identified in Abbay basin by USBR (1964), which have been refereed by recent studies are shown in the figure 3.3-2

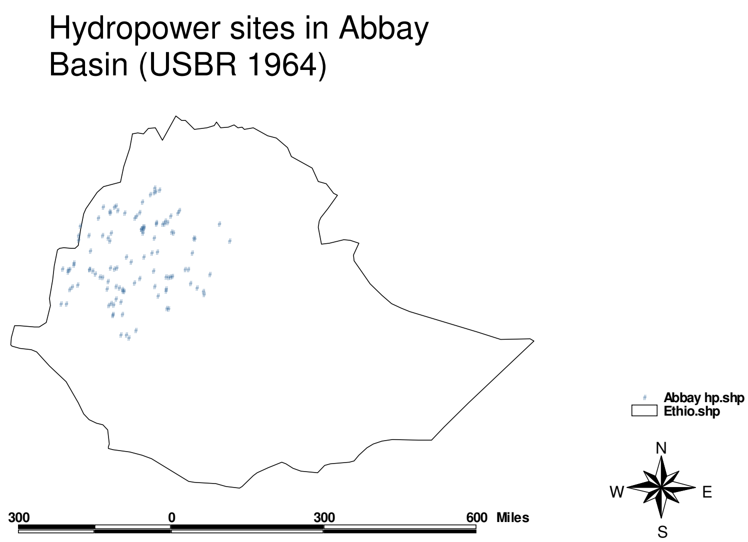


Figure 4.3-2 hydropower sites in Abbay basin (USBR1964)

4.3.5 Preliminary power potential of the considered sites

Using the dependable flows of table 3.3 and the corresponding head of the sites given in table 3.2 the preliminary power potential of the considered sites has been determined by using equation.

$$P = 9.81\eta_m Q_d H \qquad \text{Equation 4.3-1}$$

Where P-power in KW

η_{m}=efficiency (assumed 75%)

Q_d=design discharge (m3/s)

H=head available for power production (m).

Table 4-6 Summary of preliminary power potential at selected sites.

S-No	Site name	Head (m)	P90(KW)	P85(KW)	P80(KW)
1	G-Abbay	6	1523.44	1601.14	1643.52
2	Andassa(1)	5.4	172.03	185.14	199.84
3	Andassa(2)	11.6	369.55	397.72	429.3
4	Andassa(3)	11.6	369.55	397.72	429.3
5	Teme	76	631.9	701.8	730.3
6	Azuari(1)	5	60.5	73.2	90.9
7	Azuari(2)	76	919.3	1112.75	1381.15
8	Suha	92.1	1768.6	1978.7	2202.3
9	Muga	47.3	1653.0	1792.3	1931.5
10	Temecha	14	771.5	799.3	844.6
11	Fettam(3)	58	9627.1	9933.5	10113.6
12	Fettam(2)	18	1048.6	1082.5	1101.6
13	Fettam(1)	25	1077.9	1112.5	1133.0
14	Ardy	14.5	473.7	504.6	522.75
15	Dura	26	3883.3	3915.8	4095.6
16	Fato	14	663.35	684.0	697.3
17	Belo	16	295.5	369.6	436.7
		Total	25374.82	26642.3	27983.3

4.3.6 Comparison of Demand with Supply potential

Comparing the calculated power potential with the sum total of demand of load centers within the basin from table3.6 and table3.1it can be seen that the total power that can be developed with 90% of dependability is 25375KW where the demand of the selected load centers is 15538KW.

This indicates that the demand of the load centers within the basins could be covered with unregulated supply of 90%.

5 Cost of small hydropower plants

One of the biggest challenges facing investment analysis of hydropower projects in developing countries is the absence of a quick and reliable basis for estimating capital costs. The high variability of KW and KWH prices of small hydropower systems world wide prevent realistic cost estimation for small hydropower plants under Ethiopian conditions where almost no experience in this field is available. The costs of MH plants for electricity generation schemes ranged from USD1136 (Pucara-Peru) to USD 5630(Pedro Ruiz-Peru) with an average installed cost of USD3085 (Smail Khennas and Andrew Barnett-2000). Hydropower plants involve major civil, mechanical, and electrical engineering components. The multi-disciplinary nature of the development requires the coordinated team-work of an inter-disciplinary design group which is absent in developing countries (Zelalem-2002).

A well known characteristic of all hydropower developments, but especially small hydro, is the site specific nature of the types and costs of civil works, mechanical and electrical equipment. These site specific situations require very specific cost and economic analyses. Under these situations there are no “rough” cost guidelines. That means, each hydropower site has unique features which makes it difficult to transfer cost information without considering these features to some extent.

The adoption of cost functions specific to particular sites requires the consideration of project costs from a number of power plants. Countries with long experience of small hydropower have developed such functions. But this may not correspond to the power market conditions in Ethiopia. Therefore, it is necessary to establish a better approximation (Zelalem-2002).

5.1 Non linear nature of hydropower cost functions

Specific cost can be described in the form of an equation with the cost as a function of power and head. Alternatively, it can be defined as a function of power only. Some functions in use are described below:

1) Gordon and Roger(1991)

$$C_T = (16100 KW^{0.82} HR(m)^{-0.35}).sitefactor. \qquad \text{Equation 5.1-1}$$

Where CT=Total cost of hydropower project

KW=total plant capacity in KW

HR=rated head in m

In the above equation, the term in bracket results in satisfactory equipment cost estimates with (±20) percent for a plant capacity range from 50 to 40,000KW and a hydraulic head range from 4 to 100m.

Civil works (construction) costs are an important, undefined variable in hydropower development because the costs are very site specific. An estimate of the cost of hydropower project, therefore, requires prior knowledge of the civil works costs associated with the site. To give consistency to these estimates, Gordon and penman introduced the concept of a site factor, which is the total project cost divided by the total equipment cost for a hydropower project.

2) Harvey 1993 suggests the following equation

$$C_T = F.(P)^{-0.3}.(H)^{-0.15} \qquad \text{Equation 5.1-2}$$

Where:

C_T=total cost in US dollars

F= constant in the range 3500-4500

P=power in KW

H=head in m

3) Dubach in Babanek 1982 recommends a function with power capacity as the only dependent variable for the approximate calculation of the total investment costs.

$$K_{DM} = \frac{9400}{P^{0.15}} = \frac{C_o}{P^n} \qquad \text{Equation 5.1-3}$$

Where, C_O=constant term for specific cases

P=Power in KW

K_{DM}=Specific cost in Deutschmark (DM)/KW

The above equations do not yield comparable values of the specific cost. Hence, it is necessary to develop site specific cost functions corresponding to a hydropower site than to apply any of them.

All of the above equations indicate the nonlinear form of the cost function which in turn indicates the need to apply nonlinear optimization techniques. The third type of relation ship is preferred in this thesis as it allows reduced number of decision variables in the optimization problem.

5.2 Specific cost functions for the sites in the Abbay Basin

To ensure reliable decision making, representative cost functions have to be developed based on local unit prices. Accordingly, cost functions are prepared for the alternative hydropower sites by treating the power capacities at the sites as independent variables. These specific cost functions will later be used to establish the objective function. The development of cost functions is based on design of the essential hydraulic components of small hydropower schemes. For instance the components of diversion type hydropower plant includes diversion weir and intake, head race canal, settling basin, forebay,penstock, power house, electro-mechanical equipment and other relayed accessories. Brief descriptions of the design principles for various components are made in the following sections. The FORTRAN programme written by Zelalem -2002 is used based on the description below.

5.3 Basic features of hydropower components relevant to the development of cost function

5.3.1 Diversion weir/Barrage

Hydro system must extract water from the river in a reliable and controllable way. The water flowing in the channel must be regulated during high river flow and low flow conditions. A weir/barrage can be used to raise the water level and ensure a constant supply to the intake. Sometimes it is possible to avoid building a weir by using natural features of the river. A permanent pool in the river may provide the same function as a weir. For most of the sites considered in this thesis, a masonry barrage is proposed by the study team from the P.R.China (1989).

5.3.2 Intake structures

An intake structure is necessary to reduce sediment load and to withdraw the desired amount of water. Intake structures as hydroelectric facilities consist of sluicing facilities for trash, ice, and sediment; fish protection facilities; trash racks; and a flow constriction to bring the water from the reservoir into the penstock. The intake structures are a matter of concern for design engineers because the depth of the intake, the horizontal location, the location of the various structures relative to each other, and the approach flow angle are site specific.

The intake of hydro scheme is designed to divert a certain part of the river flow. This part can go up to 100% as the total flow of the river is diverted via the hydro installation.

The following points are required for an intake

- The desired flow must be diverted
- The peak flow of the river must be able to pass the intake without causing damage.
- As less as possible maintenance and repairs
- It must prevent large quantities of loose material from entering the channel
- It must have the possibility to remove piled up sediment

From these points it follow that the positioning and shape of the intake are very important.

Different types of intakes are characterized by the method used to divert the water into the intake. For micro hydro schemes only the smaller intakes will be suitable. The following three types of intakes will be discussed here: the side intake with and without a weir and the bottom intake. For these types the advantages and disadvantages will be mentioned.

5.3.2.1 Side intake without weir

Relatively cheap, no complex machinery required for construction, requires regular maintenance and repairs, at low flows very little water will be diverted and therefore this type of intake is not suitable for rivers with great fluctuations in flow (Klunne-2002).

5.3.2.2 Side intake with weir

This type of intake normally consists of two structures, the weir and the intake. The weir is situated in the river and its function is to dam up the water level in order to ensure a stable minimum depth of water upstream of the weir and to allow the quantity of water for power production to be diverted from the river irrespective of the regime (Zelalem-2000). The weir used in this configuration can be partly or completely submerged into the water (Klunne-2002).

Side intakes are used on all types of rivers from mild sloping silt –and sand –bed Rivers to steep boulder-bed rivers or step-pool type of rivers. Side intakes Control water level, require little maintenance. The disadvantages with side intakes are: low flow can not be diverted properly, modern materials like concrete is necessary.

5.3.2.3 Bottom intake

The bottom intake combines intake and damming up in one structure (Zelalem-2000). The water to be diverted is taken in through a collection canal built in to the river bottom and covered with screen. The bars of the screen are laid in the direction of the tail water so that coarse bed load is kept out of the collection canal and transported further downstream. An interesting aspect of bottom intakes is that they can be adapted to include sediment exclusion and boulder impact resisting mechanisms (Drobir 1981).At a bottom intake the whole weir is submerged into the water. Excess water will pass the intake by flowing over the weir.

The major dimensions considered in the design of the intake are the length of the screen and the cross-section of the collecting canal. These dimensions can be determined from graphical design-chart or empirical formula in which the dimensions are related with the specific discharge. Because of the operation simplicity this type of intake are preferred. In addition the bottom intake is very useful at fluctuating flows, even the lowest flow can be diverted; no maintenance required.

5.3.3 Open channel

The channel conducts the water from the intake to the forebay tank. The total net head available for energy production will require careful considerations of energy losses through the conveyance system. Losses due to friction in the head race canal can be approximately calculated using the uniform flow equation. A reasonable slope should be chosen by laying the canal such that it follows a longer route along the contour line from the intake to the forebay so that friction head losses are minimized.

The length of the channel depends on local conditions. In one case a long channel combined with a short penstock can be cheaper or necessary, while in other cases a combination of short channel with long penstock suits better (Harvey-1993).

Most channels are excavated, while sometimes structures like aqueducts are necessary. To reduce friction and prevent leakages channels are often sealed with cement, clay or polythene sheet. Canals must be designed to convey water at velocities below that at which objectionable erosion will occur. Hence for unlined canals, additional consideration of protection measures should be made (Klunne-2002).

Size and shape of a channel are often a compromise between costs and reduced head. The best hydraulic cross-section gives the optimum cross-sectional area and perimeter corresponding to the design discharge (Tullis-1989).As water flows in the channel; it loses energy in the process of sliding past the walls and bed material. The rougher the material, the greater the friction loss and the higher the head drop needed between channel entry and exit.

One of the important cost components in canal construction is the lining cost. Therefore, the advantage of canal lining has to be carefully considered against the additional lining cost. Mini hydro plants around the world have usually found it economic to include a canal lining rather than omit it, and with out a lined canal, cleaning and maintenance can become a big problem for economic operation (Harvey-1993).

Where small streams cross the path of the channel very great care must be taken to protect the channel. A heavy storm may create a torrent easily capable of washing the channel away. Provision of a drain running under the channel is usually not adequate protection. It will tend to block with mud or rocks when needed the most. In the long term it is economic to build a complete crossing over the channel. Incorporated in the channel are the following elements, settling basin, spillway and forebay.

5.3.4 Settling basin

The water drawn from the river and fed to the turbine will usually carry a suspension of small particles. This sediment will be composed of hard abrasive materials such as sand which can cause heavy damage and rapid wear to turbine runners. To remove this material the water flow must be slowed down in settling basins so that the silt particles settle on the basin floor. The deposit formed is then periodically flushed away (Klunne-2002).

5.3.4.1 Design of settling basin

The hydraulic design of settling basins involves: 1) exploration of sediment conditions-involving the quantitative and qualitative analysis of sediment carried by the river.2) Determination of the necessary degree of removal on the basis of theory and practical experience. Consequently, the sand trap/settling basin must be dimensioned in such a way that grains with diameters bigger or equal to limit particle size must be settled. It should be noted, however, that no standard values or specifications have yet been developed (Mosonyi-1991).

For medium head (15-50m); d_1=0.3mm in diameter

For high head up to 100m; d_1=0.15mm in diameter

Very high head > 100m; d_1=0.03mm in diameter

Where d_1-limit particle size(mm)

Having determined the basic data as suggested above, one can proceed to establish settling velocity of the smallest fraction, i.e, of the limit particle size to be removed. This can be established theoretically (Stoke's law) or by experiments (Surdy graph). The so-called horizontal-flow settling system is usually applied at power developments. Neglecting the effect of turbulent flow up on settling velocity, the following basic relations may be written:

I) $Q = WDV$ Equation 5.3-1

Where, D=depth of basin, W=width, V=flow velocity, Q=discharge passing through the basin.

II) $t = \dfrac{D}{\omega}$ Equation 5.3-2

Where, ω =settling velocity, t=settling time

Finally, the length of the basin will be governed by the consideration that water particles entering the basin and sediment particles conveyed by them with equal horizontal velocity should only reach the end of the basin after a period longer than the settling time. Thus the settling particle may reach the bottom of the basin within the settling zone. In other words, the retention period should not be shorter than the settling time. The required length of the basin is thus:

III) $L = Vt$ Equation 5.3-3

eliminating t from II) and III) two relations can be established between six parameters governing the hydraulic design.

$Q = WDV, L = \dfrac{DV}{\omega}$ Equation 5.3-4

Where the discharge (Q) is usually known, the settling velocity (ω) is established from graph by L-Surdy for this thesis, and the highest permissible flow-through velocity V should also be specified, considering that particles once settled should not picked up again. According to Camp, the critical flow-through velocity is estimated from:

$V=a.d_1^{0.5}$ [m/s], where, d_1 is the equivalent diameter of the smallest sediment particle to be settled in mm and a is a constant given as:

a=0.36, for d>1mm, a=0.44, for 0.1mm<d1<1mm, a=0.55, for d1<0.1mm

Depth of the basin should be specified considering that long and/or wide basins are economical than deep ones. The depth of settling basins in water power projects is generally between 1.5 and 4m with flow through velocities not higher than 0.5m/s (Mosonyi).

5.3.5 Forebay tank

The forebay tank forms the connection between the channel and the penstock. The forebay receives water from the canal and distributes it to the penstock.

5.3.5.1 Functions of forebay

1. Distribution of flow –forebays should be long enough to distribute the flow smoothly and uniformly and be wide enough to accommodate the intake or penstock and other structures

2. Regulation of flow-the forebay should have a certain storage volume capable of regulating the flow.

3. Protection against silting and floating debris-the forebay is the final defense against harmful particles of silt and floating debris. Therefore, sand sluices and trash racks should be provided in the forebay.

5.3.5.2 Design guidelines for a forebay

The layouts and dimensions of the forebay are mainly determined by the topographical and geological conditions and the layouts of its other associated structures. The site of both the forebay and the power house should be selected simultaneously with a view to ensuring the shortest possible penstock/pressure shafts. The size of a forebay varies depending on the sediment content of the water conveyed in the power canal and whether it is to serve for storage. To be most cost effective, the forebay must be of size adequate to fulfill its function, neither significantly larger nor smaller. A gradual transition section should be provided between the power canal and the forebay basin. The bottom of the forebay basin should be provided with a proper slope to enable periodical flushing of the silt deposited. A bottom lining of the forebay basin is required in soils where large seepage is expected. In designing a forebay tank, it is important to keep the entrance to the penstock fully submerged. This is to prevent air being drawn in to the penstock because of vortex which can be formed if the penstock entrance is closer to the water surface in the basin. A spill way completes the forebay (Silesh-2003).

5.3.6 Penstock

The penstock is the pipe which conveys water under pressure from the forebay tank to the turbine. The penstock often constitutes a major expense in the total micro hydro budget, as much as 40 % is not uncommon in high head installations, and it is therefore worthwhile optimizing the design (klunne-2000). Penstock diameter optimization accomplishes the following objectives: To minimize investment and operating costs, to minimize head losses, to minimize the penstock length for a given gross head, to maximize stability (against pressure surges). These conditions can be fulfilled by selecting the appropriate pipe material, diameter and wall thickness (Harvey-1993).

5.3.6.1 Economic diameter of penstocks

The so-called economic diameter of penstocks should be determine by techno-economic comparative studies, taking in to account the incremental energy benefits from a lower friction loss in a larger penstock, as well as the water hammer speed regulations of the units. The trade-off is between head loss and capital cost. Head loss due to friction in the pipe decrease dramatically with increasing pipe diameter. Conversely, pipe costs increase steeply with diameter. Therefore a compromise between cost and performance is required. The design philosophy is first to identify available pipe options, then to select a target head loss, 5 % of the gross head being a good starting point. The details of the pipes with losses close to this target are then tabulated and compared for cost effectiveness. A smaller penstock may save on capital costs, but the extra head loss may account for lost revenue from generated electricity each year. This method is very complicated and may also result in some uncertainties. For Small hydropower plants, empirical formulas for ascertaining the economic diameter for penstocks are recommended by various authors on the basis of data from existing penstocks. Some of such formulas are given below.

1) $De = C_1 C_2 Q_o^{0.43} H_o^{-0.24}$ Equation 5.3-5

Where De=economic diameter (m)

Qo=design discharge of the penstock or plant (M^3/s)

Ho= design head of plant (m)

C_1=coefficient taking in to consideration the energy cost in the area.

C_1=1.2 for areas where the energy cost is low

C_1=1.3 for areas where the energy cost is medium

C_1=1.4 for areas where the energy cost is high or no alternative source exists.

C_2=coefficient taking in to account the material for the penstock; 1 for steel penstocks, 1.05-1.1 for wood stave pipes, 0.90-0.95 for plastic pipes.

2) $D = E_p P^{0.43} H^{-0.57}$ Equation 5.3-6

Where Ep=0.49 for metric units

D= diameter, m

P= turbine rated capacity, KW

H=turbine rated head, m

3) Gordon and Penman (1979) suggested the following formula for estimating the optimal diameter (mm) from the design discharge Q_d given in m^3/s.

$d_p = 720 Q_d^{0.5}$ Equation 5.3-7

5.3.6.2 Thickness of penstock

The penstock wall thickness grows in proportion to the diameter of the pipe. To ensure a reasonable economic design, the following equation is used with an extra allowance of 1-2mm to account for corrosion.

$$t_k = \frac{d_p . P_i}{2\sigma_{allow}}$$ Equation 5.3-8

Where

t_k-thickness mm,dp-diameter in mm,

σ_{allow}-allowable tensile strength (N/m^2)

P_i-internal pressure (N/m^2)

d_p-diameter in mm

5.3.6.3 Penstock materials

The following factors have to be considered when deciding which material to use for a particular penstock: surface roughness, Design pressure, soil type, design life and maintenance, weather conditions, method of jointing, weight and ease of installation, accessibility of the site, terrain, availability, relative cost, and Likelihood of structural damage (Harvey-1993).

Mild steel, uPVC (unplasticized polyvinyl chloride) and HDPE (high density polyethylene) are the most common used materials.

5.3.6.4 Penstock jointing

Pipes are generally supplied in standard lengths and have to be joined together on site. There are several ways of doing this and the following factors should be considered when choosing the best joint system for a particular scheme: suitability for chosen pipe material, skill level of personnel installing the pipe, relatively costs and Ease of installation.

Methods of pipe jointing fall roughly into four categories: flanged, spigot and socket, mechanical, and welded.

5.3.6.5 Burying or supporting the penstock

Penstock pipelines can either be surface mounted or buried underground. The decision will depend on the pipe material, the nature of the terrain and environmental considerations.

Buried pipelines should be ideally at least 750 mm below ground level, especially when heavy vehicle are likely to cross it. Burying a pipe line removes the biggest eyesore of a hydro scheme and greatly reduces its visual impact. However, it is vital to ensure a buried penstock is properly and meticulously installed because any subsequent problems such as leaks are much harder to detect and rectify.

Where the nature of the ground renders burying the penstock impossible there is sometimes no option but to run the line above ground, in which case piers, anchors and thrust blocks will be needed to counteract the forces which can cause undesired pipeline movement.

The three types of forces that need to be designed against are:

- the weight of the pipes plus water,
- expansion and contraction of the pipe,
- fluid pressure (both static and dynamic).

Support piers are used primarily to carry the weight of the pipes and enclosed water. Anchors are large structures which represent the fixed points along a penstock, restraining all movements by anchoring the penstock to the ground. A thrust block is used to oppose a specific force, for example at a bend or contraction.

The different support structures can usually be built of rubble masonry or plain concrete. Anchor blocks may need steel reinforcement and triangulated steel frames are sometimes used for support piers.

The size and cost of support structures for a given penstock are minimized by: keeping the penstock closer to the ground, avoiding tight joints, avoiding soft and unstable ground.

5.3.7 Power house

The power house should enclose and protect the turbo-generating and associated equipment. Its design and equipment depends on the specific case, for example if block and tackle or cranes traveling on overhead rails are required. For reasons of economy, these equipments are not included in the power house cost for the small hydropower sites under study.

The cost of super-structure is based on an estimate of the required floor area from the design chart given in Fig 4.3-1(J.J.Fritz 1984.)

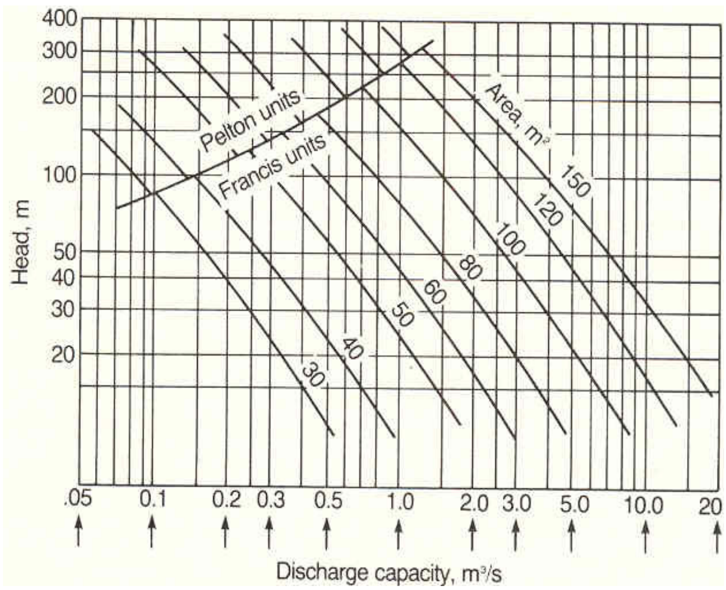


Figure 5.3-1Power house floor area required

The sub-structure should be calculated separately since the volume of concrete in a power house sub-structure comprises a major percentage of the power house cost. Gulliver suggests the use of the formulae which are based on the turbine throat diameter. Accordingly, for reaction units, the volume of concrete within a power house substructure on a competent rock foundation should be about:

$$V = E_h K(N + 0.5)d^{2.4} \qquad \text{Equation 5.3-9}$$

Where Eh=1.0 for metric units

V=volume, m^3 N=number of units, d=turbine throat diameter (m)

K=140 for vertical-axis Francis, propeller, and Kaplan units

=130 for horizontal- axis tube or bulb units

For high-head horizontal-and vertical shaft impulse units, the concrete volume should be in the region of:

$V = 50E[hMW/n]^{0.83}(N+0.5)$ Equation 5.3-10

Where E=1.0 with h in m and V in m^3 V=concrete volume, m^3 h=turbine rated head(m)

MW =generator rating, n=unit synchronous speed, r/min, N=number of units

Where the powerhouse is built on a soft rock or other foundation, concrete substructure volume could increase to about twice the volume estimated with the above formulae. The formulae above can be made use of after the appropriate type of turbine for the site configuration is selected. The appropriate type of turbine is determined on the basis of specific speed calculated from site data and for the standard 50Hz electrical frequency used in Ethiopia.

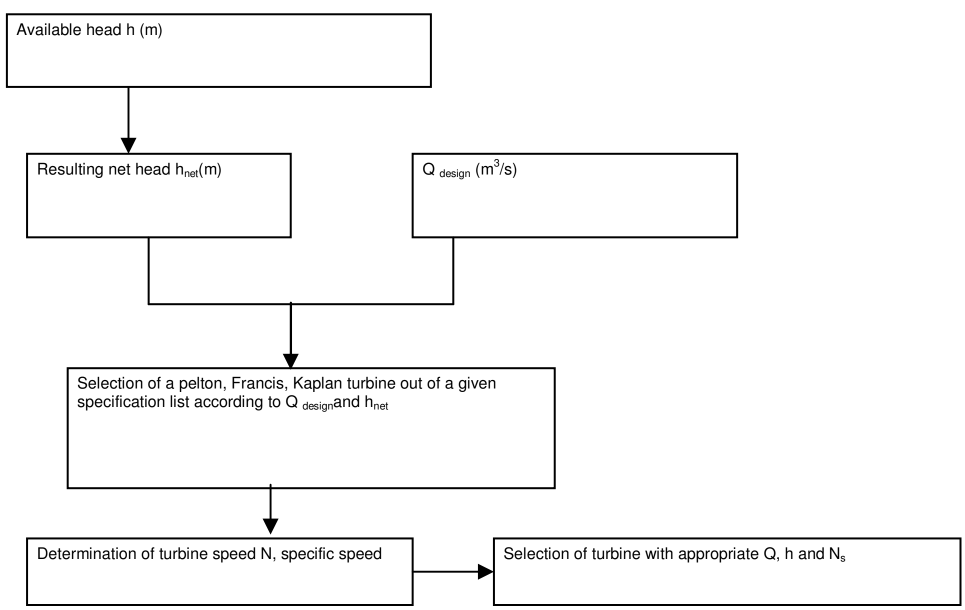


Figure 5.3-2. Flow chart for turbine selection.

To find out, which type of turbine is appropriate under the given site conditions, meaning available head and runoff, the specific speed Ns is determined according to the formula:

$$N_s = \frac{\left(1.155 * N * P^{0.5}\right)}{H_{net}^{\ 1.25}} \qquad \text{Equation 5.3-11}$$

Where Ns=specific speed (RPM), N=turbine speed [rpm], P=turbine capacity [KW], Hnet=net head[m]

5.3.8 Electromechanical equipment cost

Developing the total system cost requires a proper estimation of electro-mechanical costs which accounts for the bulk of investment costs. Knowledge of current prices of turbines for a wide range of capacities can enable the development of cost estimation function for turbo-generating units. Manufacturers however, do not readily supply price list of a wide range of turbine capacities. The estimation of turbo-generating costs is, therefore, based on empirical guide lines available in hydropower literature and previous studies.

5.3.8.1 Empirical Formulae for Equipment Cost Estimate

Several formulae have been published from which an approximate cost of mechanical and electrical equipment can be derived. GATE 1980 recommended the following approximate formula for machinery.

$$K_{DM} = K\left(\frac{P}{H}\right)^{0.53} \qquad \text{Equation 5.3-12}$$

Where, k=48000, K_{DM}-Cost of machinery (Deutschmark), P-installed capacity (KW), H-available head (m)

Swedish experience indicates that for units below 1.5MW capacity, the cost of the equipment can be approximated by the following formula, Gordon and Penman 1979.

$$K_{US} = K\left(\frac{P}{H^{0.3}}\right) \qquad \text{Equation 5.3-13}$$

Where, k=40000, Kus-cost of machinery in USD, P-installed capacity in (KW), H-available head in (m)

Another formula based on North American experience, which is applicable for units below 5MW capacity is given in equation below.

$$K_{US} = K\left(\frac{P^{0.7}}{H^{0.35}}\right) \qquad \text{Equation 5.3-14}$$

Where, k=9000, Kus, P, and H are similar to the equation 4.3-13

The equation for total equipment cost which was developed by Gulliver and Dotan 1984 has been updated by Gulliver and Roger 1991 based on producers price index for machinery and equipment, and is given below:

$$K_{us} = k\left(\frac{P^{0.82}}{H^{0.35}}\right) \qquad \text{Equation 5.3-15}$$

Where, k=16100,K_{US}-cost of equipment in USD as of 1987 price (J.J.Fritz).

P-total plant capacity (KW)

H-head (m)

The equation 4.3-15 results in satisfactory equipment cost estimates for plant capacity range from 50 to 40,000KW and a hydraulic head range from 4-100m (Gulliver and Gordon-1991).

The equipment costs of the power station (turbines, generator, control, transformers) are estimated using equation 4.3-16 derived on the basis of international prices (table 4.1) (Abbay River Basin integrated development Master plan-sep, 1998).

$C=KP^{-0.29}$ Equation 5.3-16

Where, k=1470, C-cost per installed KW in USD, and P is installed capacity in MW.Contingences of 15% have been added to estimate the final equipment cost. One USD is taken to be equal to 8.5 Ethiopian birr.

Table 5-1 showing power plant equipment cost for different countries in The World

Country	P(MW)	Head (m)	C(US$/KW)	comments
Pakistan	30.40	255	772	Under construction
Pakistan	15.00	5	979	Turn key contract awarded
USA	2.60	7	957	In operation
Equatorial guinea	3.80	150	2053	In operation
Tlokoeng	0.72	200	2944	In operation
Portugal	0.80	166	938	In operation
Sao Tome	1.85	260	2040	Competitive bidding
Peru	80.00	500	522	Competitive bidding
Pakistan	270.00	7	496	Under construction
Indonesia	19.00	120	457	Under construction
Indonesia	360.00	180	273	Competitive bidding
Pakistan	245	53	255	Competitive bidding
Gabon	0.30	6.5	2418	In operation
France	0.50	4	1055	In operation

For this thesis the above equation (4.3-16) is used for estimating equipment cost as it is the recent one, developed after examining the data base costs.

5.3.9 Transmission

If the power house is remote from the load as it is often, transmission line is required. A transformer at the start of the transmission line near the power house is used to step up the voltage to a higher value to minimize transmission losses and large voltage drops. Step-down transformers at the end of the transmission line provide a lower voltage to the consumers. By comparing the investment and maintenance costs for transformers and high voltage lines, including accessories, such as expensive insulators, with the costs of the power losses and the larger cable cross-sections for low voltage (LV) lines, it is found that in general, LV lines are economical only for distances of less than about 1.5KM (Harvey-1993). For distances of more than 1.5km, to avoid large voltage drops on long LV lines, their cross section has to be increased significantly. Given the fact that the cost increases as the square of the transmission distance, smaller conductor cross-sections, either accommodated by using transformers or acceptance of higher losses, should be chosen to avoid exaggerated costs. As the small hydropower stations may often be located at a distance greater than 1.5km, the possibility of purely LV option becomes an exception in rural Ethiopia. In most cases, a medium voltage system (15KV) is appropriate solution. Up to now the 15KV level was the one mainly used by EEPCO for rural electrification starting from national grid (ICS). The limited reach of 15KV system led the requirement of a large number of low rated substations, yielding high fixed cost per unit of energy supplied. Therefore, EEPCo decided to switch to a 33KV transmission system for new grid extensions. Standard conductor sizes are specified as 25 and 35mm^2 'copper equivalent', which are equivalent in aluminum content to 35 and 50mm^2 all-aluminum conductor, respectively.

5.3.9.1 Transmission Losses

The optimization of the hydropower system takes the cost of transmission lines and the cost of associated power losses in to account. This is so because the cost of transmission is an important component of the cost of rural electrification. The optimization seeks a balance between investment costs and costs of losses.

Transmission losses are dependent on the characteristics of the transmission material, on the length and cross-section of the transmission line, on the transmission voltage level and on the magnitude of transmitted power. In general, the power losses are proportional to the electrical resistance RI(in ohm) through the transmission line. Line losses through a transmission line at 20^{0}C can be estimated from basic resistance formula given in equation (4.3-17)

$$RI_{20} = \frac{L}{K_{20} a_q} \quad (\Omega) \qquad \text{Equation 5.3-17}$$

Where: L-length of transmission line (m)

a_q- cross-sectional area of transmission line (mm^2)

K_{20}-coefficient of conductance of the transmission material at 20^0C(m/ Ωmm^2)

For copper and aluminum which are the commonly used materials values of coefficient of conductance of 56m/ Ωmm^{2}and 35.4 Ωmm^2 are used respectively. As a material constant, the specific resistance is to large or small extent dependent on the temperature. Temperature coefficients are used to calculate the specific resistance at any other temperature.

5.3.9.2 Power losses in a transmission line carrying alternating current

The power losses over a km length of a three phase transmission line is calculated using equation (4.3-18).

$$PI = \left(\frac{0.003.RE}{V^2}\right)PT^2 \qquad \text{(kw/kw)} \qquad \text{Equation 5.3-18}$$

Where, Pl-losses in KW/km

RE-Resistance in Ω/km

V-transmission voltage (v)

PT-transmission level (W)

The above expression is useful in that the transmission losses can be obtained in terms of the decision variables of the objective function to be developed. For a 33KV copper transmission with a cross-section of 50mm^2, equation (4.3-19) can be used to calculate the power loss as a function of the transmitted power (Zelalem-2002).

$$PI = 10^{-6}.PT^2 = \varepsilon_R PT^2 \qquad \text{Equation 5.3-19}$$

Though for MHP systems, a 15KV line with 3 wires is the adequate option up transmission distances of about 20km (Hedi Feibel-2003), the 33KV transmission line has been adopted for further analysis in this study as per the decision made by EEPCo to switch to 33 KV transmission line and as most of the demand centers may be located at a distance greater than 20KMs. In addition, the use of a higher voltage transmission makes the voltage drops relatively smaller than the over all capacity and thus keeps the percentage of drop within tolerable limit. These are the reasons for the preference of the 33KV transmission line to the 15KV line (Zelalem-2002).

5.3.10 Cost Functions For selected sites

The procedures described in the previous sections are applied to determine representative cost functions for the identified sites. The sites had been identified by the study team from P.R. china as mentioned before. This study can be referred with regard to the available construction materials and accessibility of some of the sites.

For developing cost functions, different development alternatives should be considered. For instance, for a site with three possible location of intake, depending on the location of power house, there may be more than one alternative corresponding to one intake. This may result in different layouts. For this thesis, considering different location of intake and power house location is impossible as the sites are already identified by previous studies and as it requires a site visit which in turn requires sufficient time and rigorous surveying work as well as resource which will be beyond the limit of the thesis. Therefore, for comparing different alternatives and to establish cost functions for the sites, different flow values are considered from the flow-duration curve with maximum value of flow corresponding to 90 % dependable discharge. Discharges less than or equal to 90% has been preferred to ensure safer supply (lower installed capacity-higher reliability).

The cost estimate for each possible alternative has to be determined. For this purpose, it is necessary to arrive at a preliminary cost estimate for all sites.

Cost function for Andassa SHP as an example

For illustration, the site at Andassa River is taken to establish a cost function based on the procedure described above. The site requires the following components of small hydropower plant: settling basin, diversion canal, power house, masonry diversion structure, penstock and turbine .Therefore, the design of these components is necessary at different discharge level to develop the function. Accordingly for different values of discharge the corresponding power generation in KW has been determined. For these discharge values the costs of each component found. Finally the cost function is developed as in the fig4.3-3.A similar procedure has been applied to develop the cost function for the other sites and the graph of the function is given in the appendices.

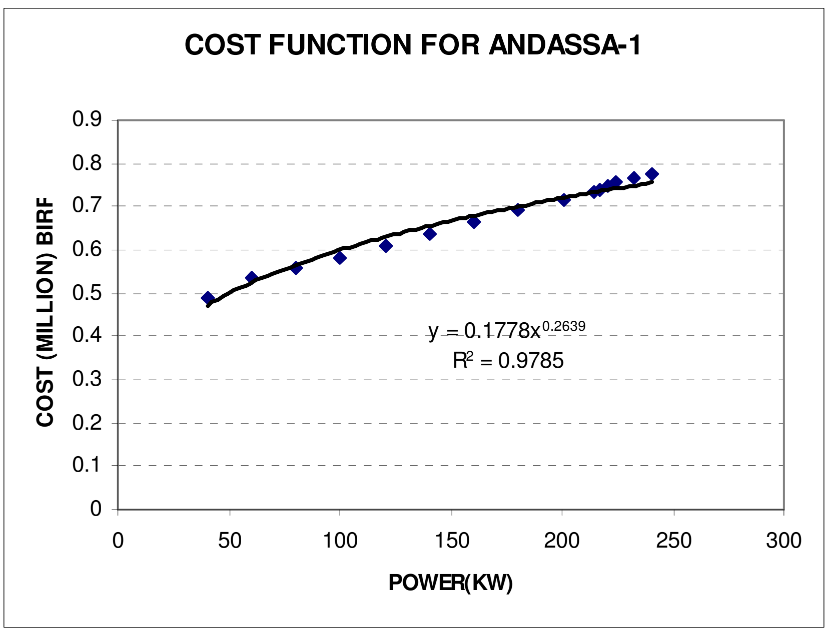


Figure 5.3-3 Cost function for Andassa-1

6 Optimization procedures applicable to the small hydropower system

Optimization is the procedure of selecting a set of decision variables which maximizes or minimizes the objective function subject to the system's constraints. Mathematically, this means finding the extremes of a function of n number of variables, $f(x_1, x_2, x_3 \ldots x_n)$, where n may be any integer greater than zero. The function may be unconstrained or it may be subject to certain constraints on the variables of the function g, say $gi(x_1 \ldots x_n) = bi$ for $i=1 \ldots m$.

Optimization procedure must be based on as complete a description as possible of all constraints, rules and objectives, and provisions for resolving conflicts among them.

Depending upon the nature of the objective function and the constraints, the optimization problem can be linear or nonlinear. As described in section 4.1, hydropower cost functions are nonlinear and hence the nonlinear optimization technique will be described in the following sections.

6.1 Necessity of optimization for small hydropower

1. Optimization is necessary to inform decision makers and/or investors interested in small hydropower development to make proper decision in allocating very limited financial resources among competing development alternatives.
2. It is necessary for prioritizing development alternatives in a country like Ethiopia where there is a large potential of hydropower and where there is also a large unsatisfied demand.

6.2 Nonlinear programming (NLP)

Nonlinear programming (NLP) deals with optimization models with at least one nonlinear function. In nonlinear programming, the concepts of convexity and concavity are used to establish whether a local optimum, (local minimum, or local maximum) is also the global optimum, which is the best among all solutions. In the univariate case, a function f(x) is said to be convex over a region if for every x_a and x_b, $x_a \neq x_b$, the following holds

$$f[\theta x_a+(1-\theta) x_b] \leq \theta f(x_a)+(1-\theta)f(x_b),\ 0\leq \theta\leq 1 \qquad \text{Equation 6.2-1}$$

The function is strictly convex when the above relation holds with a less than (<) sign.

Conversely, a function is concave over a region if for every $\mathbf{x_a}$ and $\mathbf{x_b}$, $\mathbf{x_a} \neq \mathbf{x_b}$ the following holds

$$f[\theta\, x_a+(1-\theta)\, x_b] \geq \theta f(x_a)+(1-\theta)f(x_b),\ 0\leq \theta\leq 1 \qquad \text{Equation 6.2-2}$$

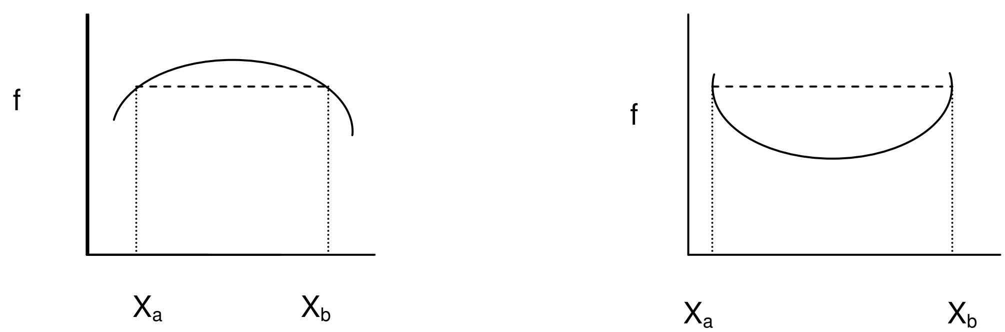
The function is strictly concave when the above relation holds with a greater than (>) sign.

In addition to Equations 5.2.1and 5.2.2, the sign of the second derivative, $d^2f(x)/dx^2$can be used in testing the convexity or concavity of a function. From fundamental calculus, if

$\frac{d^2 f}{dx^2}$ <0, then the function is concave and if

$\frac{d^2 f}{dx^2}$ >0, then the function is convex

Fig 5.2-1a and 5.2-1b show concave and convex functions



a) Concave function **b) Convex function**

Figure 6.2-1 a) concave function b) convex function

Convex regions or sets are used to classify constraints. A convex regions exists if for any two points in the region, $X_a \neq X_b$, all points $X=\theta Xa+(1-\theta)X_b$, where $0\leq\theta\leq 1$, are on the line connecting X_a and X_b are in the set.

The convexity of feasible region and the objective function in nonlinear optimization has an extremely important implication with regard to the type of optimal solution to be obtained. Unlike linear programming, the convexity of both the objective function and feasible region in a nonlinear programming problem can not be ensured. The optimal solution achieved, therefore, can not be guaranteed to be global. For this thesis, the nature of the objective function is checked in section 5.10.5.

6.3 Unconstrained nonlinear optimization

Understanding unconstrained optimization procedures is important because these techniques are the fundamental building blocks in many of the constrained nonlinear optimization algorithms.

6.3.1 Basic concepts

The problem of unconstrained minimization can be stated as

Minimize f(x) Equation 6.3-1

$X \, \varepsilon \, E^n$

in which X is a vector of n decision variables $X=(x_1, x_2 \ldots x_n)^T$ defined over the entire Euclidean space E^n. Since the feasible region is infinitely extended without bound, the optimization problem does not contain any constraints.

If f(x) is nonlinear, the necessary conditions for a solution to equation 5.3-1, at x* are:

(1) $\nabla f(x^*)=0$; &

(2) $\nabla^2 f(x^*)=H(x^*)$ is semi -positive definite.

The sufficient conditions for unconstrained minimum are:

(1) $\nabla f(x^*) = 0$; and (2) $\nabla^2 f(x^*) = H(x^*)$ is strictly positive definite.

In theory, the solution to the above equation can be obtained by solving the following system of n non linear equations with n unknowns.

$\nabla f(x^*) = 0$ Equation 6.3-2

An iterative numerical procedure is required to solve these systems of non linear equations (5.3-2) which tend to be computationally inefficient.

By contrast, the preference is given to those solution procedures which directly attack the problem of minimizing f(x). These procedures, during the course of iteration, generate a sequence of solution points in E^n that terminate or converge to a solution of equation5.3-1. Such methods can be characterized as search procedures.

In general, all search algorithms for unconstrained minimization consist of two basic steps.

- The first step is to determine the search direction along which the objective function value decreases.
- The second step is called a line search (or one dimensional search) to obtain the optimum solution point along the search direction. Mathematically, minimization for the line search can be stated as

$$\min_{\beta} f(x^0+\beta d) \qquad \text{Equation 6.3-3}$$

in which x^0 is the current solution point, d is a vector indicating the search direction, and β is a scalar, $-\infty<\beta<\infty$, representing the step size whose optimal value is to be determined. Due to the very nature of search algorithms, it is likely that different starting solutions might converge to different local minimum. Hence, there is no guarantee of finding the global minimum by any search technique applied to solve the objective function, unless the objective function is a convex function over E^n.

In implementing search techniques, specification of convergence criteria or stopping rules is an important element that affects the performance of the algorithm and the accuracy of the solution. Several commonly used stopping rules in an optimum seeking algorithm are:

a) $\left\|x^k - x^{k+1}\right\| < \varepsilon_1;$ Equation 6.3-4a

b) $\dfrac{\left\|x^k - x^{k+1}\right\|}{\left\|x^k\right\|} < \varepsilon_2$ Equation5.3-4b

c) $\left|f(x^k) - f(x^{k+1})\right| < \varepsilon_3;$ Equation 5.3-4c

d) $\left|\dfrac{f(x^k) - f(x^{k+1})}{f(x^k)}\right| < \varepsilon_4$ Equation5.3-4d

In which superscript k is the index for iteration, ε represents the tolerance or accuracy requirement, $\|x\|$ is the length of the vector x, and $|x|$ is the absolute value. The specification for the tolerance depends on the nature of the problem and on the accuracy requirement. Too small value of ε (corresponding to high accuracy requirement) could result in excessive iterations. On the other hand, too large value of ε could make the algorithm terminate prematurely at a non optimal solution.

6.3.2 One dimensional search

The line search techniques for solving one-dimensional optimization problems form the back bone of non linear programming algorithm. Multidimensional problems are ultimately solved by executing a sequence of successive line searches. One-dimensional search techniques can be classified as curve-fitting (approximation technique) or as interval elimination techniques. Interval elimination techniques for a one dimensional search essentially eliminate or delete a calculated portion of the range of the variable from consideration in each successive iteration of the search for the optimum of f(x). After a number of iterations when the remaining interval is sufficiently small the search procedure terminates. These methods determine the minimum value of a function over a closed interval [a, b] assuming that a function is unimodal, that is, it has only one minimum value in the interval. Two interval elimination techniques commonly used are the golden section method and the Fibonacci search method. Because these two methods are similar, only the golden section is described.

Golden Section Method

The golden section method is based upon splitting the line segment into two segments in which the ratio of the whole line to the larger segment (ΔL) is the same as the ratio of the larger segment (ΔL) to the smaller segment (ΔS).

The objective of the golden search algorithm is to apply the fractions FL= ΔL and FS = ΔS for any particular interval to compute the proper distances.

The golden section algorithm for minimizing a function, f(x), can be stated as follows.

Step 0 k=0. Select values of a^0 and b^0 that bracket the minimum of f(x).

Step1. Determine the interior points $x_1^k=a^k+0.382(b^k-a^k)$ and

$$x_2^k=b^k-0.382(b^k-a^k) = a^k+0.618(b^k-a^k).$$

Step 2. Determine $f(x_1^k)$ and $f(x_2^k)$

Step 3.

If $f(x_1^k) <f(x_2^k)$: $a^{k+1}=a^k$ and $b^{k+1}= x_2^k$

If $f(x_1^k)>f(x_2^k)$: $a^{k+1}= x_1^k$ and $b^{k+1}= b^k$

If $f(x_1^k)=f(x_2^k)$: $a^{k+1}= a^k$ and $b^{k+1}= x_2^k$ or $a^{k+1}= x_1^k$ and $b^{k+1}= b^k$.

Step 4. If the convergence criteria are not satisfied (k=k+1) and return to step 1. After k iterations, the sub interval has length $(b^k-a^k)= 0.618^k(b^0-a^0)$.

6.3.3 Multi variable methods

Unconstrained optimization problems can be stated in a general form as

$$\text{Minimize } z=f(x) =f(x_1,x_2,\ldots,x_n) \qquad \text{Equation 6.3-5}$$

For maximization, the problem is to minimize $-f(x)$. The solution of these types of problems can be stated in an algorithm involving the following basic steps or phases:

Step 0. Select an initial starting point $x^{k=0}=(x_1^0,x_2^0\ldots x_n^0)$

Step 1. Determine a search direction, d^k

Step 2. Find a new point $x^{k+1}=x^k+\beta^k d^k$ where β^k is the step size, a scalar, which minimizes $f(x^k+\beta^k d^k)$

Step 3. Check the convergence criteria equation (5.3.4a-5.3.4d) for termination. If not satisfied, set k=k+1 and return to step 1.

The various unconstrained multivariate methods differ in the way the search directions are determined. The recursive line search for an unconstrained minimization problem is expressed in step 2 as

$$x^{k+1}=x^k+\beta^k d^k \qquad \text{Equation 6.3-6}$$

There are four basic groups of methods for determining the search directions: steepest descent methods, Conjugate direction methods, Quasi Newton methods and Newton's method. The simplest are the steepest descent methods while the Newton methods are the most computationally intensive.

In the steepest descent method the search direction is $-\nabla f(x)$. Since $\nabla f(x)$ points to the direction of the maximum rate of increase in the objective-function value, therefore, the negative sign is associated with the gradient vector in equation (5.3-6) because the problem is minimization type. The recursive line search equation for the steepest descent method is reduced to

$$x^{k+1} = x^k - \beta^k \nabla f(x^k) \qquad \text{Equation 6.3-7}$$

Using Newton's method, the recursive equation for the line search is

$$x^{k+1} = x^k - H^{-1}(x^k)\ \nabla f(x^k) \qquad \text{Equation 6.3-8}$$

Although Newton's method converges faster than other algorithms, the major disadvantage is that it requires inverting the Hessian matrix in each iteration which is a computationally cumbersome task.

The conjugate direction methods and Quasi Newton methods are intermediate between the steepest descent and Newton's method. The conjugate direction methods are applied by the need to accelerate the typically slow convergence of the steepest descent method. Conjugate direction methods define the search direction by utilizing the gradient vector of the objective function of the current iteration and the information on the gradient and search direction of the previous iteration. The motivation of quasi-Newton methods is to avoid inverting the Hessian matrix as required by Newton's method. These methods use approximation to the inverse Hessian with a different form of approximation for the different quasi-Newton methods.

6.4 Constrained optimization: optimality conditions

6.4.1 Lagrange multiplier

Consider the general non-linear programming problem with the non linear objective:

$$\text{Minimize } f(x) \qquad \text{Equation 6.4-1a}$$

Subject to

$g_i(x) = 0 \quad i=1\ldots m$ Equation 5.4-1b

and

$X_{\ell j} \leq X_j \leq X_{uj} \quad j=1, 2\ldots n$ Equation 5.4-1c

In which equation (5.4.1c) is bound constraint for the j^{th} decision variable X_j and $x_{\ell j}$ and X_{uj} being the lower and upper bounds, respectively.

In constrained optimization problem, the feasible space is not infinitely extended, unlike an unconstrained problem. As a result, the solution that satisfies the optimality condition of the unconstrained optimization problem does not guarantee to be feasible in constrained problem. In other words, a local optimum for a constrained problem might be located on the boundary or a corner of the feasible space at which the gradient vector is not equal to zero. Therefore, modifications to the optimality conditions for unconstrained problems must be made.

The most important theoretical results for nonlinear constrained optimization are the Kuhn-Tucker conditions. These conditions must be satisfied at any constrained optimum, local or global, of any linear and nonlinear programming problem. They form the basis for the development of many computational algorithms.

Without losing generality, consider a nonlinear constrained problem stated by equation (5.4-1a) with no bounding constraints. Note that constraint equations (5.4-1b) are all equality constraints. Under this condition, the Lagrange multiplier method converts a constrained nonlinear programming problem into an unconstrained one by developing an augmented objective function, called the Lagrangian. For a minimization, the Lagrangian function $L(x, \lambda)$ is defined as

$$L(x, \lambda) = f(x) + \lambda^T g(x)$$ Equation 6.4-2

In which λ is the vector of Lagrange multipliers and $g(x)$ is a vector of constraint equations. Algebraically, equation 5.4-2 can be written

$L(x_1,\ldots,x_n, \lambda_1,\ldots\ldots\ \lambda_m)=f(x_1,\ldots,x_n)+ \sum_{i=1}^{m} \lambda_i g_i\,(x_1,\ldots,x_2)$ Equation 6.4-3

$L(x, \lambda)$ is the objective function, with m+n variables, that is to be minimized. The necessary and sufficient conditions for X* to be the solutions for minimization are:

1. f(x*) is convex and g(x*) is convex in the vicinity of x*;

2. $\frac{\partial L(x^*)}{\partial x_j} = \frac{\partial f}{\partial x_j} + \sum_{i=1}^{m} \lambda_i \frac{\partial g_i}{\partial x_j} = 0$ j=1,…,n, Equation 6.4-4a

3. $\frac{\partial L}{\partial \lambda_i} = g_i(x) = 0$ i=1,2,…m Equation 5.4-4b

4. λ_i is unrestricted-in-sign i=1,…, m Equation 5.4-4c

Solving equations (5.4-4a) and (5.4-4b) simultaneously provides the optimal solution.

Lagrange multipliers have an important interpretation in optimization. For a given constraint, these multipliers indicate how much the optimal objective function value will change for a differential change in the RHS of the constraint. That is,

$\frac{\partial f}{\partial bi}_{x=x} = \lambda i$ Where bi is the RHS of the constraint

illustrating that the Lagrange multiplier λ_i is the rate of change of the optimal value of the original objective function with respect to the change in the value of the RHS of the i^{th} constraint. The λ_i's are called dual variables or shadow prices.

6.4.2 Kuhn-Tucker Conditions

Equations (5.4-4a)-(5.4-4c) form the optimality conditions for an optimization problem involving only equality constraints. The Lagrange multipliers associated with the equality constraint are unrestricted-in-sign. Using the Lagrange multiplier method, the optimality conditions for the following generalized nonlinear programming problem can be derived.

Minimize f(x)

Subject to

$g_i(x) = 0 \qquad i=1\ldots.m$

And $X_{\ell j} \le X_j \le X_{uj} \qquad j=1\ldots.n$

In terms of Lagrangian method, the above nonlinear minimization problem can be written as

$$\text{Min } L=f(x)+\lambda^T g(x)+ \lambda^T(x\ell\text{-}x)+ \lambda^T(x\text{-}xu) \qquad \text{Equation 6.4-5}$$

In which λ_s are vectors of Lagrange multipliers corresponding to constraints $g(x) = 0$, $X_\ell\text{-}X \le 0$ and $X\text{-}X_u \le 0$ respectively.

The Kuhn-Tucker conditions for the optimality of the above problem are

$$\nabla_x L = \nabla_x f + \lambda^T \nabla_x g - \lambda_\ell + \lambda u = 0 \qquad \text{Equation 6.4-6a}$$

$$g_i(x)=0, \qquad i=1,2,\ldots m \qquad \text{Equation 5.4-6b}$$

$$\lambda_{\ell j}(X_{\ell j}-X_j)=\lambda_{uj}\,(x_j-x_{uj})=0, \quad j=1,2,\text{---},n \qquad \text{Equation 5.4-6c}$$

$$\lambda \text{ unrestricted-in-sign}, \ \lambda_\ell \ge 0, \ \lambda_u \ge 0 \qquad \text{Equation 5.4-6d}$$

6.5 Constrained nonlinear optimization: generalized reduced gradient (GRG) method

6.5.1 Basic concepts

Similar to the linear programming simplex method, the fundamental idea of the generalized reduced gradient method is to express m (number of constraint equations) of the variables, called basic variables, in terms of the remaining n-m variables, called nonbasic variables. The decision variables can then be partitioned into the basic variables, X_B, and the nonbasic variables, X_N,

$$X= (X_B, X_N)^T \qquad \text{Equation 6.5-1}$$

Nonbasic variables not at their bounds are called super basic variables (Mays and Tung-1992).

The optimization problem can now be restated in terms of the basic and non basic variables

Minimize f (X_B, XN) Equation 6.5-2a

Subject to

g (X_B,X_N)=0 . Equation 5.5-2b

and

$X_{\ell B} \leq X_B \leq X_{UB}$ Equation 5.5-2c

$X_{\ell N} \leq X_N \leq X_{UN}$ Equation 5.5-2d

The m basic variables in theory can be expressed in terms of the n-m non-basic variables as $X_B(X_N)$. Assume that constraints g(x) =0 is differentiable and the m by m basis matrix B can be obtained as

$$\mathbf{B}= \left[\frac{\partial g(x)}{\partial x_B}\right]$$

This is non singular such that there exists a unique solution of X_B (X_N). Non singularity means that det (B) $\neq$0.

The objective called a reduced objective can be expressed in terms of the non basic variables as

$\mathbf{F\ (X_N) = f\ (X_B\ (X_N),\ X_N)}$ Equation 6.5-3

The original nonlinear programming problem is transformed into the following reduced problem

Minimize F (X_N) Equation 6.5-4a

Subject to

$\mathbf{X}_{\ell N} \leq \mathbf{X}_N \leq \mathbf{X}_{UN}$ Equation 5.5-4b

Which can be solved by an unconstrained minimization technique with slight modification to account for the bounds on the non-basic variables. Generalized reduced gradient algorithms, therefore, solve the original problems (5.4-1) by solving a sequence of reduced problems (5.5.4a, b) using unconstrained minimization algorithms.

6.5.2 General algorithm and Basis changes

Consider solving the reduced problem (5.5-4a) starting from an initial feasible point X^0. To evaluate $F(X_N)$by equation(5.5.3), the values of the basic variables X_B must be known. Except for a very few cases, $X_B(X_N)$cannot be determined in closed form; however, it can be computed for any X_N by an iterative method which solves a system of m nonlinear equations with the same number of unknowns as equations. A procedure for solving the reduced problem starting from the initial feasible solution $X^{k=0}$ is:

Step 0. Start with an initial feasible solution $X^{k=0}$ and set $X_N^k= X^{k=0}$

Step 1. Substitute X_N^k into eq (5.5.2b) and determine the corresponding values of X_B by an iterative method for solving m nonlinear equations $g(X_B(X_N^k), X_N^k)=0$.

Step 2. Determine the search direction d^k for the non basic variables.

Step 3. Choose a step size for the line search scheme, β^k such that

$$X_N^{k+1}= X_N^k+ \beta^k d^k \qquad \text{Equation 6.5-5}$$

This is done by solving the one dimensional search problem

$$\text{Minimize } F(X_N^k+ \beta d^k)$$

With x restricted so that $X_N^k+ \beta d^k$ satisfies the bounds on X_N. This one dimensional search requires repeated applications of step1 to evaluate f for the different β values.

Step 4. Test the current point $X^k = (X_B^k, X_N^k)$ for optimality, if not optimal, set k=k+1 and return to step 1.

6.5.3 The reduced gradient

Computation of the reduced gradient is required in the generalized reduced gradient (GRG) method in order to define the search direction.

6.5.4 Non linear programming codes

The non linear programming codes that have been applied to solve NLP problems are:1)GRG2(Generalized reduced gradient2)developed by Lasdon and his colleagues(Mays and Tung-1992)2)GINO(Mays and Tung-1992) 3)MINOS(Modular in core nonlinear optimization system)Developed by Murtagh and Saunders(Mays and Tung-1992)and

4)GAMS-MINOS. Among the nonlinear programming codes mentioned above, the GRG2 code which is used in the Microsoft Excel 2000 solver, as developed by Lasdon and Waren is used to solve the optimization problems in the study at hand. After experimentation with gradient search (penalty function method, method of feasible directions& generalized reduced gradient) and Genetic algorithm (genetic algorithm solver,GENOCOPIII) methods, a reduced gradient type method has been found to be appropriate method for optimization of small hydropower systems (Zelalem-2002).GRG2 computer code utilizes the fundamental ideas of the generalized reduced gradient algorithm described in section 5.5. This code requires that the user to provide an objective function and constraints of the nonlinear programming problem. Accordingly the objective function and the constraints developed in the previous sections will be provided and the solution obtained.

6.6 Optimization of the selected sites

6.6.1 Preliminary work for optimization

In the previous sections, different aspects of small hydropower have been dealt with the view to develop cost relationships for the objective function and the constraints.

6.6.2 Concept of optimization

Before proceeding to the optimization of the specific system, the basic idea is discussed below.

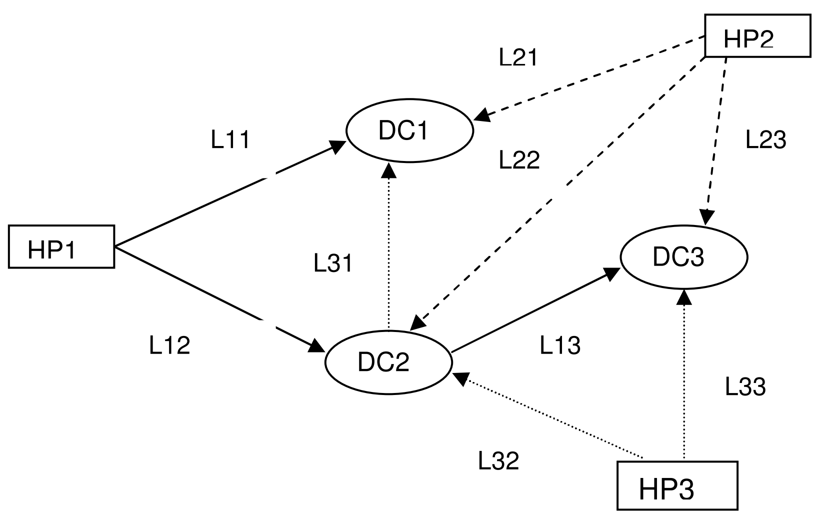


Figure 6.6-1 Planned optimization in Abbay basin**.**

The above figure 5.6-1 is used to illustrate the planned optimization in the Abbay Basin.

The arrows indicate optional transmission lines. The description of variables for a general case of many power plants and demand centers is as below.

DC_1, DC_2… DC_n---demand centers

HP_1, HP_2,…, HP_m----identified power plants

n-number of demand centers

m-number of power plants

L_{ji}-distance(km) of demand center i from power plant j or from an intermediate load center served by the power plant

P_{ji}-power(KW) transmitted to load center i from power plant j

D_i-power demand at load center i (KW)

PG_j-power generated at site HP_j

$Pmax_{,j}$-maximum power potential at site j

Pl_{ji}-power loss(KW) associated with the transmission of P_{ji}

The general form of transmission losses through a particular transmission line can be described in terms of its length and the power transmitted through it. That is, transmission loss is the function of power and distance as described in section 4.3.9 i.e., $Pl_{ji}=f(L_{ji},P_{ji})$.

In cases of power plants serving more than one demand center, the power transmitted through each line has to be modified according to the actual load transmitted through it. This is because of the fact that the difference has an effect on the power transmission losses. This case is illustrated in the Figure5.6-2 below.

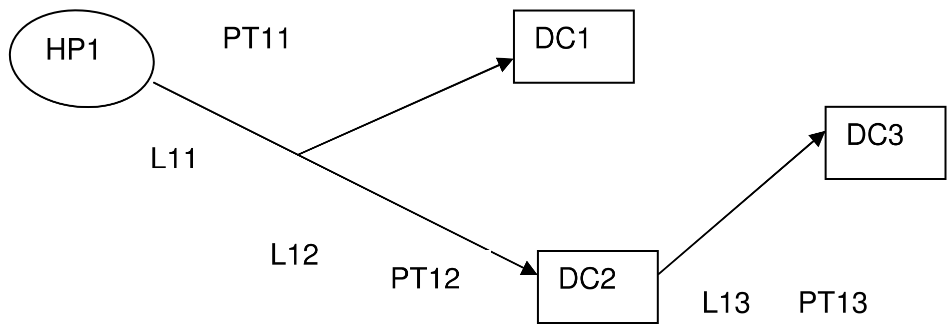


Figure 6.6-2Power transmission to load centers

If PT11, PT12, and PT13 denote the power transmitted through the lines L11,L12 and L13 respectively, then:

$$PT11 = P11 + P12 + P13$$
$$PT12 = P12 + P13 \qquad \text{Equation 6.6-1}$$
$$PT13 = P13$$

A new variable PTji is introduced which takes the difference in power transmission through each line into consideration. Accordingly, power losses through the different lines are calculated based on the relationship

$PI_{ji} = \varepsilon_R PT_{ji}^{\ 2}.Lji$ where $\varepsilon_{R=}$factor of power loss through electric resistance=0.000001.

6.6.3 Cost of power generation at the difference sites

The total power generated at any plant j is given by the sum of power generated at plant j and transmitted to demand center i and the power loss from j to i.

Mathematically this is expressed as:

$$PG_j = \sum_{i=1}^{n} (P_{ji} + Pl_{ji}) \qquad \text{j=1,2,…m} \qquad \text{Equation 6.6-2}$$

The optimization requires the cost function developed in the previous sections. The total present cost of power production in the system can be calculated by taking summation over the involved power plants as given below.

$$\sum_{j=1}^{m} (\omega_j PG_j^{\varsigma_j}) \qquad \text{Equation 6.6-3}$$

Where ω_j and ς_j are constants for the cost function obtained by fitting the nonlinear curve to the cost function

6.6.4 Cost of transmission

The cost of transmission line in the system is found based on the layout of power transmission lines. Therefore, it is necessary to prepare an initial layout of power transmission at the beginning of optimization. At the beginning each load centre is assumed to be served by each power plant as shown in the Figure5.6-3.

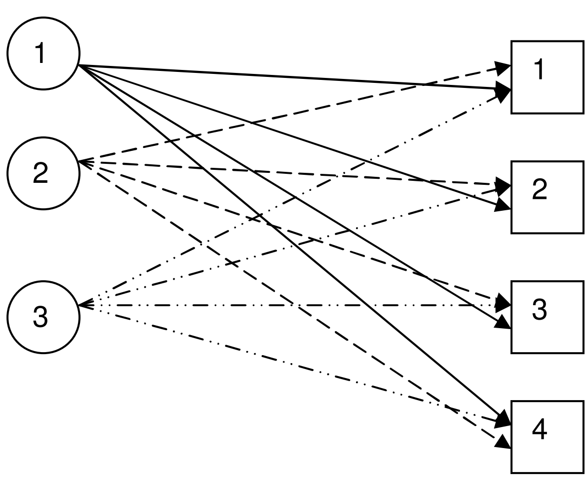


Figure 6.6-3 Hydropower sites and demand centers

In Figure5.6-3 the circle represents hydropower sites and the rectangle represents the demand centers.

This assumption results in the value of number of active transmission lines n_a in the system being equal to the product of number of demand centers and power plants.

$n_a = n.m$

As the optimization progress, some of the redundant transmission lines will be eliminated from the system thus contributing to the minimization. Therefore, the number of active transmission lines will be less than the above product after optimization starts unless the initial layout is optimum already.

i.e. $n_a \leq n.m$

The cost of transmission line can be found by summing up the length of active transmission lines or by using the weighted average length of transmission. The first approach uses equation 5.6-4 whereas the second approach is based on equation 5.6-5

$$KT = \sum_{i=1}^{n}\sum_{j=1}^{m} L_{ji}\xi \qquad \text{Equation 6.6-4}$$

$$KT = \left[\frac{\sum_{i=1}^{n}\sum_{j=1}^{m} PT_{ji}L_{ji}}{\sum_{i=1}^{n}\sum_{j=1}^{m} PT_{ji}}\right](n_a)\xi \qquad \text{Equation 6.6-5}$$

Where , $PT_{ji} \neq 0$

n_a number of active transmission line

ξ Per km cost of power transmission line

Comparing the two formulas, the second one is more realistic from the practical point of view since the cost of accessories of power transmission is expected to increase in proportion with the transmitted power (Zelalem-2002). In addition it is advantageous in numerical computation in multi-variable systems and ease of integration in the objective function.

6.6.5 General form of the objective function

The objective function f is:

Minimize f= $\sum_{j=1}^{m}(\omega_j PG_j^{\vartheta_j} + \beta_j) + \dfrac{\sum_{i=1}^{n}\sum_{j=1}^{m} P_{ji} L_{ji}}{\sum_{i=1}^{n}\sum_{j=1}^{m} P_{ji}}(n_a)\xi$ Equation 6.6-6

Since PG_j is a function of P_{ji}S as given in equation 5.6.2 the value of the objective function is dependent on the values of P_{ji}S which are the decision variables of the system. The minimization yields values of P_{ji}S which when analyzed give the optimum sites for power generation and also the level of optimum power generation at these sites. Moreover, the optimum distribution layout can be ascertained from the values of P_{ji}s.

6.6.6 General form of constraints

The limits of power potential and existing demand of load centers impose constraints on the system. These constraints are described below in mathematical form. In addition to these constraints, there are also bound constraints on each decision variable which are mandatory in some types of optimization techniques.

Satisfaction of the power demand at each site

$\sum_{j=1}^{m} P_{ji} \geq D_i$ j=1,2,…,m Equation 6.6-7

Limitation of power potential at the hydropower sites

$PG_j = \sum_{i=1}^{n}(P_{ji} + Pl_{ji}) \leq P\max_j$ j=1,2,…m Equation 6.6-8

Non-negativity condition of the decision variables

$P_{ji} \geq 0$ $\quad\quad$ i=1,2,…,n

j=1,2,…,m

The sum total of the delivered energy must be greater than the sum total of demand

$$\sum_{i=1}^{n}\sum_{j=1}^{m} P_{ji} > \sum_{i=1}^{n} D_i \quad\quad \text{Equation 6.6-9}$$

Each of the above will be discussed in brief in the following sections.

6.7 System constraints

The objective function to be minimized is within restricted feasible space. The restriction consists of constraints of power potential at the site, domain constraints of the variables, constraints to satisfy the projected demand, and constraints to satisfy balance of power at nodes.

6.7.1 Constraints of power potential

The power potential at a site is limited by the physical and topographical condition of the site and the limits of power potential are set based on the dependable flows which are determined for the sites. In this thesis, the 90% dependable flow is used to establish the maximum potential of the sites for the current analysis. This is used because it is assumed to be the safe value of upper limit of power potential for diversion and run-off river schemes. The sum total of outgoing power from a plant has, therefore, an upper bound due to this restriction. Hence, the sum of outgoing power from the power plants plus the power losses in the transmission lines must be less than the selected upper bound of potential.

6.7.2 Domain constraints

Domain constraints keep the decision variables within bounded upper and lower limits. The lower limit is taken to be zero to satisfy the non-negativity criteria. The upper limit restricts the maximum power transmitted through each transmission line. The domain constraints are defined by the system of inequalities where the units of the boundary values are in KW. Initial values are given which lie within the boundaries and these values are free to change within the domain constraint.

6.7.3 Constraints of power demand

The hydropower system must be able to supply power to all identified demand centers. The demand up to the year 2025 has been calculated in the previous sections by projecting the population size of the demand centers.

6.8 Decision variables

The decision variables for n power plants and m demand centers will be n.m. this is because n hydropower plants are assumed to cover part of the demand of each demand centre at the beginning. The decision could be to supply power from one of the sites or by combination of the sites. This implies that all the sites are given equal chance of remaining in the final optimum system. That way, the final decision will be left open to the optimization which would ascertain the location of the optimal site and the magnitude of power to be developed from it.

6.9 Optimization of Suha and Muga small hydropower

The decision model for this system minimizes the objective function which is composed of the sum cost functions of two proposed power plants. A sketch of the demand centers and hydropower sites is given in figure 5.9-1. For purpose of description of the objective function, the cost equations for the two sites are rewritten in equation 5.9-1. by substituting y by K and X by P.

$K = 0.141(P)^{0.4474}$ (Muga)

Equation 6.9-1

$K = 0.0606(P)^{0.552}$ (Suha)

Equation 5.9-2 is a representation of the hydropower system based on the above two equations. The first four terms in the equation represents the cost of power corresponding to a given level of installation.

The terms in the bracket stand for the power generation level at the site plus the associated power losses in the system corresponding to the given level of power generation.

The last two terms represents the weighted average cost of the transmission system layout.

$$f=0.141\left(\sum_{i=1}^{\frac{n}{2}} X_i + \varepsilon_R \sum_{i=1}^{\frac{n}{2}} l_i (PT_i)^2\right)^{0.4474} + 0.0606\left(\sum_{i=\frac{n+2}{2}}^{n} X_i + \varepsilon_R \sum_{i=\frac{n+2}{2}}^{n} l_i (PT_i)^2\right)^{0.552}$$

$$+\xi\left[\frac{n}{2}\left(\sum_{i=1}^{\frac{n}{2}} (PT_i.l_i) / \sum_{i=1}^{\frac{n}{2}} PT_i\right) + \frac{n}{2}\left(\sum_{i=\frac{n+2}{2}}^{n} (PT_i l_i) / \sum_{i=\frac{n+2}{2}}^{n} PT_i\right)\right] \qquad \text{Equation 6.9-2}$$

Where:

X_i i=1, 2... n decision variables (kw)

l_i i=1,2,…n distance between load centers(km)

PT_i i=1, 2…n power transmitted through a particular line

ε_R =factor of the power loss through resistance of transmission line

ξ =per km cost of power transmission line=50000 Birr

Values of PTi are used to evaluate the power losses through the transmission lines and are defined by the set of expressions in table 5-1based on initial system layout.

Table 6-1-power transmitted through lines

$PT7=X_{11}$	$PT1=X_5$
$PT8=X_{11}+X_{10}$	$PT2=X_4+X_5$
$PT9=X_9$	$PT3=X_4+X_5+X_3$
$PT10=X_9+X_8$	$PT4=X_2$
$PT11=PT8+PT10+X_7$	$PT5=PT3+PT4$
$PT12=X_{12}$	$PT6=X_6$

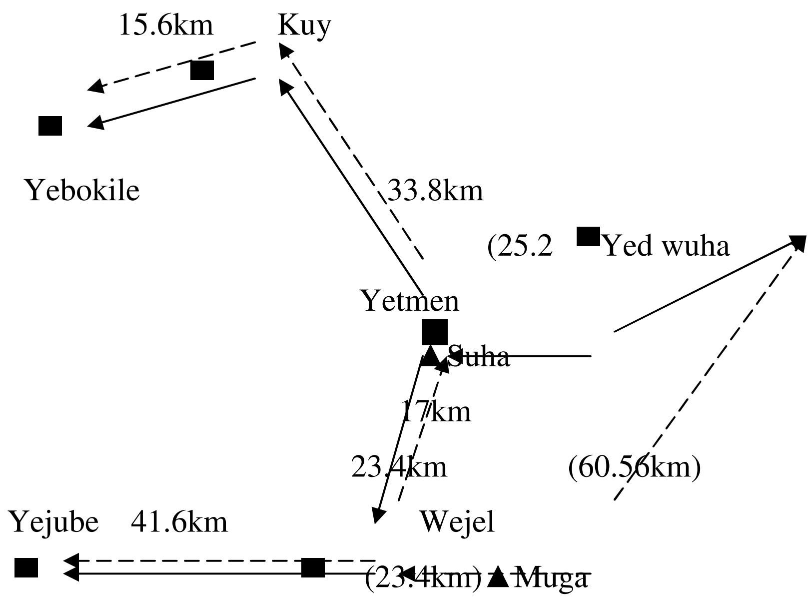


Figure 6.9-1A sketch of demand centers and Muga & Suha Hydropower sites

6.9.1 Constraints of power potential

As discussed in section 5.7.1 above the sum of the outgoing power from the power plants plus the power losses in the transmission lines must be less than the selected upper bound of potential.

Accordingly for Muga and Suha;

$$\sum_{i=1}^{6} Xi + \varepsilon_R \sum_{i=1}^{6} l_i (PT_i)^2 \leq 1653 \quad \text{For Muga} \qquad \text{Equation 6.9-3}$$

$$\sum_{i=7}^{12} Xi + \varepsilon_R \sum_{i=7}^{12} l_i (PTi)^2 \leq 1769 \quad \text{For Suha} \qquad \text{Equation 6.9-4}$$

Where 1653 and 1769 are power potential of Muga and Suha small hydropower sites corresponding to 90% dependable discharge taken from table 3.6.

6.9.2 Domain constraints

As discussed in section 5.7.2 the domain constraints for Muga and Suha are given below.

$$\begin{aligned}
&0 \leq X_1 \leq 125; 0 \leq X_8 \leq 125 \\
&0 \leq X_2 \leq 407.5; 0 \leq X_9 \leq 407.5 \\
&0 \leq X_3 \leq 125; 0 \leq X_7 \leq 125 \\
&0 \leq X_4 \leq 235.1; 0 \leq X_{10} \leq 235.1 \\
&0 \leq X_5 \leq 115.6; 0 \leq X_{11} \leq 115.6 \\
&0 \leq X_6 \leq 181.4; 0 \leq X_{12} \leq 181.4
\end{aligned} \qquad \text{Equation 6.9-5}$$

The upper bound of the domain constraints are the power demand of the demand centers taken from table 3.1.

6.9.3 Constraints of power demand

The equation 5.9-6describes the equality constraints of power demand.

$$
\begin{aligned}
&X_1 + X_8 - D_1 = 0; D_1 = 125\\
&X_2 + X_9 - D_2 = 0; D_2 = 407.5\\
&X_3 + X_7 - D_3 = 0; D_3 = 125\\
&X_4 + X_{10} - D_4 = 0; D_4 = 235.1\\
&X_5 + X_{11} - D_5 = 0; D_5 = 115.6\\
&X_6 + X_{12} - D_6 = 0; D_6 = 181.4
\end{aligned}
\qquad \text{Equation 6.9-6}
$$

Where X_i 1=1…12 are decision variables and D_i i=1…6 are demand centers with corresponding power demands taken from table 3.1.

6.9.4 Decision variables

As discussed in section 5.8 though only two hydropower plants are involved, the decision variables actually used for optimization are 12.

6.9.5 Nature of the objective function

As discussed in section 5.2 the nature of the cost function gives an idea as to the certainty in finding the global value. If the objective function is a convex one, the local optimum is the same as the global optimum. In contrast to this if the objective function is concave; the local optimum may not be the global optimum. If this is the case, the investigation could not be restricted to convex programming techniques.

In order to determine the nature of the cost function the 12 decision variables are summarized to 2 major decision variables.

The surface of the hydropower cost function is illustrated in the three dimensional plot in figure 5.9-2 the surface is a non-convex type as shown by the surface of cost function.

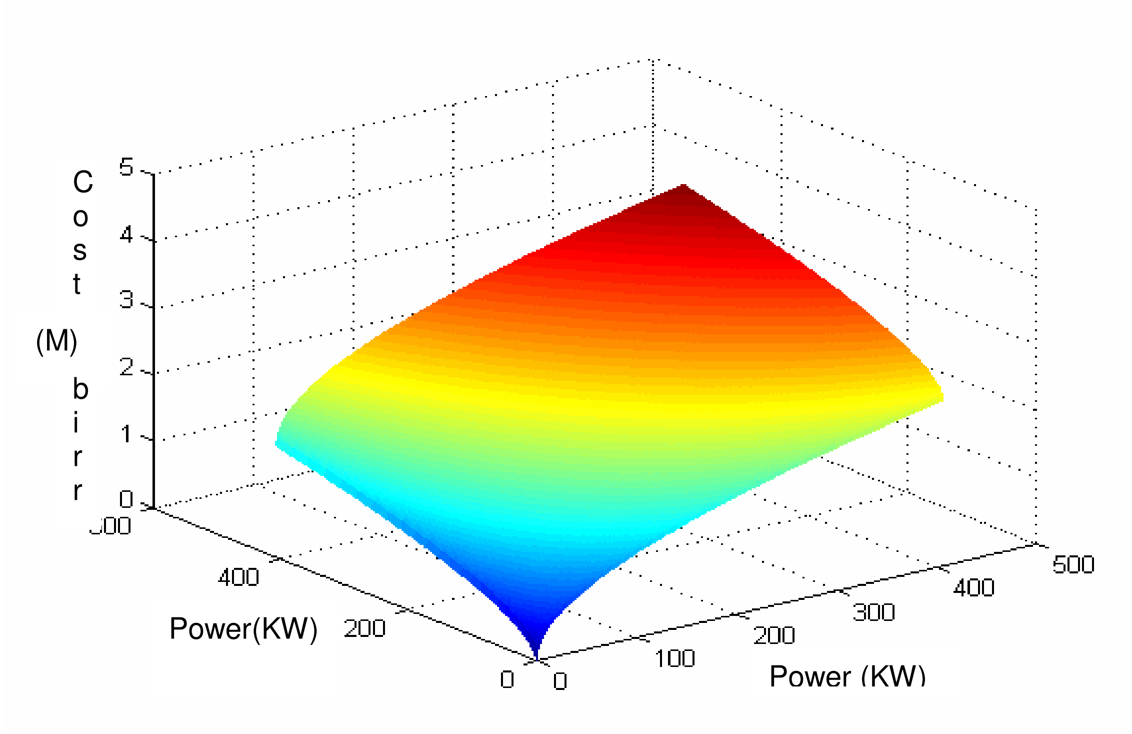


Figure 6.9-2 surface of cost function for Suha and Muga

6.9.6 Initialization of decision variables

Optimization with the gradient methods may sometimes fail to yield an appreciable result. In such cases, it is advisable to use other sets of initial values for the optimization. For optimizing Suha and Muga sites, a possible set of initial values were given which assume that each load centre gets half of its power from one and the other half from the other power plant. Table 5-2 shows the set of initial values. As the optimization progresses, these values show change in favor of the better site or optimum level of development at the site. Based on the outcome of each optimization run, another set of initial values can be assumed for repetitive optimization.

Table 6-2 initial assumed values

INITIAL VALUES ASSUMED
X_5=X_{11}=57.8KW
X_4=X_{10}=117.55KW
X_3=X_7=62.5KW
X_2=X_9=203.75KW
X_8=X_1=62.5KW
X_6=X_{12}=6=90.7KW

Table 6-3-Results of optimization

Power transmitted	Original value of power transmitted(KW)	Final value of power transmitted(KW)	Decision variables final value(KW)
PT1	57.8	115.6	0
PT2	175.35	350.7	0
PT3	237.85	350.7	0
PT4	203.75	0	235.1
PT5	504.1	350.7	115.6
PT6	90.7	0	0
PT7	57.8	0	125
PT8	175.35	0	125
PT9	203.75	407.5	407.5
PT10	266.25	532.5	0
PT11	504.1	657.5	0
PT12	90.7	181.4	181.4

From the tables 5-2 &5-3and the sketch of demand centers and hydropower site, the optimum value of the decision variables and the optimum power to be transmitted through each transmission line can be seen. In addition, the optimum level of development of each hydropower site can determined. Hence, the optimum level of development at Muga site is 350.7KW whereas that of Suha site is 838.9KW. The total cost of the development reduces from 16,502,697.86 to 15,342,161.29 in the course of optimization.

6.9.7 Optimization results

Regardless of the starting point, three of the five trials in the fig 5.9-3 ended up at the same value of the objective function. If the terminal point of the three trials is to be assumed to be the global optimum, then the other two trials which failed responding solver encountered error with target or constraint indicates the necessity of changing initial values assumed. The three values terminating at optimum value have indicated an improvement of 7% as indicated in the figure 5.9-3 below.

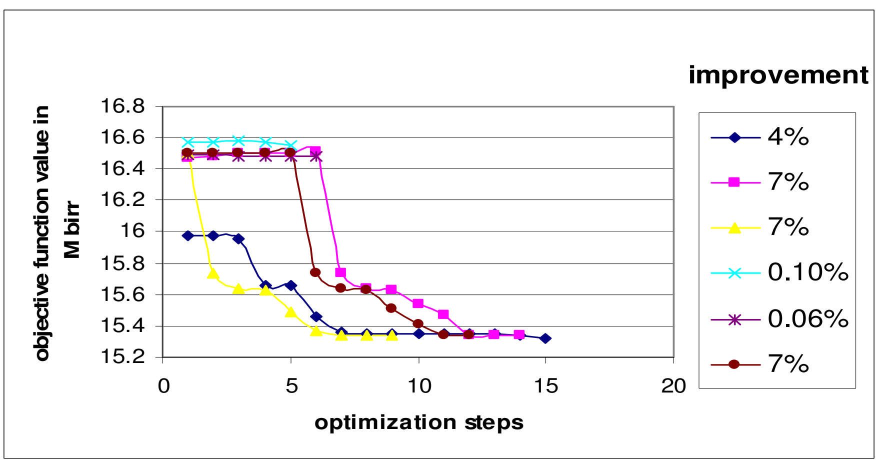


Figure 6.9-3 improvement in the objective function

6.10 Optimization of G.Abbay and Andassa small hydropower

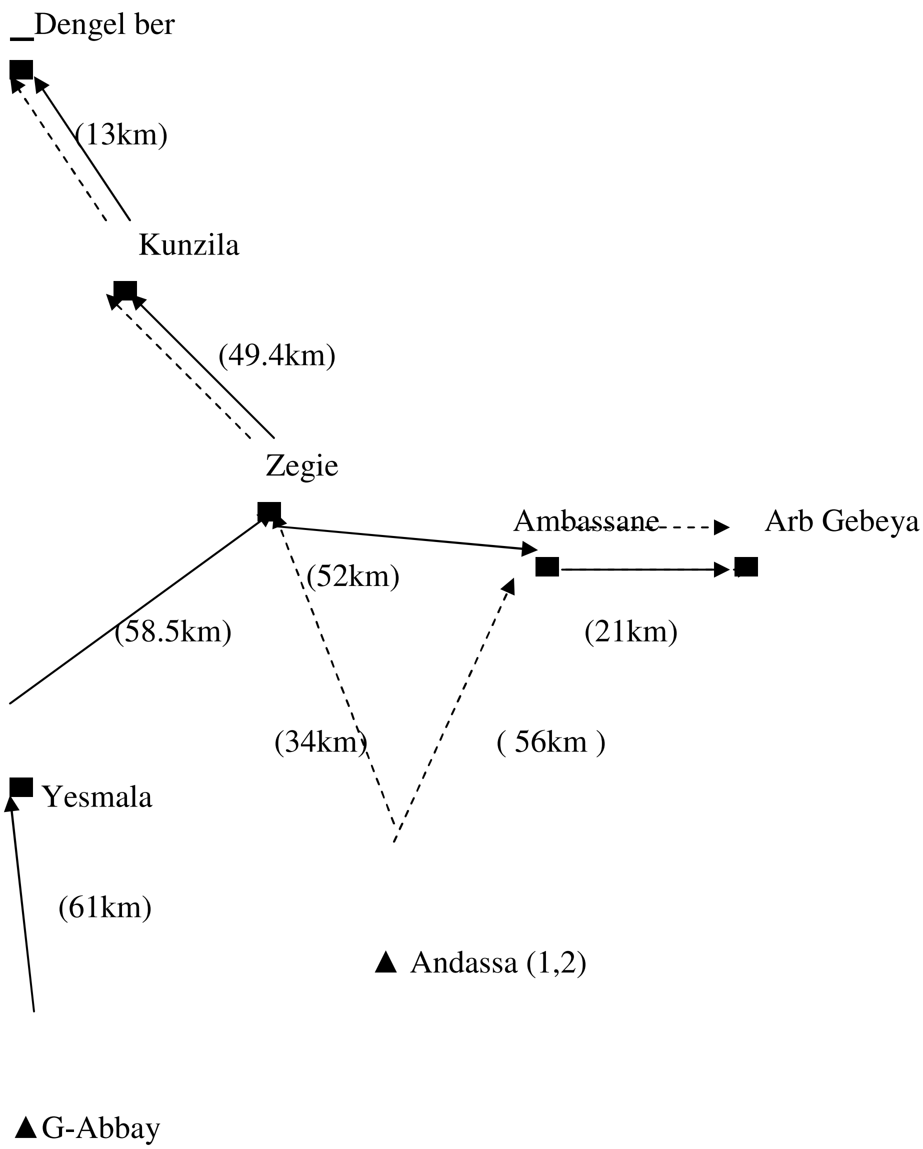


Figure 6.10-1 sketch of demand centers and G.Abbay & Andassa(1,2) SHP sites

Table 6-4 optimization results for Andassa(1,2) and Gilgel Abbay

	Decision variables		Power transmitted		Level of development		
S-no	Initial value	Final value	Initial value	Final value	Andassa -1	Andassa-2	G-abbay
1	17.2	57.60	17.2	57.60	170.80	115.11	1333.88
2	34.4	0	51.6	57.60			
3	34.4	113.20	86	170.80			
4	34.4	0	34.4	0			
5	17.2	0	51.6	0			
6	34.4	0	34.4	0			
7	73.91	55.92	73.91	55.92			
8	73.91	0	147.82	55.92			
9	73.91	59.20	221.73	115.11			
10	73.91	0	73.91	0			
11	36.955	0	73.91	0			
12	36.955	0	36.955	0			
13	44.69	39.50	44.69	39.50			
14	255.69	364.00	300.38	403.50			
15	71.69	7.60	822.06	969.40			
16	256.19	364.50	1078.25	1333.88			
17	213.145	267.30	449.99	558.30			
18	236.845	291.00	236.845	291.00			

6.11 Optimization of Dura and Ardy SHP site

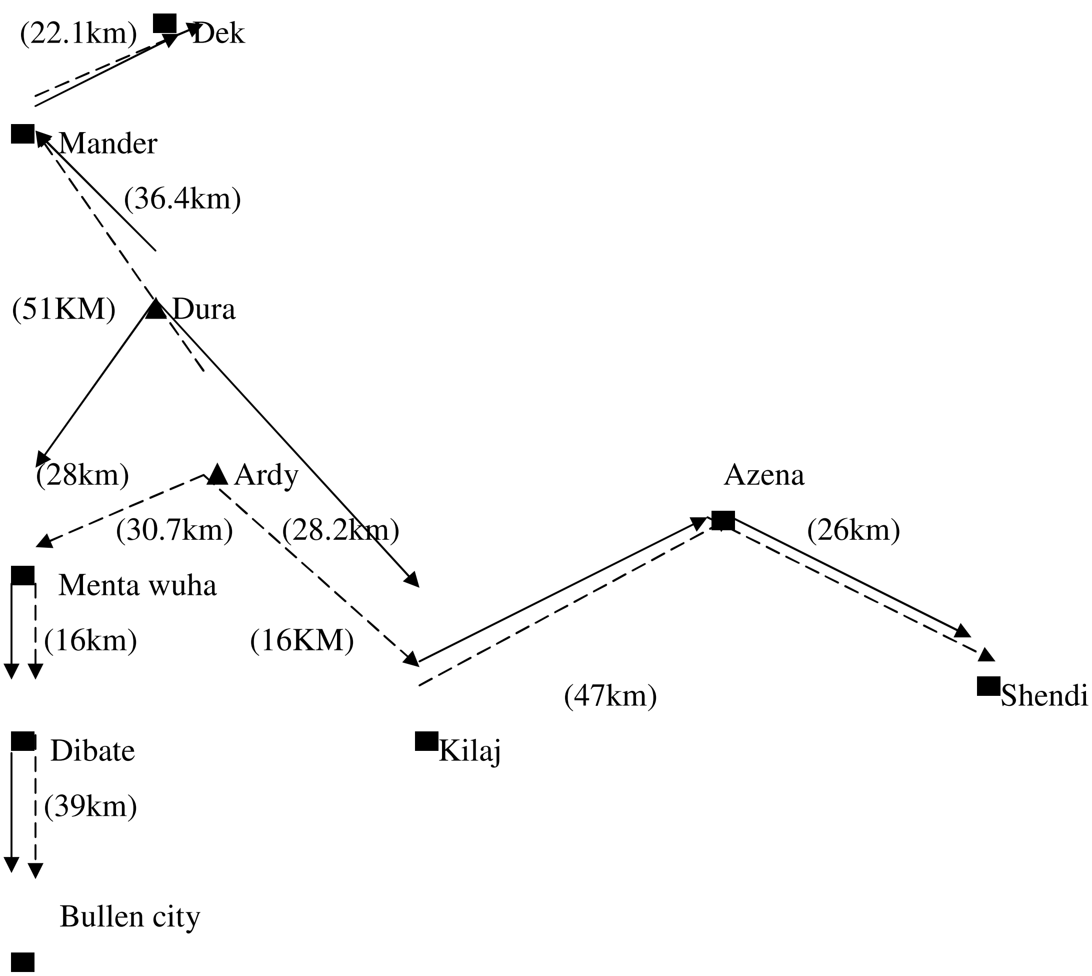


Figure 6.11-1 sketch of demand centers and Ardy & Dura hydropower sites

▲-Represents small hydropower site

■-Represents Demand centers

**-The distance between is given in brackets by providing 30% increase for the purpose of considering Zigzags during the actual laying of transmission line.

Table 6-5 Optimization Results for Ardy and Dura

S -no	Decision variables		Power transmitted		Level of development	
	Initial value	Final value	Initial value	Final value	Dura	Ardy
1	96.73	170.73	96.73	170.73	2124.1	470.44
2	93.46	157.46	190.19	328.19		
3	462.84	512.84	1025.45	964.5624		
4	252.66	96.77238	562.61	451.7224		
5	309.95	354.95	309.95	354.95		
6	175.55	0	906.9	831.35		
7	171.63	246.63	731.35	831.35		
8	560.72	584.72	559.72	584.72		
9	74	0	74	4.62E-14		
10	64	0	138	4.62E-14		
11	50	0	159	219.8876		
12	64	219.8876	109	219.8876		
13	45	4.87E-14	45	2.84E-14		
14	75	250.55	175	250.55		
15	75	0	100	0		
16	24	0	25	0		

6.12 Optimization of Temcha and Azuari

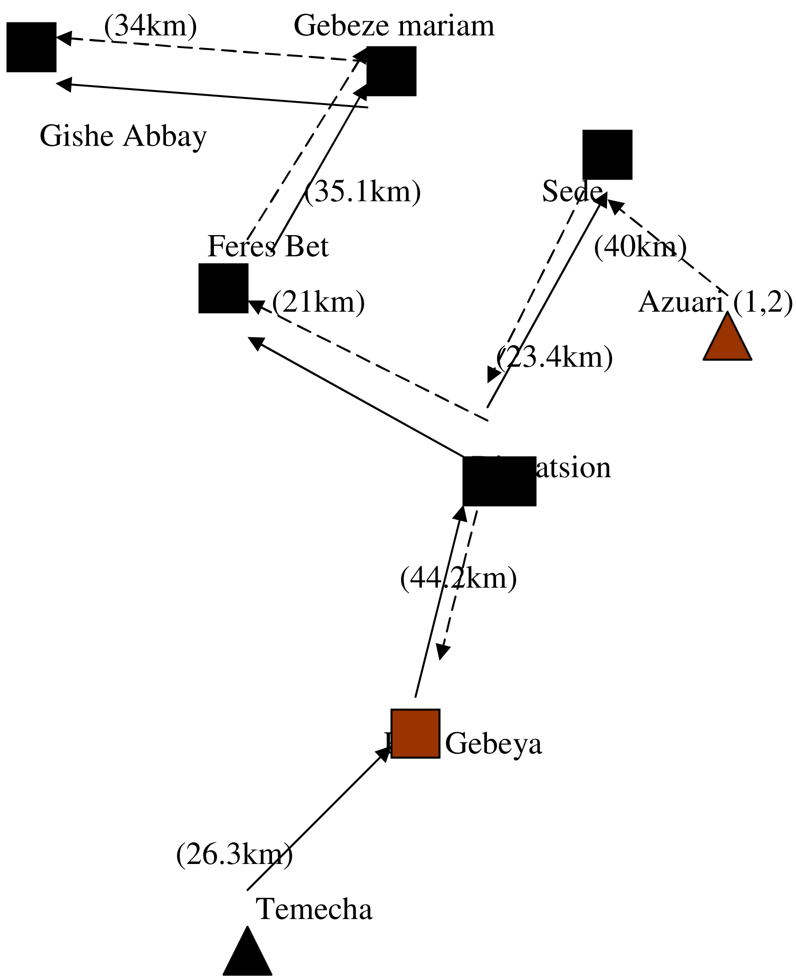


Figure 5.12-1 sketch of demand centers and Azuari (1, 2) & Temcha SHP site

Table 6-6 optimization results of Temcha and Azuari (1,2)

	Decision variables		Power transmitted		Level of development		
S-No	Initial value	Final value	Initial value	Final value	Temecha	Azuari - 1	Azuari - 2
1	40	49.91	463.62	484.966656	484.97	34.88	387.80
2	50	53.118894	423.62	435.056656			
3	85	62.380876	293.36	289.175792			
4	113.36	123.28646	208.36	226.794916			
5	95	103.50846	95	103.508458			
6	80.26	92.76197	80.26	92.7619697			
7	15	0	74.5	34.8795183			
8	20	1.165E-09	59.5	34.8795183			
9	10	0	10	0			
10	9	34.879518	29.5	34.8795183			
11	10	0	20.5	0			
12	10.5	0	10.5	0			
13	56	53.49803	395.79	387.803825			
14	40	41.711106	339.79	334.305795			
15	7	0	7	0			
16	89.43	87.169606	292.79	292.594689			
17	95	95.073542	203.36	205.425084			
18	108.36	110.35154	108.36	110.351542			

7 Conclusion and recommendation

The promotion of SHP is an important pre-requisite for the prevention of deforestation and the associated environmental degradation in Ethiopia. People in rural Ethiopia live in unattractive environment due to the lack of energy and also suffer the consequences of climatic change caused by associated environmental impacts.

In spite of the abundant hydropower potential of the country, there are several thousands of rural villages without electric energy supply. The energy supply status of a majority of the villages is unknown. Therefore; the investigation of small hydropower system and the applicability of resource allocation methodologies are necessary. Optimization provides flexible method which is able to assess the hydropower cost function. By applying optimization, it is possible to compare alternative site through site specific cost functions which summarize the specific features of each site. The cost functions developed in this thesis relate the costs of power production to the installation capacity. This enables the consideration of favorable features of each alternative site as a primary factor, in the selection of optimum site(s).

In addition to the site specific costs, the cost of transmission systems is included as the other important factor in the optimization. As a result, it was possible to develop a decision making tool that considers favorable site conditions as well as proximity of demand centers to alternative sites in the decision making. The optimization is applied in the search for optimum hydropower site or sites and their level of development. This technique could be extended to other basins provided that a representative mathematical expression of the system to be optimized is established. Though the optimization could be used in prioritizing small hydropower sites, it could be supported by the B/C ratio for final decision. For sites with the same benefits, it could be noted that the optimum site favored by optimization will have the maximum B/C ratio among the considered sites.

The objective functions developed in this thesis considered a system cost for a stand alone run-off-river (diversion) development. As mentioned previously, these small hydropower sites are taken from the sites studied by the study team from P.R.China (1989). Arc view spatial analysis technique has been applied by developing (creating) the map of the basin. With the current development of spatial analysis techniques, proper analysis and study can be conducted for such research activities. The objective functions are developed after identifying the necessary components of the small hydropower at each site. All the components of small hydropower are subject to optimization by taking different alternative layouts with the same or different head for optimization. In this thesis only discharge is varied to establish the objective function as it is based on layouts prepared during previous study.

Pre-feasibility and feasibility studies of small hydropower development should not end by merely making a preliminary design of a chosen installation capacity for a specified demand. Rather there should be a need to establish cost function of the investigated hydropower site which can characterize the site in a unique manner.

Information collected during the execution of such studies should be documented so that the information gap in this area could be minimized.

Small hydropower is technically more feasible than large hydropower for covering rural energy demand in remote localities. Grid-based central power plants are unable to promote rural electrification in Ethiopia, However; the constraints on decentralized energy systems in several and on small hydropower in particular are much more complex. Hence, attention should be given to small hydropower plant and other decentralized power supply scheme development.

It is also apparent that rural electrification should be treated as part of infrastructural development in Ethiopia for development of country in relation to rural centered development policy of the government.

Generally, communal and private investors engaged in rural electrification should be encouraged.

Efforts should be made to develop an all-rounded data base of the small hydropower resources of the country.

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Appendices

Appendix 4.1coordinates of demand centers and hydropower sites

	Location of demand Centers		
s-no	Town-name	Lat_dd	Lon_dd
1	Rob Gebeya	10.53	37.78
2	Kuy	10.5	38
3	Yetmen	10.31	38.16
4	yed Wuha	10.44	38.36
5	Wegel	10.21	38.04
6	Dibate	10.78	36.26
7	Menta wuha	10.71	36.35
8	Kilaj	10.7	36.5
9	Azena	10.75	36.82
10	Sede	10.92	37.91
11	Shendi	10.64	36.95
12	Mander	11.1	36.43
13	Dek	11.21	36.52
14	Gishe Abbay	10.99	37.23
15	Feres Bet	10.86	37.6
16	Digua Tsion	10.84	37.75
17	Yesimala	11.6	36.9
18	zege	11.7	37.32
19	Kunzila	11.88	37.05
20	Dengel Ber	11.96	37
21	Ammasane	11.71	37.64
22	Arb Gebeya	11.64	37.75
23	Yejub	10.14	37.75

24	Yebokile	10.44	37.91
25	Bullen	10.59	36.08
26	Gebeze Mariam	11.03	37.44
	Location Of SHP sites		
S_no	site Name	Lat_dd	Lon_dd
1	Dura	10.96	36.52
2	Ardy	10.94	36.56
3	Suha	10.34	38.24
4	Auari	10.95	38.05
5	Temecha	10.5	37.62
6	Muga	10.14	38.24
7	G.Abbay	11.32	36.98
8	Andassa	11.54	37.38

Appendices

Appendix 4.1- coordinates of SHP sites and selected demand centers

Appendix 4.2- sample calculation of demand determination using the approach used in this thesis

Appendix 4.3-calculation of demand determination using the EEPCo method for Rural electrification (for comparison)

Appendix 4.4-Attributes of Abbay basin gauging stations

Appendix 4.5-correlation between concurrent discharges for near by stations

Appendix 4.6-flow duration curves for selected sites

Appendix 4.7-Attributes of hydropower sites in the Abbay Basin (USBR-1964)

Appendix 5.1-Layouts of selected small hydropower sites

Appendix 5.2-cost functions for the selected sites

Appendix 5.3-optimization results for suha and Muga small hydropower sites

Appendix 5.4-Flow charts

Appendix 4.2 Sample power demand Calculation by the approach used in the thesis