# COST EFFECTIVE DESIGN OF THE LOW HEAD SMALL HYDRO PROJECT

**A DISSERTATION**

*Submitted in partial fulfillment of the requirements for the award of the degree*
*of*
**MASTER OF TECHNOLOGY**
**in**
**ALTERNATE HYDRO ENERGY SYSTEMS**

By

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# Declaration

I hereby declare that the work being presented in this Dissertation report entitled "**COST EFFECTIVE DESIGN OF THE LOW HEAD SMALL HYDRO PROJECT**" in partial fulfilment of the requirement for the award of the degree of **MASTER OF TECHNOLOGY IN ALTERNATE HYDRO ENERGY SYSTEMS**, submitted in the Alternate Hydro Energy Centre, Indian Institute of Technology Roorkee, is an authentic record of my own work carried out during the period from August, 2008 to May, 2010 under the guidance of Dr. S. K. Singal, Senior Scientific Officer, Alternate Hydro Energy Centre, Indian Institute of Technology, Roorkee.

I have not submitted the matter embodied in this Dissertation report for the award of any other degree.

Dated: 31 May 2010

Place: IIT Roorkee

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## CERTIFICATE

This is to certify that declaration made by the candidate is correct to the best of my knowledge and belief.

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# ABSTRACT

Most of the energy we use today comes from non-renewable energy sources like coal, oil, fossil fuels etc. These sources are being consumed much rapidly than they are created. Also use of such resources leads to pollution and environmental issues. Because our world depends on energy, we need to use sources of energy that will last forever; these sources are called renewable sources where occur repeatedly in nature. Renewable energy resources are solar, wind, biomass and small hydro power (SHP), tidal, geothermal.

Among renewable sources small hydro power is one of the most widely used source of electricity generation in the world. With rapidly depleting fossil fuels and increasing power demand, more attention is concentrating towards hydro power, particularly smaller sites, under low head which are untapped till now.

As the cost of construction is increasing rapidly, special attention has to be given to the development in economically viable manner. The main components of SHP are civil works and electromechanical works. Most of the components are common in different type of schemes. Thus the present study is planned with following aspects

i. Study of various components of low head small hydro power schemes.
ii. Carried out sizing of various components under civil works and selection of electro mechanical equipment for different schemes.
iii. Development of software for determining the sizes of components.
iv. Analysis of cost of different components for developments of cost correlation in order to determine the total installation cost.
v. Development of software for cost computation of power house building having different types of layouts.
vi. Hydraulic design of different types of spillway, generation of profile of spillways and cost calculation of various types of spillways.

It is seen that low head projects are inherently expensive, as the discharges, which need to be passed through the turbines require large size machines. It is therefore always desirable to minimize their capital cost. Civil structures cost contribute to a major part of the costs in these projects.

Software programming has been developed to predict the power house cost based on type of turbine, dimensions of civil components under given conditions of head and capacities. To analyze the cost of low head SHP projects, cost data were analyzed for development of correlation. The correlation for cost of all civil components, electromechanical equipments, has been developed by linear regression method for run-of-river scheme.

The cost effective layout under low head scheme has been determined based on type of turbine. It has been found that the power house building contribute maximum in the cost. It depends on type and number of turbine. The canal based and dam toe scheme under low head also the power house building is the important structure. The software has been developed to determine the cost of power house building to find out cost effective layout.

In dam based project the components are intake, spillway, penstock, power house building, and tailrace channel. The components penstock, power house and tailrace are similar to the components in run-of river scheme. However the spillway is different. Spillway design and generation of profile curve has been carried out using Indian standard code. The volume and area has been calculated using GAMBIT software.

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# NOMENCLATURE

| | | |
|---|---|---|
| h | - | Head |
| $H_a$ | - | Head due of velocity approach |
| $H_c$ | - | Head from reservoir level up to the centerline of the opening of the gate, |
| $H_d$ | - | Design Head |
| H | - | Head of overflow |
| Hd | - | Head due to to velocity of approach |
| Q | - | Discharge |
| V | - | Velocity of approach |
| P | - | Height of the Spillway crest measured from the river Bed. |
| P1 | - | Power generated in kW |
| A | - | Horizontal Dimension defining upstream quadrant of the crest, |
| B | - | Vertical dimension defining upstream quadrant of crest |
| C | - | Non – dimensional discharge coefficient |
| Cb | - | Discharge coefficient for spillways with breast wall, |
| Cd | - | Discharge coefficient for design head |
| Cg | - | Discharge coefficient or flow under the gate |
| D | - | Net opening for the spillway with breast wall |
| Go | - | Gate Opening |
| g | - | Acceleration due to gravity |
| K | - | Variable Parameter |

Ka - Abutment contraction co-efficient

Kp - Pier contraction co-efficient

L - Effective length of overflow crest

L' - net length of overflow crest

M - Riser of Crest

N - Number of Piers

$n_1$, $n_2$ - Variable Parameters

X, Y - Co. Ordinates of the profile

$\beta$ - Angle formed by the tangent to the gate lip and the tangent to the gate lip and the tangent to the crest curve at the nearest point of crest curve.

# CHAPTER 1

# INTRODUCTION

## 1.1 PRESENT SCENARIO OF ENERGY

Energy is the primary and most universal measure of all kinds of development by human being and nature. Everything, what happens in the world is the expression of flow of energy in one of its form. Energy is an essential input for economic development and improving the quality of life. The conventional sources of energy are depleting and may be exhausted by the end of the century or beginning of the next century. Hence the task will be to manage a transition from a dependence on conventional source of energy and greater reliance on other fossil fuels, nuclear energy to renewable energy.

Global energy related carbon dioxide ($CO_2$) emission is expected to increase by 55% between 2004 and 2030, or 1.7% per year, in the reference scenario. They may reach 40 gigatonnes (Gt) by the year 2030. Coal overtook oil in 2003 as the leading contributor to global energy-related $CO_2$ emissions. Thus interventions are required to include efforts to improve in energy production and use, to increase reliance on non-fossil fuels energy that is clean and green like hydro ,wind, solar and bioenergy [1].

Hydropower is a renewable, non –polluting and environmentally benign source of energy. It is perhaps the oldest renewable energy techniques known to the mankind for mechanical energy conversion as well as electricity generation. Hydropower represents use of water resources towards inflation free energy due to absence of fuel cost with mature technology characterized by highest prime moving efficiency and spectacular operational flexibility. Hydro power contributes around 22% of the world electricity supply generated from about 7,50,000 MW of installed capacity. In many countries, it is the main source of power generation e.g. Norway - 99%, Brazil-86%, Switzerland-76% and Sweden -50% [1]. Despite hydroelectric projects being recognized as the most economic and preferred source of electricity, the share of hydropower in India has been declining since 1963. For grid stability the ideal hydro- thermal mix ratio is considered as 40: 60. It is therefore, necessary to improve the hydro-thermal ratio to meet the grid requirements and peak power shortage.

### 1.1.1 Hydro Power Potential in India

The present installed capacity of power generation in India is 1,59,398 MW [2] out of which hydro power is 26%, thermal is 66% renewable is 5% and nuclear is 3 %. The scenario of energy generation from different sources and consumption up to 2009 are as follows.

*Table 1.1 Renewable energy in India at a glance as on 31.03.2009*

| Sr. No. | Major Programme/ System | Cumulative Achievements |
|---|---|---|
| 1 | Biomass power ( 703.3 MW) | 703.30 MW |
| 2 | Wind Power | 10,242,52 MW |
| 3 | Small Hydro Power (up to 25 MW) | 2,429.67 MW |
| 4 | Cogeneration – Biogases | 1048.73 MW |
| 5 | Family Type Biogas Plans (nos.) | 41.2 Lakh |
| 6 | Street lighting systems | 75,376 Nos. |
| 7 | Home lighting Systems | 434692 Nos. |
| 8 | Solar Lanterns | 697419 Nos. |

Small Hydro Projects Potential, installed & Under Implementation (As on 31.03.2009)

*Table 1.2 State wise number and Aggregate Capacity (up to 25 MW)[2]*

| Sr. No. | State | Potential | | Projects Installed | | Projects under Implementations | |
|---|---|---|---|---|---|---|---|
| | | Nos. | Total Capacity | Nos. | Capacity (MW) | Nos. | Capacity |
| 1 | Andhra Pradesh | 489 | 552 | 59 | 180.830 | 12 | 21.50 |
| 2 | Arunachal Pradesh | 566 | 1333 | 81 | 61.320 | 43 | 25.94 |
| 3 | Assam | 60 | 213 | 4 | 27.110 | 4 | 15.00 |
| 4 | Bihar | 94 | 213 | 12 | 54.60 | 4 | 3.40 |
| 5 | Chhattisgarh | 164 | 706 | 5 | 18.50 | 1 | 1.00 |
| 6 | Goa | 9 | 9 | 1 | 0.050 | - | - |
| 7 | Gujarat | 292 | 196 | 2 | 7.000 | 2 | 5.60 |
| 8 | Haryana | 33 | 110 | 5 | 62.700 | 1 | 6 |
| 9 | H.P. | 547 | 2268 | 79 | 230.915 | 9 | 26.75 |
| 10 | J & K | 246 | 1411 | 32 | 111.830 | 5 | 5.91 |
| 11 | Jharkhand | 103 | 208 | 6 | 4.05 | 8 | 34.85 |
| 12 | Karnataka | 128 | 643 | 83 | 563.450 | 14 | 85.25 |
| 13 | Kerala | 247 | 708 | 19 | 133.870 | 2 | 3.20 |
| 14 | M.P. | 99 | 400 | 10 | 71.160 | 4 | 19.90 |
| 15 | Maharashtra | 253 | 762 | 29 | 211.325 | 5 | 31.30 |
| 16 | Manipur | 113 | 109 | 8 | 5.450 | 3 | 1.70 |
| 17 | Meghalaya | 102 | 229 | 4 | 31.030 | 3 | 1.70 |
| 18 | Mizoram | 75 | 166 | 18 | 24.470 | 1 | 8.50 |
| 19 | Meghalaya | 102 | 229 | 4 | 31.030 | 3 | 1.70 |
| 20 | Mizoram | 75 | 166 | 10 | 24.470 | 1 | 8.50 |
| 21 | Nagaland | 99 | 196 | 10 | 28.670 | 4 | 4.20 |
| 22 | Orissa | 222 | 295 | 8 | 44.300 | 6 | 23.93 |
| 23 | Punjab | 234 | 390 | 29 | 123.900 | 2 | - |
| 24 | Rajasthan | 67 | 63 | 10 | 23.850 | - | 5.20 |
| 25 | Sikkim | 91 | 265 | 16 | 47.110 | 2 | 13.00 |
| 26 | Uttarakhand | 458 | 1609 | 93 | 127.920 | 33 | 79.25 |
| 27 | Uttar Pradesh | 220 | 292 | 9 | 25.100 | - | - |
| 28 | West Bengal | 203 | 393 | 23 | 98.400 | 16 | 79.25 |
| 29 | Andaman & Nikobar Island | 12 | 8 | 1 | 5.250 | - | - |
| | Total | 5415 | 14,292 | 674 | 2429.770 | 188 | 483.23 |

## 1.2 SMALL HYDRO POWER

### 1.2.1 Definition of SHP

A SHP station is an installation where hydraulic power is used to generate electricity by means of one or more turbine-generator units. In India the power plants having installed capacity up to 25 MW are considered small hydropower plants. Internationally, there is no concern over capacity of small hydropower projects- It varies from 5 MW to 50 MW.

### 1.2.1 Classification of Small Hydro Power Plants

The hydropower plants can be classified according to their function as follows. [3]

(1) Run-of-River

(2) Canal Based

(3) Dam Toe Based

(4) Pump Storage

#### 1.2.1.1 Run-Of-River Plants

Run-of-river SHP plants are those which utilize the instantaneous flow having no pondage. A weir or barrage is constructed across the river, to raise the water level slightly. Such a scheme may be low head, medium scheme or high head scheme, and is adopted in case of a perennial river which has minimum dry weather flow of such magnitude which makes the development worthwhile.

Run-of-river plants are of two types: (i) Those utilize the varying flow, and (ii) Those operate on minimum available discharge and work throughout the year. A typical layout of Run-of-River scheme is shown in Figure 1.1.

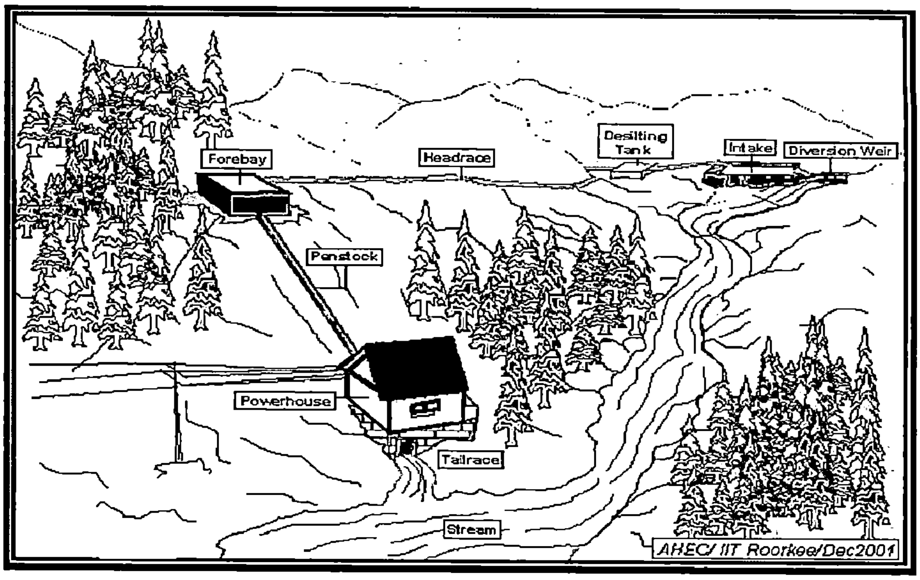



*Figure 1.1 Typical Arrangement of Run-of-River Scheme*

### 1.2.2.2 Canal based Plants

Canal based small hydropower scheme is planned to generate power by utilizing the discharge and fall in the canal. These schemes may be planned in the canal itself or in the bye pass channel. These are low head and high discharge schemes. These schemes are associated with advantages such as low gestation period, simple layout, no submergence and rehabilitation problems and practically no environmental problems. The typical canal based scheme is shown in Figure 1.2.

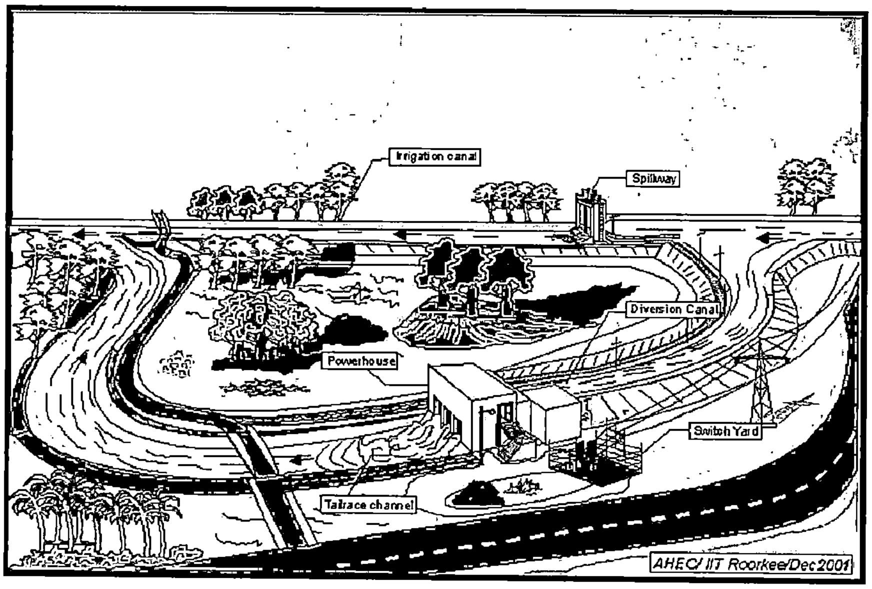



*Figure 1.2 Typical Arrangement of Canal based Scheme*

### 1.2.2.3 Dam Toe Plants

In this case, head is created by raising the water level behind the dam by storing natural flow and the power house is placed at the toe of the dam or along the axis of the dam on either sides. The water is carried to the powerhouse through penstock. Such schemes utilize the head created by the dam and the natural drop in the valley. Typical dam toe based scheme is shown in the Figure 1.3.

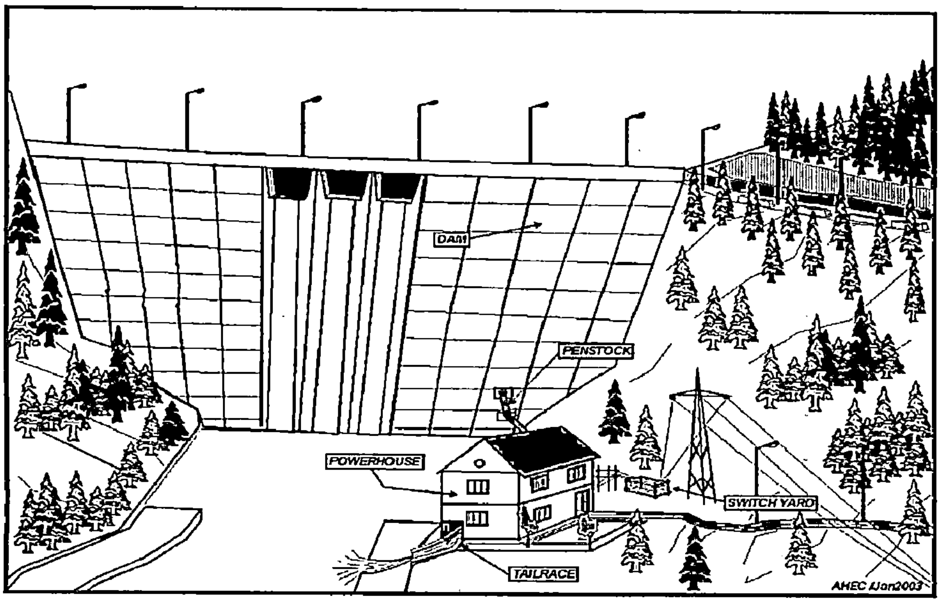



*Figure 1.3 Typical Arrangement of Dam Toe Scheme*

### 1.2.3 Classification of SHP based on their Head

The net head and the plant discharge are two important parameters to be considered in the small hydropower design. The following classification is adopted based on hydraulic head under which the turbine will operate [4]:

1. Ultra Low head scheme Below 3 meter
2. Low head scheme Less than 20 meter
3. Medium / High head scheme Above 25 meter

## 1.3 BASIC COMPONENTS OF SHP SCHEMES

Basic components of SHP schemes are broadly categorized into two parts:

i. Civil works

ii. Electromechanical equipment

Most of the components are common in different types of schemes; however, some components are different for different schemes. A broad classification of SHP components is given in figure 1.4 as follows.

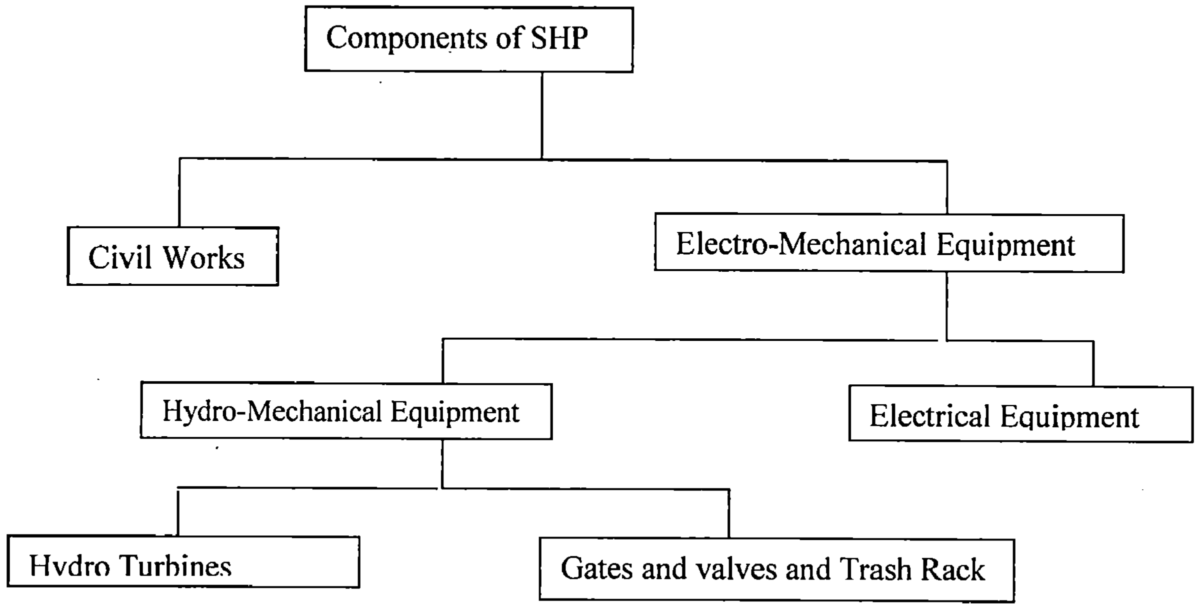



*Figure 1.4 Different Components of Small Hydro Power Schemes*

## 1.4 ADVANTAGE OF HYDRO ENERGY OVER OTHER RENEWABLE SOURCES [5]

Hydro energy has very interesting properties compared to other sources of energy:

- It is one of the cheapest renewable sources of energy. Hydropower energy is mainly in competition with thermal source of energy. Because of increasing prices on the electricity market during the recent past years, small and medium-sized hydropower plants are coming under increasing pressure.

- Impact on environment is minimal, if sufficient precautions are taken.
- Long term planning for hydro energy is possible and long-term assurance of viable payments for supply to the network and stable costs can be guarantee compared to fluctuating prices of the fossil energy.
- Due to fast start or stop of hydraulic turbines and the large operating range of these machines, hydro energy permit easy control of load on the grid. Comparable flexibility is only possible with gas turbines.
- The possibility of energy storage in a reservoir permits to manage the production in the best economical interests, to store energy during off-peak hours and to release during peak hours.
- Hydro plants are well adapted to decentralized energy production in remote areas and easily adjustable to local energy demand.
- Hydraulic power permits very secure regulation techniques, that permits to guarantee high quality of current in comparison of other sources like wind energy where rapid unpredictable fluctuations are present.

### 1.5 LITERATURE REVIEW

R. Montanari [6] in his paper presents an original method for finding the most economically advantageous choice for the installation of micro hydroelectric plants. More precisely, the paper that follows is to be considered in a context defined as "problematic" by those who have the job of constructing water-flow plants with only small head and modest flow rates. Traditional plant solutions using Kaplan or Francis type turbines must be rejected because of the high levels of initial investments. Much more simple configurations must be analyzed, such as plants with propeller turbines or Michel–Banki turbines, in order to reduce the investment costs. The general methodology applied provides a powerful decision-making instrument which is able to define the best plant configuration. The method is based on the use of economic profitability indicators, such as the Net Present Value (NPV), calculated using the plant project parameters, the nominal flow rate and head, and the particular hydrologic characteristics of the site, such as the type of distribution, the average value and the standard deviation of the flow rates in the course of water supplying the plant.

S.K. Singal and R.P.Saini [7] has presented methodology to determine the correlations for the cost of different components of canal based small hydro power schemes. The cost based on the developed correlations, having different head and capacity, has been compared with the available cost data of the existing hydropower stations. It has been found that these correlations can be used reasonably for the estimation of cost of new canal-based SHP schemes.

K.V. Alexander, E.P. Giddens [8] in their paper an overview of a program that is in the final stages of developing a modular set of cost-effective micro hydro schemes for site heads below those currently serviced by Pelton Wheels. The rationale has been that there is a multitude of viable low-head sites in isolated areas where micro hydro is a realistic energy option, and where conventional economics are not appropriate, especially in Third World countries. The goals of this project have been to provide low-cost, soundly based turbine design solutions that systematically cover the 0.2–20kW supply, that are uniquely resistant to debris blockage and are easily built by tradesmen of medium skills in regional workshops. The paper presents the results as a matrix of the most cost-effective penstocks matched to modular turbines using established electronic controls. It discusses practical issues of site selection and options for sites where exact matches are not achieved.

S.K. Singal and Varun [9] in his paper planning and designs of small hydroelectric schemes is an evolving process leading to safe and cost effective refinements in designs. The major factor for high cost of civil works of the these schemes is conventional designs coming out of designers with a mind set, that of miniaturizing a major hydro model for small/mini hydro, there-by including many of the components not required or used in small hydro operation at all. First and foremost step needed is to break this mindset and reduce the civil cost of small hydro projects by innovative and practical designs. The lesson learnt from the experience and use of new technologies make small hydropower plants economically viable. Use of local materials and site-specific design/solutions make the scheme cost effective and reduce the operation and maintenance cost.

O.D. Thapar [10] presented that existing state of art in respect of relevant alternative technologies available for small low head and micro hydro power plants with particular reference to turbine. Author that the water resources, though renewable, are certainly limited. Their exploitation hence will depend on careful application of technological innovation. Small scale hydro-power development may have only a small impact on the locality in which it is sited can be significant in as much as it can help stimulate growth of rural industry and assure the population of basic energy needs comprising of domestic, community and agriculture requirement.

US Army Corps of Engineers [11] presented discussion of the general, architectural & structural consideration applicable to the design of hydroelectric power plant structures. Discussions could be used in establishing minimum criteria for the addition of hydro power facilities at existing projects, like location of the power house, switchyard, highway, railroad access and other site features. Location of the powerhouse is determined by the overall project development factors like location of spillway navigation locks and accessibility. For location of switchyard, considerations should be given to the number and direction of outgoing transmission lines, however the most desirable and economical location adjacent to the power house was suggested. Highway and railroad should be easily approachable for easy transportation and thus reducing the cost.

B. Ogayar [12] one of the most important elements on the recovery of a small hydro-power plant is the electromechanical equipment (turbine–alternator), since the cost of the equipment means a high percentage of the total budget of the plant. The present paper intends to develop a series of equations which determine its cost from basic parameters such as power and net head. These calculations are focused at a level of previous study, so it will be necessary to carry out the engineering project and request a budget to companies specialized on the construction of electro-mechanical equipment to know its cost more accurately. Although there is a great diversity in the typology of turbines and alternators, data from manufacturers which cover all the considered range have been used. The above equations have been developed for the most common of turbines: Pelton, Francis, Kaplan and semi Kaplan for a power range below 2 MW. The obtained equations have been validated with data from real installations which have been subject to analysis by engineering companies working on the assembly and design of small plants.

## 1.6 OBJECTIVES OF THE STUDY

Energy is becoming dearer day by day. The consumption is increasing and conventional sources are depleting. The small hydro power plants were considered costlier so far, especially in the low head range. It is therefore proposed to study various components of small hydropower scheme especially run-of-river scheme in order to establish cost effective designs. Under the present dissertation, the layout of low head run-of-river hydropower schemes has been analyzed to achieve the cost effective generation. Keeping this in view, various components run-of-river schemes have been studied, with respect to their costs and sensitive parameters in order to form cost effective planning.

## 1.7 SCOPE OF WORK

The civil works of small hydropower project constitutes a major portion of the total cost. In this respect, low head developments are inherently expensive, as the high flows need large machines. In this present work, it is proposed to study and compare various layouts with different types of turbines for installation of low head hydropower schemes. it is also proposed to develop software to facilitate optimum design for different head conditions.

The present study is planned with following scope of work

i. Study of various components of low head small hydro power schemes.
ii. To carry out sizing of various components under civil works and selection of electro-mechanical equipments.
iii. Development of software for determining the sizes of components.
iv. Analysis of cost of different components for developments of cost correlation in order to determine the total installation cost.
v. Selection of most cost contributing component
vi. Development of software for cost computation of power house building having different types of layouts.
vii. Hydraulic design of different types of spillway, generation of profile of spillways and cost calculation using computer software.

# CHAPTER 2

# SMALL HYDROPOWER SCHEMES AND THEIR COMPONENTS

## 2.1 LOW HEAD SMALL HYDRO POWER PLANTS

As discussed in the chapter 1, Small hydro power sites are classified into three categories based on head as low, medium or high. Each category requires different design criteria. Under a head range from 3m to 20 m, SHP schemes are considered as low head SHP schemes. A large number of sites identified irrigation works are in the low head category. Medium head sites are having head range from 20 to 60m, while above 60m head sites are considered as high head schemes. These limits are not rigid but are merely a means of categorizing the sites. Low head small hydro power (SHP) schemes up to 20m head can be run of river, dam based, however most of these sites are canal based schemes. [13]

## 2.2 COMPONENTS OF LOW HEAD SHP

A small hydro power plant mainly consists of two major components: A time diagram for SHP scheme is shown in figure 2.1.

1. Civil Works
2. Electrical and Mechanical Equipment

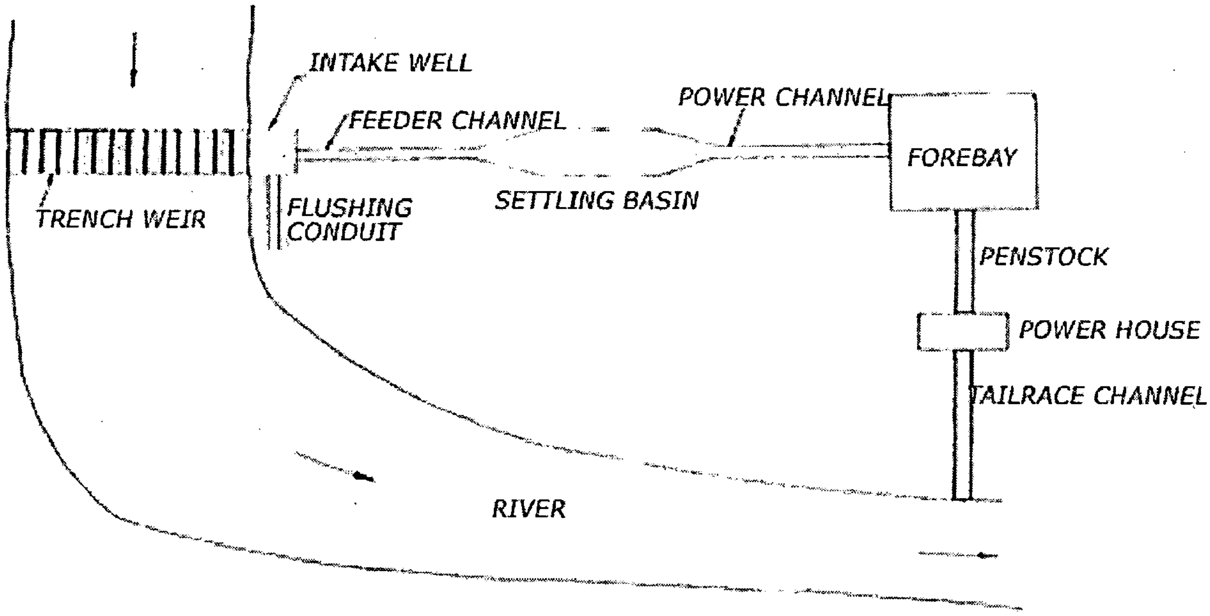



*Fig. 2.1 Schematic diagram of ROR Plant*

### 2.2.1 Civil Works [14]

The purpose of civil work components is to divert the water from the stream and convey towards the power house. In selecting the layout and types of civil components, due consideration is given to the requirement for reliability. The various civil components of run-of-river plant are shown in figure 2.2.

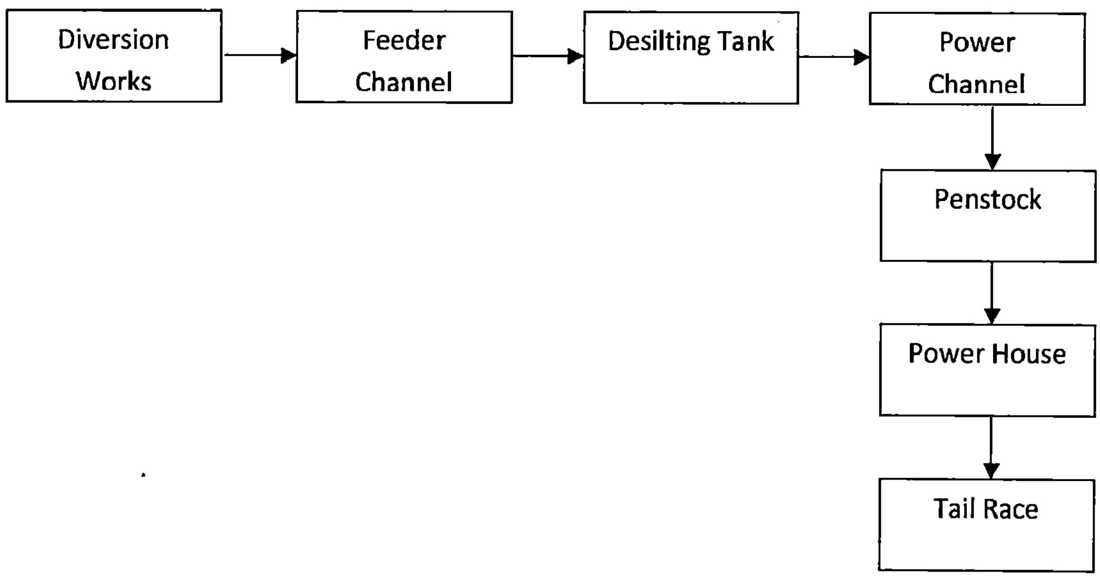



*Fig. 2.2 Block Diagram of Civil Components of ROR Plant*

#### 2.2.1.1 Diversion Structures

It's a structure built across a natural stream to divert the water towards the power house for power generation. It may be in the form of barrage or weir, may be gated or non-gated, and may be temporary or permanent. It is designed in such a manner that.

i) It should have a narrow, well defined section.
ii) Location should be such that discharge intensity is high.
iii) The desired amount of water should be diverted most of the time.
iv) The sediments in water should not be allowed to enter the water intake (as far as possible).
v) Accumulated objects should be easily flushed downstream.
vi) The flow velocity should be controlled to protect the structure from scouring.

#### 2.2.1.2 Intake works

Intake is a structure to facilitate the entry of water to the conduit system. It may or any not be submerged. For SHP, it should be of permanent nature. Usually intake includes trash-racks, a gate and an entrance to a canal. The intake is generally built of reinforced concrete, the trash-rack of steel, and the gate of steel.

**2.2.1.3 Feeder Channel and Power Channel:** The channel provided between intake and desilting tank is termed as feeder channel and channel provided between desilting tank and forebay tank is termed as power channel. The alignment of the channel is usually following the contours and cross drainage works should be avoided as far as possible as they result in head loss.

#### 2.2.1.4 Desilting basin

A system for protecting the turbine by preventing solid particles from entering the penstock is installed as part of the intake works near the intake or the forebay, depending on flow, terrain and the material from which the channel is constructed. The desilting basin is designed based on head and size of silt particles to be eliminated.

#### 2.2.1.5 Forebay

Forebay is provided at the end of water conductor system. Main function of the forebay is to provide immediate water demand on starting the generating units. It also provides enough depth water over the penstock to prevent vortex formation and air entry. The location of the forebay should be carefully chosen in rock/ soil strata. The structure should be leak proof so that the transition between open channels to pressure flow in a pipe can occur smoothly. The important components of the forebay are: (i) spillway, (ii) silt flushing, (iii) penstock intake and (iv) trash rack. In case of flow entering the forebay exceed the flow flowing through the penstock, or the valve to turbine is closed during heavy rains or excess flows enter the canal from stream or from runoff uphill of the canal, then the excess water is to be disposed off through spillway. Spillway is provided in forebay to become operative in case of sudden load rejection or at partial load on machines. An opening with its bottom at the maximum water level may be provided at a suitable location on the forebay and connected to a natural drain through spilling channel. Suitably spillway channel is provided to safety dispose off the excess inflows.

Incoming water may carry sizable quantities of floating debris, in such conditions forebay functions as final settling basin. Then debris is removed through a silt flushing pipe. A trash rack is provided at the inlet to the penstock to prevent floating material from entering the penstock and turbine. Drainage arrangement is also required when forebay is being repaired.

#### 2.2.1.6 Trashrack

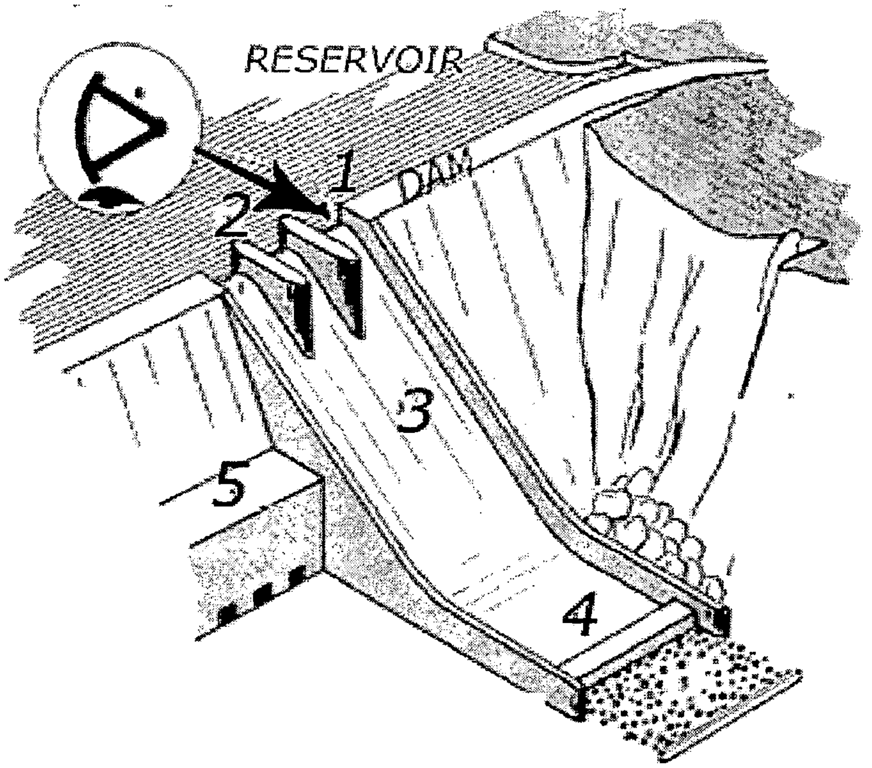
The purpose of trash rack is to remove debris from in front of the trash racks. Removing the trash will reduce head loss across the trash racks and improve the generating efficiency of the turbines. Trash racks may be divided into three types according to construction and installation requirements:

i) End bearing.
ii) Side bearing.
iii) Integral trashrack.

#### 2.2.1.7 Spillway

A *Spillway* channel is a concrete channel connecting from spillway crest to downstream river over which the water flows at a supercritical velocity. It is used to release the excess water in the forebay over the spillway crest in a safe manner.

Spillways can be provided with gates, to provide a better control on the discharges passing through. However, in remote areas, where access to the gates by personnel may not be possible during all times as during the rainy season or in the night un-gated spillways may have to be provided.
A typical layout of spillway is shown in figure 2.3.




*1.Spillway Crest* *2. Pier*
*3.Spillway Face* *4. Stilling Basin*
*5. Powerstation*

*Figure 2.3 Layout of Spillway*

### 2.2.1.8 Balancing Reservoir

At some sites, where lean season discharges are very low and not sufficient to run the turbine, it is preferred to have a balancing reservoir for storage of water for some hours in a day and utilized for power generation for some hours. It is called diurnal storage. The function of balancing reservoir is same as forebay. The main purpose of the balancing reservoir is to store water during off-peak hours to supply the same during peak hours and thus meet peak electricity demand.

### 2.2.1.9 Surge tank

Surge tank is a structure for compensating over-pressure; not often used in low head SHP, The surge tank is designed depending on the head and length of penstock, velocity of water in the penstock, materials of which penstock is made and time needed to close the main valve.

### 2.2.1.10 Energy Dissipater

The water flowing down from the spillway possess large amount of kinetic energy that is generated by virtue of its losing the potential head from the reservoir level to the level of the river on the downstream of the spillway. If this energy is not reduced, there is danger of scour to the riverbed which may threaten the stability of the dam or the neighboring river valley slopes. The various arrangements for suppressing or killing of the high energy water at the downstream toe of the spillways are called energy dissipaters.

### 2.2.1.11 Stilling Basins

The Stilling basin employs the hydraulic jump for energy dissipation and is the most effective method of dissipating energy in flow over spillways. The two basic parameters to be determined for design of a stilling basin are the apron elevation and length.
The effective energy dissipation can be attained with a stilling basin having either a horizontal or sloping apron. The use of a sloping or horizontal apron is based solely upon economics in order to provide the least costly basin.

A. Hydraulic Jump type Stilling Basin
    1. Horizontal apron type.
    2. Sloping apron type.

B. Jet Diffusion Stilling Basin
    1. Jet Diffusion Stilling Basin
    2. Interacting Jet Dissipaters

3. Free Jet Stilling Basin
4. Hump Stilling Basin
5. Impact Stilling Basin

C. Bucket type Energy Dissipaters

1. Solid roller bucket.
2. Slotted roller bucket.
3. Ski-jump (or flip or trajectory) bucket.

### 2.2.1.12 Desilting Tank

Desilting tank is provided where the water contains large quantities of coarse silt to minimize erosion damages to the turbine runner. The extent of desilting requirements depends on the quantum and type of silt carried by the stream and the runner material. Generally, hill streams carry appreciable quantity of silt and sand during rainy season. These are more harmful due to the fact that development of such streams is generally for high heads and abrasion efforts become more pronounced with increasing head. To trap the pebbles and other suspended matter desilting tank is generally provided in the initial reaches of the water conductor.

### 2.2.1.13 Penstock

The water is taken from the forebay to the power station through the penstocks. These may be pressure conduits or shafts. The penstocks carry water to the turbine with the least possible loss of head consistent with overall economy of the project. In a power station situated at the toe of the dam, penstock are taken through the dam monoliths, with the necessary transition for smooth hydraulic flow and control devices at the intake structure

### 2.2.1.14 Power House

Powerhouse is a building that houses the turbine generator and the control units. The size of the power house depends on the dimensions of the turbine, generator and other equipments and auxiliaries. The width and centre to centre distance between the units will vary according to the size of the runner.

The equipment that is kept in the powerhouse includes:

- Inlet gate or valve
- Turbine
- Generator
- Speed increaser(if necessary)
- Control System
- Protection Systems
- Fire Protection
- Cooling Water System

**2.2.1.15 Draft Tube**

The water after doing work on the turbine moves towards the tailrace through a draft tube, which is a concrete tunnel or a riveted steel plate pipe, its cross section gradually increases towards the outlet. The draft tube is a conduit, which connects the runner exit to the tailrace. The tube should be drowned approximately one meter below the lowest tailrace level.

The following are the functions of the Draft Tube:

i) The water is discharged freely from the runner; turbine will work under a head equal to the height of the headrace water level above the runner outlet. An airtight draft tube connects the runner to the tailrace; workable head is increased by an amount equal to the height of runner outlet above tailrace.

ii) The draft tube will thus, permit a negative suction head to be established at the runner outlet thus making it possible to install the turbine above the tail race without loss of head.

iii) The water leaving the runner still possesses a high velocity and this kinetic energy would be lost if it is discharged freely as in a Pelton turbine. With the increase in net working head on the turbine, output will also increases, thus raising the efficiency of turbine.

The following are some types of draft tubes used in the Small Hydropower Station

(a) Straight Divergent Tube

(b) Moody Spreading Tube

(c) Simple Elbow Tube

(d) Elbow Type with a Circular Inlet and a Rectangular outlet section

The Figure 2.4 shows different types of draft tubes.



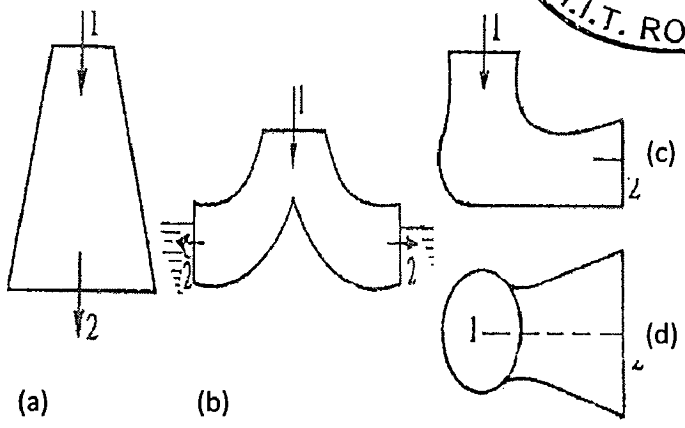



*Figure 2.4 Different Types of Draft Tube (a) Straight Divergent Tube (b) Moody Spreading Tube (c) Simple Elbow Type (d) Elbow Type with a Circular Inlet and a Rectangular Outlet Section [7]*

#### 2.2.1.16 Tailrace

The main function of the tailrace is to maintain minimum tail water elevation below the power plant and to keep the draft tube submerged. All reaction turbines require that the tail water be maintained at a minimum elevation to minimize the effects of cavitations and take the advantage of additional head available when the turbine setting is above the tail water elevation.

## 2.3 ELECTRICAL AND MECHANICAL EQUIPMENT

Electro-Mechanical equipments mainly include hydro turbine, generator, speed generator, governor, gates and valves and other auxiliaries.

### 2.3.1 Hydraulic Turbines

The hydraulic turbines are classified mainly on the basis of process of conversion of hydraulic energy into mechanical one. Turbines are classified in two general categories:

*(a)* Impulse turbines

*(b)* Reaction turbines

### (a) Impulse Turbines

If at the inlet of the turbine, the energy available is only kinetic energy, the turbine is known as impulse turbine. In an impulse turbine, all the available potential energy or head is converted into kinetic energy or velocity head by passing it through a contracting nozzle or by guide vanes before it strikes the buckets of the turbine. The characteristic features of an impulse turbine are:

a) The wheel passages are not completely filled.
b) The water acting on the wheel vanes is under atmospheric pressure.
c) The water is supplied at a few points at the periphery of the wheel (usually one point, but occasionally two or more points).
d) Energy applied to the wheel is wholly kinetic.

### (b) Reaction Turbines

The main distinguishing feature of a Reaction turbine is that only a part from the total energy converted into the kinetic energy before the runner is reached and the working fluid completely fills the passages in the runner. Thus the pressure of the flowing water changes gradually as it passes through the runner and therefore the runner must be enclosed with in a watertight casing. Such turbines may also be called pressure turbines.

The water after doing work on the runner leaves it through the draft tube and joins the tail race. The turbine space of a reaction turbine is formed by the following hydraulic elements: spiral casing, stay ring, guide wheel, runner and the draft tube.

The characteristic features of a reaction turbine are:

*a)* The wheel passages are completely filled with water.
*b)* The water action on the wheel vanes is under pressure greater than atmospheric.
*c)* The water enters all around the periphery of the wheel.
*d)* Energy in the form of both pressure and kinetic is utilized by the wheel.

The reaction turbines are, in turn, sub-divided into several types depending on the direction of flow within the runner with respect to the turbine axis.

1) *Radial flow turbine:* In a radial flow turbine, the water flows in a radial direction. The turbine may be radially inward flow type or radially outward flow type depending upon whether the flow is from the outer periphery to the inner one or vice versa.
2) *Axial flow turbines:* The flow within an axial flow turbine is parallel to the turbine axis.
3) *Mixed flow turbines:* In mixed flow turbines the flow changes its direction from radial to axial within the runner.

### 2.3.2 Turbine selection Graph [15]:

The gross head is the vertical distance, between the water surface level at the intake and at the tailrace for reaction turbines and the nozzle level for impulse turbines. The selection is particularly critical in low-head schemes, where to be profitable large discharges must be handled. When contemplating schemes with a head between 2 and 5 m, and a discharge between 10 and 100 m3/sec, runners with 1.6 to 3.2 meters diameter are required.

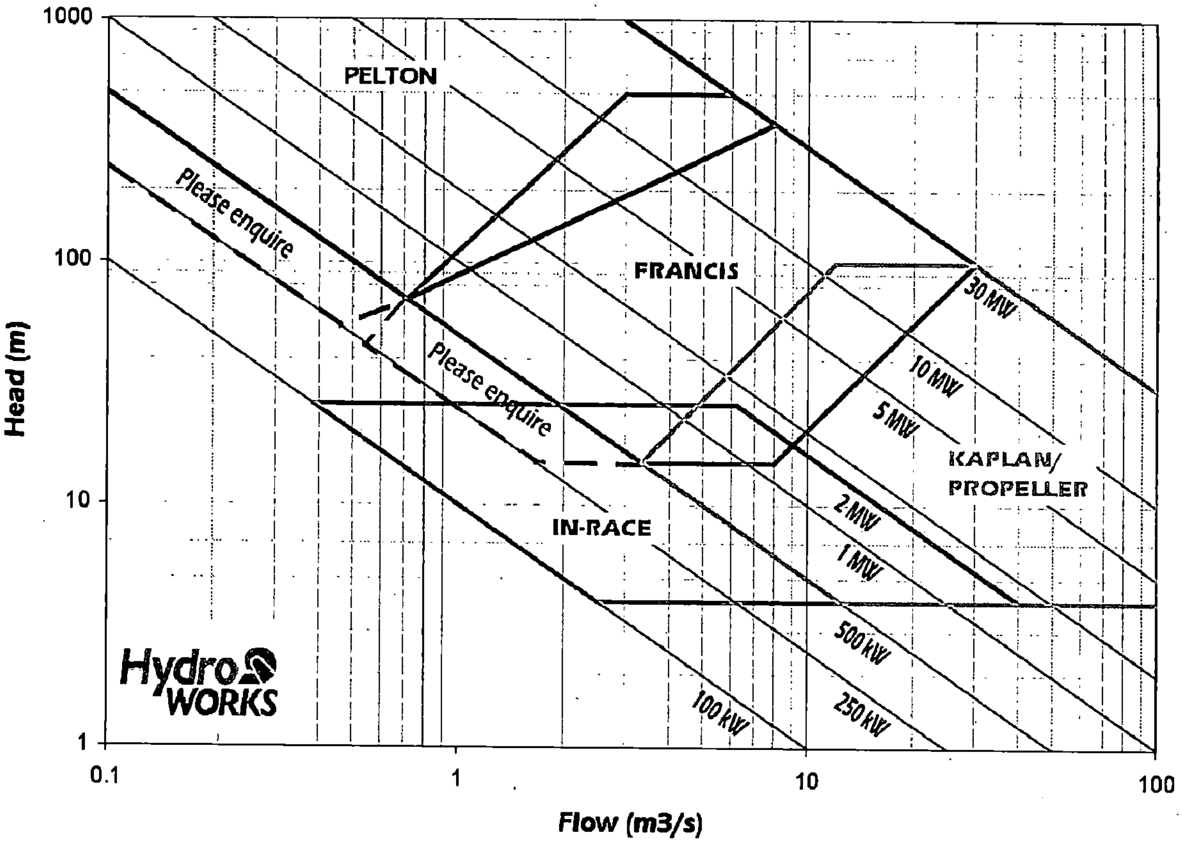



*Figure 2.5. Turbine selection graph*

### 2.3.3 Classification of turbine based on Head [16]

Head acting on the turbine is one of the most important criteria dictating type of turbine to be used for power station under construction. The turbine for a particular project is selected based on the head and discharge of that project. Based on head and discharge the selection of turbine is generally done as follows:

i. High head & low discharge - Pelton
ii. Medium head & medium discharge - Francis
iii. Low head & high discharge - kaplan

Based on above information we cannot do the demarcation of a particular type of head. But with the experience over the year, the industry has crystallized the main types of turbine for the use of different head range, which are shown in Table 2.1 below.

*Table 2.1 Head ranges for different turbines*

| Type of Turbine | Class of head | Head range for larger medium sets(m) | Head range for small sets(m) |
|---|---|---|---|
| Pelton | High head | Above 300m | Above 150m |
| Turgo impulse | - | - | 40 to 200m |
| Francis | Medium head | 30to500m | 20to200m |
| Kaplan(axial flow) | Low head | 3 to 50m | 3to20m |
| Some special type of axial flow e.g. tubular, bulb, s-type | Low head | 3to40m | 3to25m |

### 2.3.4 Classification Based On Specific Speed

The specific speed forms the common basis for selection of a turbine. Higher specific speed of turbine results in higher speed of turbine in higher specific speed of turbine results in higher speed of rotation for generator with consequent reduction in cost of generator. This criterion is very important for dictating type of turbine from cost consideration in the overlapping head range, specific speed is the speed in rpm of geometrically similar turbine that would develop one kW power working under a head of one meter. Mathematically,

$$Ns = \frac{N\sqrt{P}}{H^{1.25}}$$

Where, Ns = specific speed of turbine

N = rotational speed of turbine in rpm

P = Turbine output in metric horse power

H= head in meter.

Specific speed is a very important parameter for selection of turbine for a particular site because it involves the basic parameters such as rated head, speed, and power. The ranges of specific speed for different types of turbines are given in Table 2.2 as below.

*Table 2.2: Specific Speed of turbines*

| Sr. no. | Types of turbine | Specific speed(Ns) (Metric system) |
|---|---|---|
| 1 | Pelton(with single jet) | 10-35 |
| 2 | Pelton(two or more jet) | 35-60 |
| 3 | Turgo Impulse | 20-70 |
| 4 | Cross flow | 20-100 |
| 5 | Francis | 80-400 |
| 6 | Axial Flow | 340-1000 |

## 2.4 TURBINES FOR LOW HEAD SHP [15]

For low head small hydroelectric plants, in which the head is up to 20m with high discharge, generally a reaction turbine is used.

### 2.4.1 Vertical Francis Turbine

Flow into Francis turbine is normally conveyed through a penstock. An area must be available downstream from the impoundment to accommodate the larger site requirements of a Francis turbine. This type of turbine may be used either in an indoor or outdoor plant, depending on site conditions.

Francis turbines are available for operation at heads of 5 m and above. The hydraulic characteristics of a Francis runner are such that operating speeds are lower than comparable propeller runners, hence may not be as cost competitive due to increased physical size of the turbine and higher generator costs. The turbine shaft is connected to the generator by a flanged coupling driving the generator at the turbine speed or through a speed increaser permitting use of the higher speed lower cost generator. A Francis turbine has an efficiency curve with a slope which falls between the sixth blade propeller and the Kaplan turbine. Vertical Francis turbine is as shown in figure 2.6.

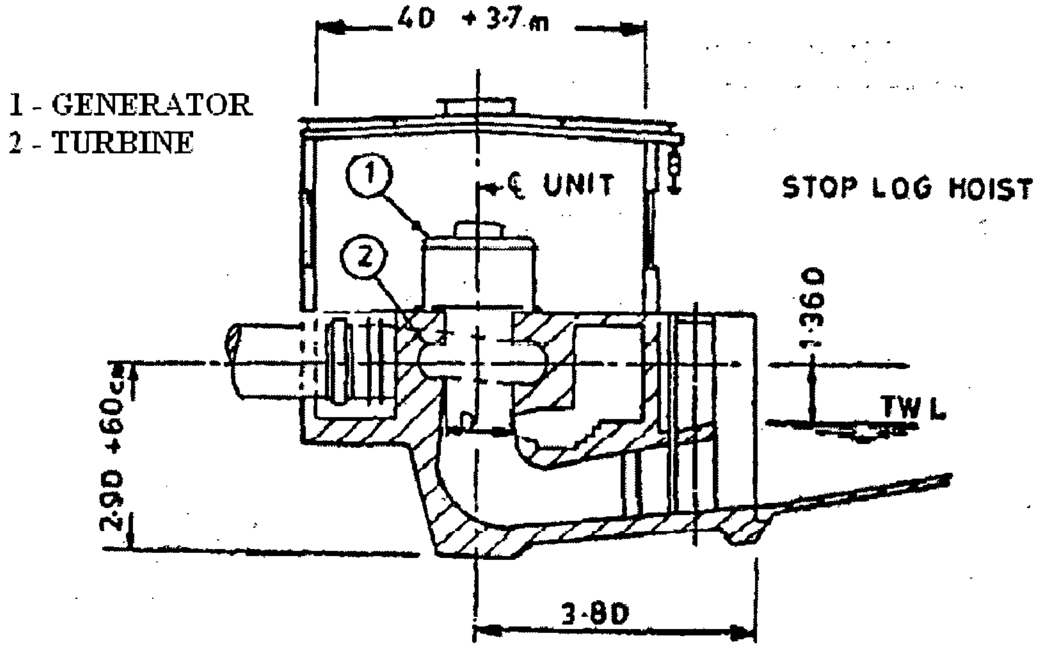



*Figure 2.6 Vertical Francis Turbine*

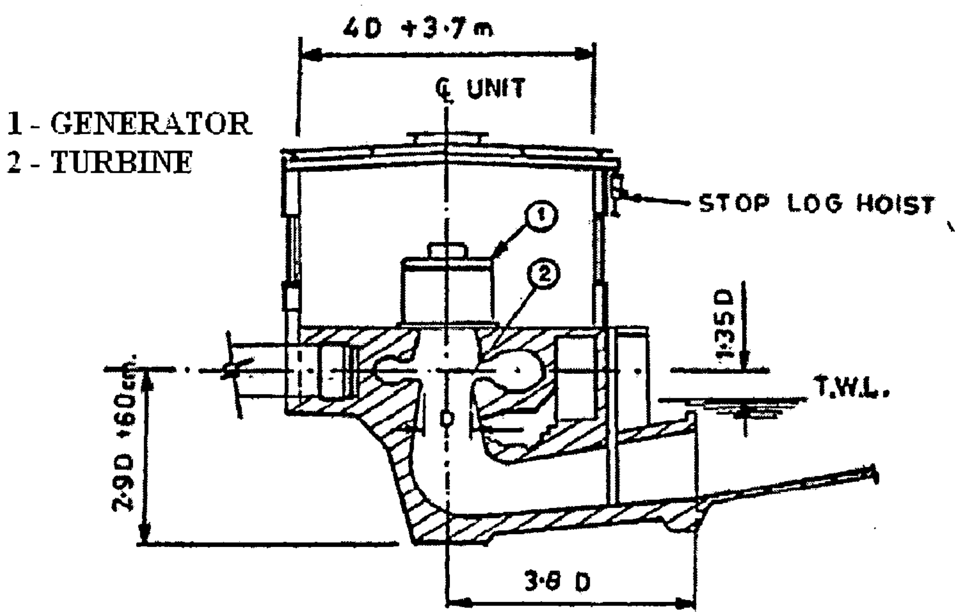



*Figure 2.7 Vertical Propeller Turbine*

### 2.4.2 Horizontal Francis Turbine

A Francis turbine may be designed for horizontal mounting. The turbine shaft is parallel with the power house flows and coupled directly or through a speed increaser to a horizontal generator. The cost of the generator is less than that of a vertical generator due to the smaller size and reduced thrust bearing requirement. However, the horizontal mounting will require greater flow speed than a vertical unit, but less vertical height.

For very small turbine, those having throat diameter less than 1.2m, there may be cost advantage in using a Francis type with a horizontal shaft. The arrangement of penstock, discharge and generator may be simpler than those for a vertical shaft unit. Horizontal Francis turbine as shown in fig 2.8

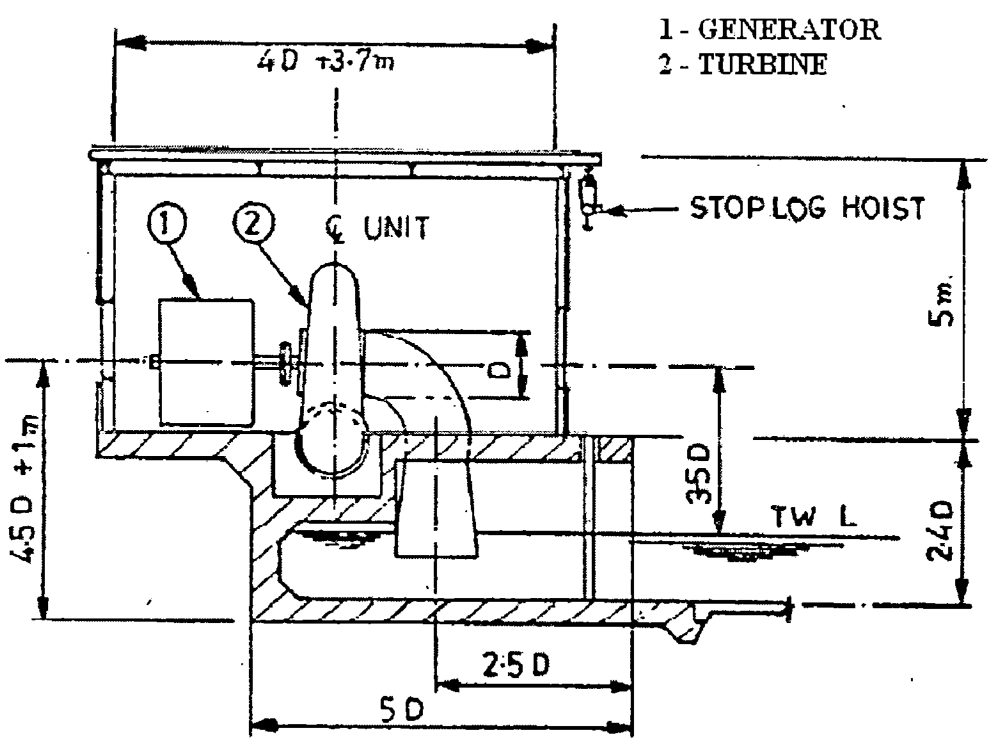



*Figure 2.8 Horizontal Francis Turbine*

### 2.4.3 Open Flume Configuration – Francis or Propeller

Either a Francis or a Propeller turbine may be used in a flume or canal at an existing drop or vertical discontinuity in the flume or canal. The design consists of a wicket gate and stay ring assembly mounted vertically in a non-pressurized flume. The turbine guide bearing and wicket gate mechanism are submerged and generally water lubricated which increases maintenance costs. This type of turbine is limited to an operating head of 10m. When this type of turbine is suitable, the equipment and civil costs are lower than the other types.

This configuration may be used for either an indoor or outdoor type of plant, depending on the site condition. Penstocks are not used with this type of configuration. Layout of Open Flume Configuration – Francis or Propeller turbine is as shown in fig. 2.9.

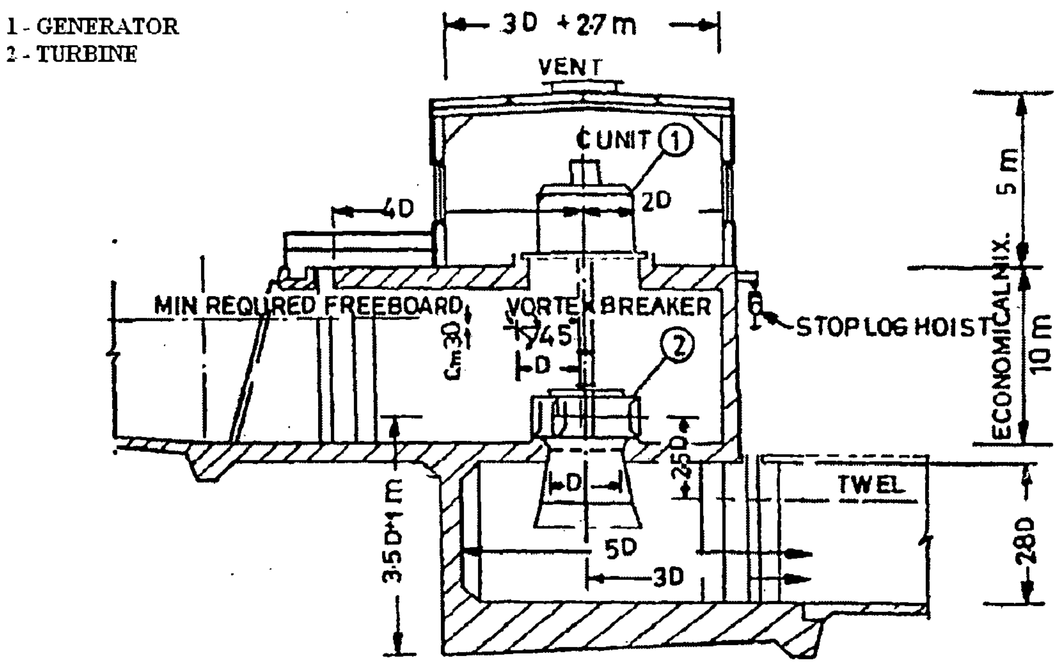



*Figure 2.9 Open Flume Turbine*

### 2.4.4 Tubular Turbine

Tubular turbines are horizontal or slant mounted turbines with fixed or adjustable blade propeller runners. The generators are either directly coupled to the turbine shaft or connected through a speed increaser. The generators are located outside the water passage way, which result in larger floor space requirement than the vertical or Bulb type units. These higher civil costs are offset by reduced height requirements for the building and lower turbine and generator cost. Slant mounting of the tubular turbine reduces the floor space requirement; however, it adds cost for the turbine and generator due to higher thrust and longer shafts.

A Tubular turbine may be efficiently located to become part of the existing outlet works and/or to be adjacent to the existing impoundment. This type is easily adapted to a canal installation. Normally, the generator will be housed within a building. However, it is feasible to have the major erection or overhaul areas outdoors. Layout of Tubular Turbine as shown in figure 2.10.

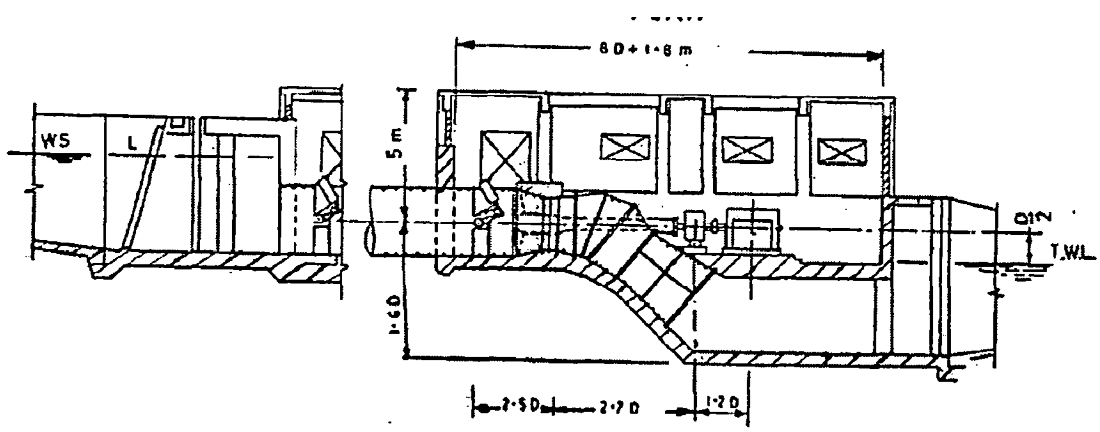



*Figure 2.10 Tubular Turbine*

### 2.4.5 Bulb and Rim Type

Bulb turbines horizontal units, which have wicket gates and fixed or adjustable blade propeller runner directly connected to the generator as shown in figure 2.11. The generator is enclosed in water tight structure (bulb) located in the water passage ways. This design permits a compact power house structure with minimum floor space and height. The straight flow water passage way also minimizes head loss. This reduction in space requirement is offset by increased turbine and generator costs due to water tight requirement.

Rim type turbine has the generator and rotor mounted on the periphery of the turbine runner blade. The rim turbine offers a potential saving in power house construction cost due to its compact design.

The possible configurations for either the bulb or rim turbine are similar to those that are appropriate for the tubular turbine. As the turbine and generator for the bulb type are in water passage, the enclosed structures above the units are relatively small, unless the erection and maintenance area are enclosed. Normally, for units less than 5 MW capacity, these types are not as economically, as the tubular type, despite the smaller power house.

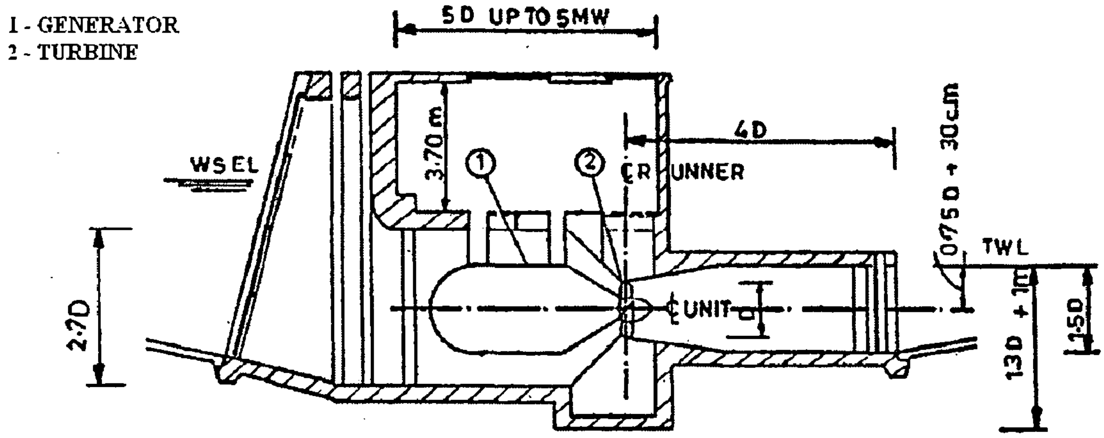



*Figure 2.11 Bulb Turbine*

## 2.5 GENERATORS

Generators transform mechanical energy into electrical energy. Depending on the characteristics of the network supplied, generators are classified as:

### (A) Synchronous Generators

They are equipped with a DC electric or permanent magnet excitation system associated with a voltage regulator to control the output voltage before the generator is connected to the grid. They supply the reactive energy required by the power system when the generator is connected to the grid. Synchronous generators can run isolated from the grid and produce power since excitation is not grid-dependent. The synchronous speed is given as:

$$\text{Synchronous speed } N_s = 120 \times f / p \qquad (1.3)$$

Where, Ns is speed in r.p.m,

f is the frequency in Hz and

p is the no of poles.

**(B) Asynchronous Generators**

They are simple squirrel-cage induction motors with no possibility of voltage regulation and running at a speed directly related to system frequency. They draw their excitation current from the grid, absorbing reactive energy by their own magnetism. Adding a bank of capacitors can compensate for the absorbed reactive energy. They cannot generate when disconnected from the grid because are incapable of providing their own excitation current. However, they are used in very small stand-alone applications as a cheap solution when the required quality of the electricity supply is not very high.

The induction machine operates as a motor when running below synchronous speed and as a generator when the rotor is above synchronous speed. Therefore any induction motor may also be used as a generator, simply by driving at above synchronous speed. The difference between the synchronous speed $n_s$ and the rotor speed $n_r$ is called slip speed $n_g$ , represents the speed of the rotating field viewed from the rotor.

$$\text{Slip speed } n_g = n_s - n_r \qquad (1.4)$$

$$\text{Slip } S = (n_s - n_r) / n_s \qquad (1.5)$$

If Slip S is negative, i.e. rotor speed is more than synchronous speed, the machine operates as generator. If Slip S is positive, i.e. rotor speed is less than synchronous speed, the machine operates as motor.

## 2.6 CONDITIONS FOR DETERMINING PROJECT LAYOUT

The following considerations are necessary when developing a project development plan and determining project layout [17]:

- Flow and flood data
- Methods of operation
- Project configuration, dimensions, and governing elevations
- Construction plan and requirement for diversion and care of water
- Elevations of headwater and tail water (minimum, normal, maximum)
- Power house and turbine type and number of units
- O & M requirement (powerhouse crane, mobile crane, roof hatches, etc.)
- Need for capacity of intake and penstock
- Need for bypass works, means of purging sediment and bed load deposited in front of intake
- Method for connecting to existing facilities
- Need for remedial or repair work
- Need for special works (fish facilities, etc.)
- Transmission line requirements
- Needs for project access
- Sensitivity to environmental and archaeological issues

## 2.7 LOCATION OF POWER HOUSE [18]

With consideration to site, load, etc. characteristics, different layouts are used for each individual development, and different structures are built or omitted accordingly. The arrangement of the individual structures may differ from shown in the above figure 2.12. The power canal itself may be designed in several different ways according to the purposes of diversion.

According to the position of power station with respect to the canal, the different types of diversion canal developments can be classified broadly into three groups as shown in the figure 2.12.

1. Layout with divided power canal, the canal consists of headrace and a tailrace.

2. Layout with the power station at the canal entrance; both channel intake and headrace are omitted.
3. Layout with the power station at the lower end of the canal; no tailrace.

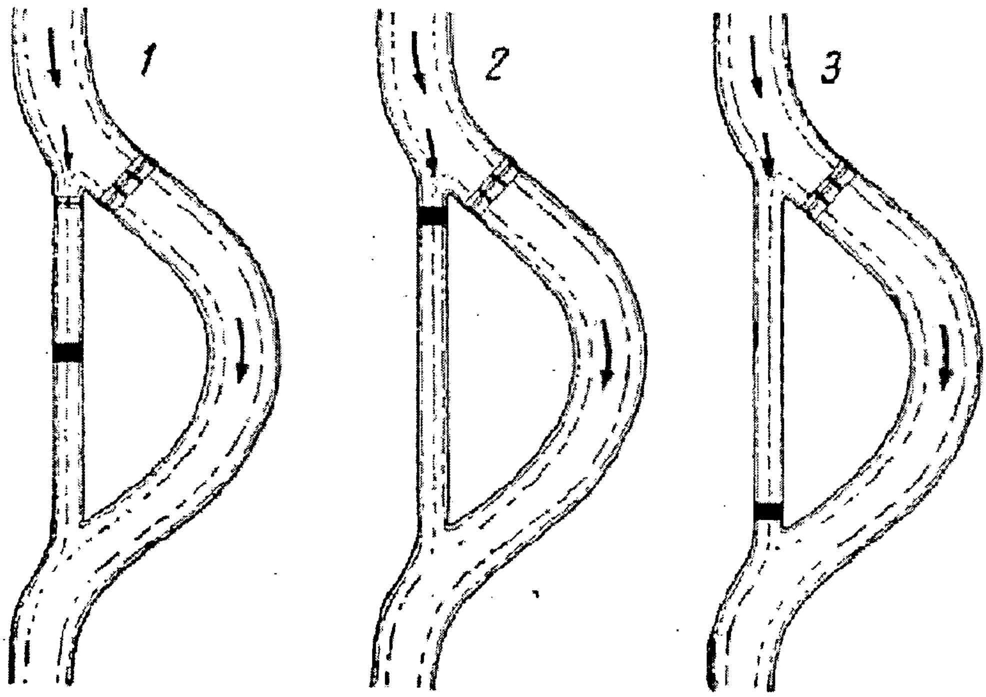



*Figure 2.12 Location of the Power Station in the Canal*

The location of the power station should be determined mainly by economic considerations. Costs of the project in turn will be affected by site conditions, that is in the last analysis geological, topographical and particularly the hydro-geological condition of the power site will be decisive in finding out which of the above layouts will prove most advantageous.

## 2.8 SITE SELECTION AND BASIC LAYOUT [19]

Site selection is conditioned by the existence of both head and flow requirements, which are necessary for hydro power generation. Before a site is selected, a preliminary assessment study should include definition of power potential, estimation of power output, identification of physical works needed, identification of critical issues, and a preliminary study of economic feasibility.

Depending upon the dimension of structures relative to the width of the river bed, two main types can be discerned.

A. The original river bed is sufficiently wide to accommodate the power station as illustrated in figure 2.13. The commonest arrangement is the so called block power plant shown as the first alternative. The second alternative represents the twin power plant. Individual machine units of the third type are incorporated into the piers of the weir. This arrangement will be referred to also as the divided power station. An overfall type weir houses, the generating units and ancillary equipment according to

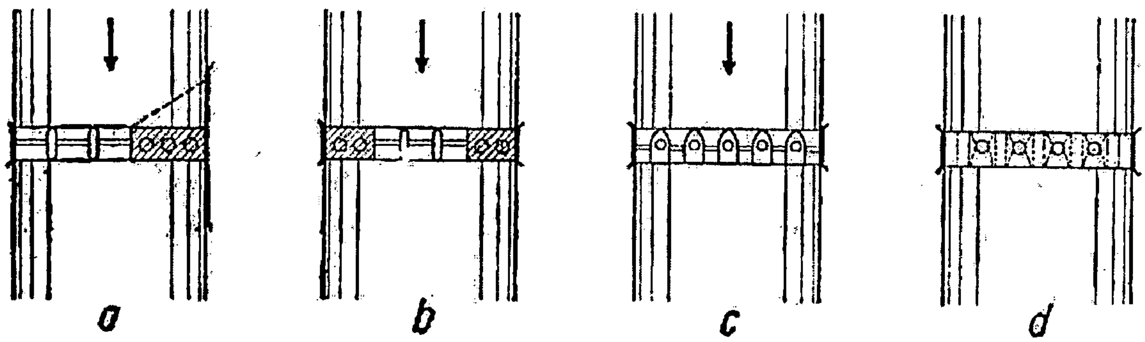
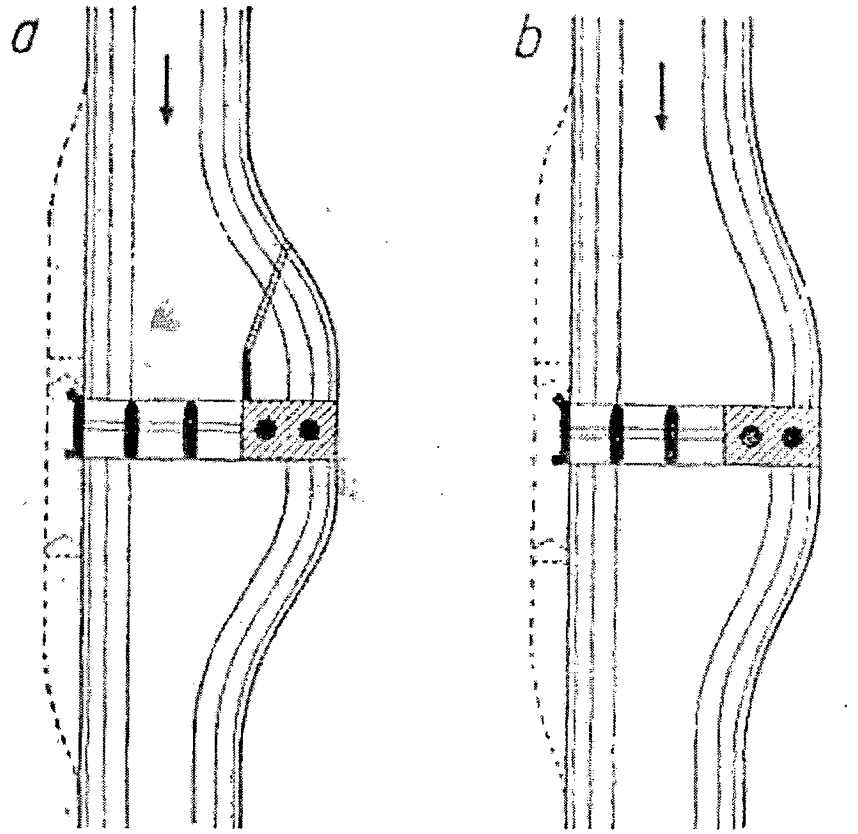



*Figure 2.13 Run-Of-River Power Stations Without Bed Enlargement. (a) Block Power Plant (b) Twin Power Plant (c) Pier Head Layout (d) Submergible Power Station*

the fourth alternative which is known as the submersible power station. The installation of these types requires favorable flood, bed load and ice conditions, permitting the inevitable contraction of the flow.

B. According to European practice, a bay is usually excavated to accommodate the Block Power plant. The widening of the bed will in most cases become necessary for twin power plants as well. Two main types can here again be discerned, those with an intake work at the entrance to the bay, and those without as shown in the figure 2.14.




*(a) With Intake Work* *(b) Without Intake Work*

*Figure 2.14 Block Type Run-Of-River Power Station with Bed Enlargement*

# CHAPTER 3

# ANALYSIS FOR SIZING OF SHP PROJECT COMPONENTS

## 3.1 COMPONENTS OF LOW HEAD SMALL HYDRO POWER PLANT

The components of a typical small hydropower are divided mainly in
(1) Civil work (2) Electromechanical equipment

The components under civil work are given as below.[20]

1. Diversion
2. Intake
3. Desilting Tanks
4. Power Channel
5. Spillway
6. Forebay Tank
7. Powerhouse
8. Penstock
9. Tail race

The major components under electro-mechanical equipment are turbine and generator. In this chapter the sizing of components has been carried out in order to compute the cost of the project based on the cost sensitive parameters.

## 3.2 FORMULATION OF HYDRO POWER POTENTIAL

The physical sizes of the civil work are hydropower scheme depends on the discharge. For a given capacity of head, discharge is calculated by the equation (3.1) given as below.

$$P = 9.81 \times Q \times H \times \eta o \quad kW \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.1)$$

Where, Q is the discharge in $m^3/s$,

H is head in m,

$\eta o$ is the overall efficiency of the turbine, generator and gear-box.

## 3.3 POWER CHANNEL

The size of channel i.e. bed width (b) and depth (h) is determined by using Manning's equation. The channel section has been worked out as hydraulically efficient section and determined as given below.

Manning's equation,

$$V = \frac{1}{n} R^{2/3} S^{1/2} \qquad \text{.......................} (3.3)$$

Where:

V is the cross-sectional average velocity ( m/s)
N is the Manning coefficient
R is the hydraulic radius (ft, m)
S is the slope of the water surface or the linear hydraulic head loss, m

$$Q = A * V \qquad \text{.........................} (3.2)$$

Where, A= Cross sectional area of the channel section.

$A = y * (b = s * y)$

Wetted perimeter, $P = b + 2y\sqrt{1+s^2}$

Channel cross sections are designed based on the design discharge, for different head and discharge. Typical cross section is shown in figure.

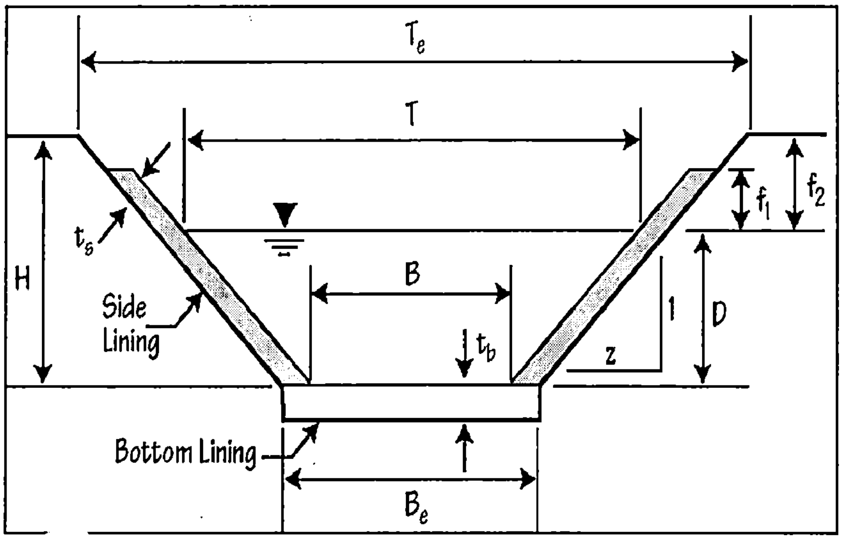



*Figure 3.1. Power Channel layout*

## 3.4 SPILLWAY [21]

The dam or weir may be constructed to that height which is permissible within the given topography of the location or limited by the expenditure that may be possible for investment. The excess flood water, therefore, has to be removed from the reservoir for investment. The excess flood water, therefore, has to be removed from the reservoir to safely pass this excess of the river during flood flows are called spillways.

The excess water is drawn from the top of the reservoir created by the dam and conveyed through an artificially created waterway back to the river. In some cases, the water may be diverted to an adjacent river valley. In addition to providing sufficient capacity, the spillway must be hydraulically adequate and structurally safe and must be located in such a way that the out-falling discharges back into the river do not erode or undermine the downstream toe of the dam. The surface of the spillway should also be such that it is able to withstand erosion or scouring due to the very velocities generated during the passage of a flood through the spillway.

## 3.5 POWER HOUSE BUILDING

Power House is a simple structure housing the generating unit, auxiliary equipment for machinery and suitable outlets for tail-water discharge. In medium and high head schemes, the units are generally horizontal pelton or horizontal pelton or horizontal Francis type which requires simple foundation fixtures arrangement and trench for tail race. In remote areas, it is desirable to utilize the locally available material for power house construction to avoid costly carriage of cement and steel for concrete structure. However basic requirement for foundation for valves, turbine and generator could be of reinforced concrete (RCC) A RCC frame structure has been considered for powerhouse building. The layout of power house building at different head and capacity by using following expression.

$$D = \frac{84.6\text{Ø}^3\sqrt{H}}{N} \qquad \text{......................} (3.4)$$

$$\text{Ø}^3 = 0.0223(Ns)^{2/3} \qquad \text{......................} (3.5)$$

$$Ns = \frac{N\sqrt{Pu \times 1.358}}{H^{5/4}} \qquad \text{......................} (3.6)$$

Where,

- $\text{Ø}^3$ is the velocity ratio at discharge diameter of runner
- Ns is the specific speed of turbine
- N is the rotational speed
- H is the rated speed, RPM
- Pu is the rated unit output power at full gate opening, kW

### 3.6 PENSTOCK

In low head scheme, lengths of penstock are short, thus penstock for each machine is considered to reduce loss on account of branching of pipe near the power house. The velocity of water in the penstock (V) is determined by using the following expression to determine the diameter of penstock.

$$V = 0.125\sqrt{2gH} \qquad \text{......................} (3.7)$$

### 3.7 INTAKE

Intake is provided on the upstream side of the dam to facilitate flow in the penstock. The intake consists of trash rack to prevent floating material and gates for regulation of water flow. The intake opening is designed as a bell mouth to minimize the head loss. The height of intake is fixed by considering size of penstock and minimum draw down water level in the reservoir to avoid air entrapment in the penstock. The bell mouth portion of intake is considered in RCC with trash rack and gates as fabricated steel structures. In order to keep head losses as minimum in the trash rack, the velocity of water through of water through the trash rack is kept as 1 m/s and the size of trash rack is worked out for 50% clogging.

### 3.8 TAIL RACE

The main function of a tail race is to maintain minimum tail water elevation below the water elevation the power plant and to keep the draft tube submerged. All reaction turbines require that the tail water be maintained at a minimum elevation to minimize the effects of cavitations and take the advantage of additional head available when the turbine setting is above the tail water elevation.

### 3.9 PROGRAM FOR CALCULATION OF DIMENSIONS OF CIVIL COMPONETS

Design parameters of civil structures are determined using the C program developed in this chapter as given below. The computer program has been developed using hydropower design equations. The dimensions of civil works components are computed using input parameter as discharge and head of the site.

```
/*
**   PROGRAM FOR CALCULATION OF DIMENSIONS OF CIVIL COMPONETS
*/

# include <stdio.h>
# include <conio.h>
# include <math.h>

double head,        //Net Head
           Q1,      //Discharge
           Q2;      //Design Discharge

//Calculations for desilting_tank
void desilting_tank()
{
```

```
    double    Vf,       //Flow Velocity
              w,        //Width
              d,        //Depth
              Vs,       //Settling Velocity
              Vl;       //Settling Length

    printf("\n\n\t-----------------------------------------------------\n");
    printf("\tDESGIN OF DESILTING TANK");
    printf("\n\t-----------------------------------------------------\n");

    Q2 = Q1 + 0.10*Q1;

    printf("\n\tDesign Discharge\t\t=\t %.3lf cumec",Q2);

    printf("\n\n\tFlow Velocity  (m/sec)\t\t=\t ");
    scanf("%lf",&Vf);

    printf("\n\tWidth (m)\t\t\t=\t ");
    scanf("%lf",&w);

    d = Q2/( w*Vf );

    printf("\n\tDepth Reqiured\t\t\t=\t %.3lf m",d);

    printf("\n\n\tSettling Velocity (m/sec)\t=\t ");
    scanf("%lf",&Vs);

    Vl = (Vf/Vs) * d;

    printf("\n\tSettling Length\t\t\t=\t %.3lf m",Vl);

    printf("\n\n\t-----------------------------------------------------\n")
}

//Calculations for forebay_tank
 void forebay_tank()
 {

    double  storage,    //Storage required
            capacity;   //Required Capacity of tank

    printf("\n\n\t-----------------------------------------------------\n");
    printf("\tDESGIN OF FOREBAY TANK");
    printf("\n\t-----------------------------------------------------\n");
```

```
    printf("\n\tDesign Discharge  \t\t=\t %.3lf cumec",Q2);

    printf("\n\n\tStorage required  (min)\t\t=\t ");
    scanf("%lf",&storage);

    storage = storage * 60;

    capacity = Q2 * storage;

    printf("\n\tRequired Capacity of Tank\t=\t %.3lf m3",capacity);
    printf("\n\n\t-------------------------------------------------------\n");

}

//Calculations for Spillway
void spillway_tank()
{
    double    C,      //Coefficient
              L,      //Length of Spillway
              H;      //Head over the crest of spillway

    printf("\n\n\t-------------------------------------------------------\n");
    printf("\tDESGIN OF SPILLWAY TANK");
    printf("\n\t-------------------------------------------------------\n");

    printf("\n\tDesign Discharge  \t\t=\t %.3lf cumec",Q2);

    printf("\n\n\tCoefficient\t\t\t=\t ");
    scanf("%lf",&C);

    printf("\n\tHead over the crest of spillway (m)=\t ");
    scanf("%lf",&H);

    L = Q2/( C*(pow(H,1.5)) );

    printf("\n\tLength of Spillway\t\t=\t %.3lf m3",L);
    printf("\n\n\t-------------------------------------------------------\n");

}
  //Calculations for Penstock
void penstock()
{

    double    g,      //
```

```
            D,      //Economical Diameter
            H,      //Head over the crest of spillway
            n,      //
            hf;     //Head loss in Penstock

    printf("\n\n\t-----------------------------------------------------\n");
    printf("\tDESGIN OF PENSTOCK");
    printf("\n\t-----------------------------------------------------\n");

    printf("\n\tDesign Discharge  \t\t=\t %.3lf cumec",Q2);

    printf("\n\n\tGravitational Acc. Constant (m)\t=\t ");
    scanf("%lf",&g);

    D = 3.55 * ( pow( (pow(Q2,2)/(2*g*head)),0.25) );

    printf("\n\tEconomical Diameter of Penstock\t=\t %.3lf m",D);

    n=1;
    hf = (10.29*pow(Q2,2)*pow(n,2))/pow(D,5.333);
    printf("\n\n\t-----------------------------------------------------\n");

}

void Calculations()
{

    printf("\n\n\tEnter Net Head (m)\t\t=\t ");
    scanf("%lf",&head);

    printf("\n\tEnter Discharge (cumec)\t\t=\t ");
    scanf("%lf",&Q1);

    desilting_tank();

    forebay_tank();

    spillway_tank();

    penstock();

}

int main()
{
  //Take inputs and calculate required values
  Calculations();

  getch();
  return(0)
}
```

### 3.10 OUTPUT

The above software has been used to develop sizes of various components of run of river SHP scheme with following input data.

#### 3.10.1 Input data

| | | |
|---|---|---|
| Enter Net Head (m) | = | 10 |
| Enter Discharge (cumec) | = | 1.5 |

#### 3.10.2 Results

| DESGIN OF DESILTING TANK | | |
|---|---|---|
| Design Discharge (cumec) | = | 1.650 |
| Flow Velocity (m/sec) | = | 0.22 |
| Width (m) | = | 1.5 |
| Depth Required (m) | = | 5.000 |
| Settling Velocity (m/sec) | = | 0.0275 |
| Settling Length (m) | = | 40.000 |

| DESGIN OF FOREBAY TANK | | |
|---|---|---|
| Design Discharge (cumec) | = | 1.650 |
| Storage required (min) | = | 2 |
| Required Capacity of Tank ($m^3$) | = | 198.000 |

| DESGIN OF SPILLWAY | | |
|---|---|---|
| Design Discharge (cumec) | = | 1.650 |
| Discharge Coefficient | = | 1.7 |
| Head over the crest of spillway (m) | = | 0.15 |
| Length of Spillway ($m^3$) | = | 16.707 |

| DESGIN OF PENSTOCK | | |
|---|---|---|
| Design Discharge (cumec) | = | 0.154 |
| Gravitational Acc. Constant (m) | = | 9.81 |
| Economical Diameter of Penstock (m) | = | 1.218 |

# CHAPTER 4
# COST ANALYSIS

The main obstacle in the development of small hydropower plants is cost. In most cases, the cost per installed kilowatt for small hydro power is still higher than fossil fuel based plants. For low head hydroelectric installation with head less than 20m, the major costs are investment in the civil works structure and the cost of electromechanical equipment. If the cost of the structure can be reduced, small hydroelectric installation would be more feasible. The head and capacity of the schemes are the basic parameters which affect the installation cost of a SHP project. These parameters are considered as the cost sensitive parameters and based on this consideration cost analysis of low head SHP is done. The design of reliable and cost effective small hydropower plants capable of large-scale electrical energy production is a prerequisite for the effective use of hydropower as an alternative resource. In this sense, the design of a small hydroelectric plant or equivalently the determination of type and energy load of the particular hydro turbines should maximize the energy output together with the life-time of the machines. In all cases, the design objective is closely related to the total annual output of the overall hydro turbine operation in power terms.

All the types of scheme i.e. run-of-river, canal based and dam toe based scheme such scheme can be under low head category. Under high head major run-of river and dam based schemes are there. The high head schemes are more sites specific as compared to low head scheme. The numbers of components under run-of-river are more as compared to other type of schemes. Thus run-of-river low head SHP schemes has been considered under the present study.

## 4.1 METHODOLOGY FOR COST EVALUATION [22]

In order to find out the overall installation cost for different alternatives, cost of individual components has been collected. Based on the selection and design criterion discussed in the previous chapter, the sizes of different components under civil works have been determined for the given alternative under low head run-of-river. For various layouts having different head and capacity combination, design discharge is worked out. Sizing of components such as intake, channel, desilting tank, forebay, and spillway and penstock, powerhouse building has been worked out based on discharge, For given scheme, type and runner diameter of the turbine has been determined, based on specific speed. For a particular layout considered and worked out sizes of various components, based on the determined quantities and prevailing prices of these items, the cost of civil works components can be worked out. Based on type and sizes and prevailing prices of electromechanical equipment, cost of electromechanical equipment can be worked out.

## 4.2 ANALYSIS FOR COST OF DIFFERENT COMPONENTS OF RUN-OF-RIVER SCHEMES

For the installation of any low head SHP station, one of the major considerations is the cost involved into the project. To estimate the cost of various components of low head SHP scheme, correlation for cost of various components of low head run-of-river SHP scheme, as function of installed capacity and head developed from the determined values of cost. Microsoft Excel software is used for carrying out the regression analysis. Different plots are worked out on linear scale, to have the best curve fitting of the data.

### 4.2.1 Development of Correlations

For a range of capacity, head and other related parameter considered under the present study, it has been found that the cost is the strong function of capacity (P) and head (H) of the scheme. Therefore the functional relationship for cost per kW (C) can be written as:

$$C= f(P, H) \qquad \text{.......................} (4.1)$$

Based on this relationship the correlation for cost components of different alternatives and schemes are developed and detailed as follows.

### 4.2.2 Run-of-river low head SHP.

Data of cost used for diversion weir and intake are given in table 1 for different head and capacity which have been used for development of correlation for cost. First order regression of the data has been shown in Fig 4.1 and average value of exponent, ($X_1$, average slope of lines) has been found as -0.29. Therefore the following first order equation can be represented as:

$$C = a_0 P^{-0.29} \quad \text{...................... (4.2)}$$

The coefficient '$a_0$' will be a function of other parameter i.e. head.

In order to induce the effect of head (H) parameter, the values of $\frac{C}{P^{-0.29}}$ are calculated from the determined cost data. These values have been plotted against respective head values as in Fig 4.2 from the first order regression of the data values of constant '$a_1$' and exponent '$y_1$' are obtained equal to 29498 and -0.08 respectively. By putting these values in the Equation (4.2), correlation has been obtained for Cost per kW of Diversion Weir ($C_1$) as given below;

## 1. DIVERSION AND INTAKE

For diversion weir and intake the regression analysis used for correlation development is as shown in figure 4.1. The developed correlation for Cost per kW of Intake ($C_I$) is expressed as equation 4.3.

$$C_I = a_I P^{x_I} H^{y_I} \qquad \text{.......... (4.3)}$$

$a_I = 18176$
$x_I = -0.23$
$y_I = -0.05$

*Table 4.1 Cost of Intake for Canal based schemes at different combination of head and capacity*

| Head(m) | Capacity(kW) | Total | Total per kW | Cost per kW of Intake/ Capacity^-0.05 with Head |
|---|---|---|---|---|
| | | | | |
| 5 | 2000 | 5545047 | 2772 | 16544 |
| 10 | 2000 | 5324697 | 2662 | 15886 |
| 20 | 2000 | 5113105 | 2556 | 15253 |
| 5 | 4000 | 9422432 | 2355 | 16541 |
| 10 | 4000 | 9048004 | 2261 | 15883 |
| 20 | 4000 | 8688454 | 2172 | 15254 |
| 5 | 5000 | 11176080 | 2235 | 16540 |
| 10 | 5000 | 10731965 | 2146 | 15885 |
| 20 | 5000 | 10305499 | 2061 | 15252 |
| 5 | 8000 | 16011090 | 2001 | 16540 |
| 10 | 8000 | 15374841 | 1921 | 15884 |
| 20 | 8000 | 14763875 | 1845 | 15254 |
| 5 | 10000 | 18990979 | 1899 | 16542 |
| 10 | 10000 | 18236316 | 1824 | 15887 |
| 20 | 10000 | 17511641 | 1751 | 15250 |

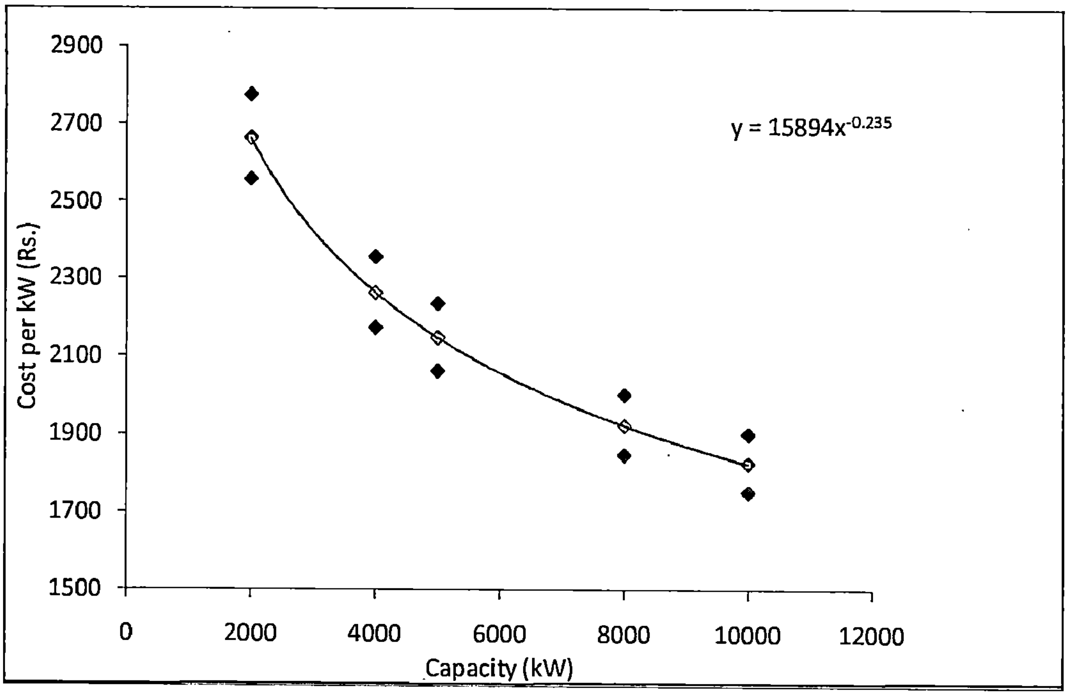



*Figure 4.1 Plot of Cost per kW of Intake with Capacity*

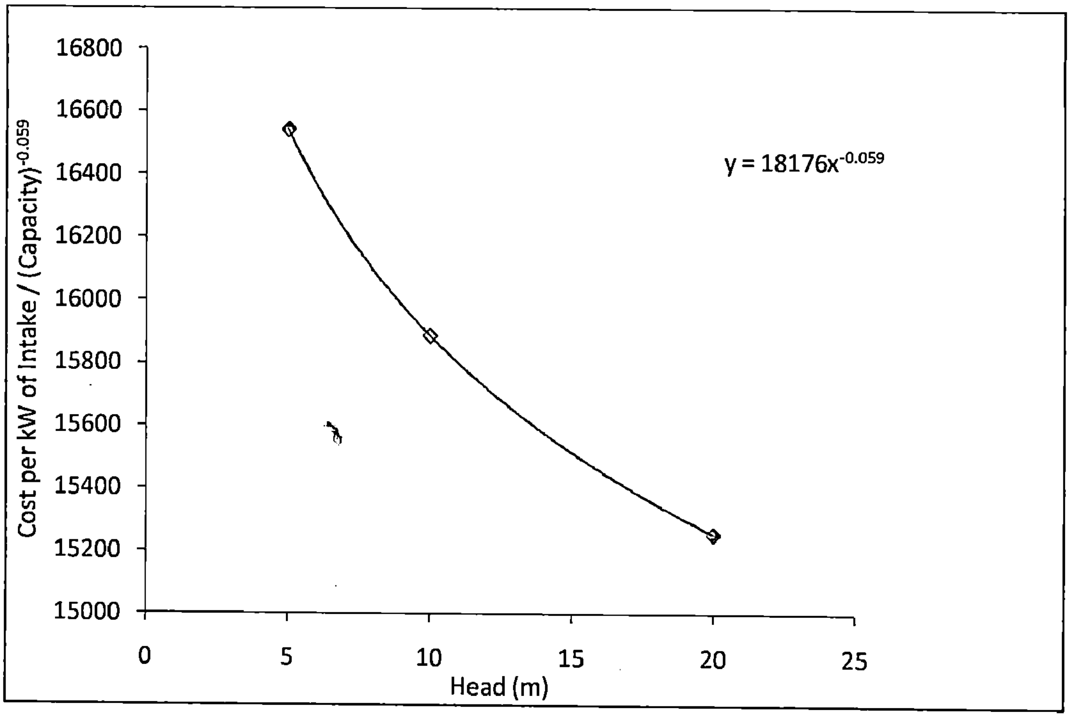



*Figure 4.2 Plot of Cost per kW of Intake/ Capacity^-0.059 with head*

## 2. POWER CHANNEL

For power channel the regression analysis used for correlation development is as shown in figure 4.3 and figure 4.4. The developed correlation for Cost per kW of Power Channel ($C_2$) is expressed as equation 4.4.

$$C_2 = a_2 P^{x2} H^{y2} \quad \text{................. (4.4)}$$

$a_2$= 13164
$x_2$= -0.24
$y_2$= -0.06

Table 4.2 *Cost of Power Channel for canal based schemes at different combination of head and capacity*

| Head(m) | Capacity(kW) | Total Cost | total cost per kW | Cost per kW of Penstock/ Capacity^-0.24with head |
|---|---|---|---|---|
| | | | | |
| 5 | 2000 | 3955837 | 1978 | 12259 |
| 10 | 2000 | 3769667 | 1885 | 11682 |
| 20 | 2000 | 3462803 | 1731 | 10731 |
| 5 | 4000 | 6342937 | 1586 | 11607 |
| 10 | 4000 | 6091820 | 1523 | 11148 |
| 20 | 4000 | 5860681 | 1465 | 10725 |
| 5 | 5000 | 7399055 | 1480 | 11428 |
| 10 | 5000 | 7251201 | 1450 | 11199 |
| 20 | 5000 | 6955183 | 1391 | 10742 |
| 5 | 8000 | 10941748 | 1368 | 11823 |
| 10 | 8000 | 10715036 | 1339 | 11578 |
| 20 | 8000 | 9965280 | 1246 | 10768 |
| 5 | 10000 | 12940269 | 1294 | 11802 |
| 10 | 10000 | 12934139 | 1293 | 11796 |
| 20 | 10000 | 11904500 | 1190 | 10857 |

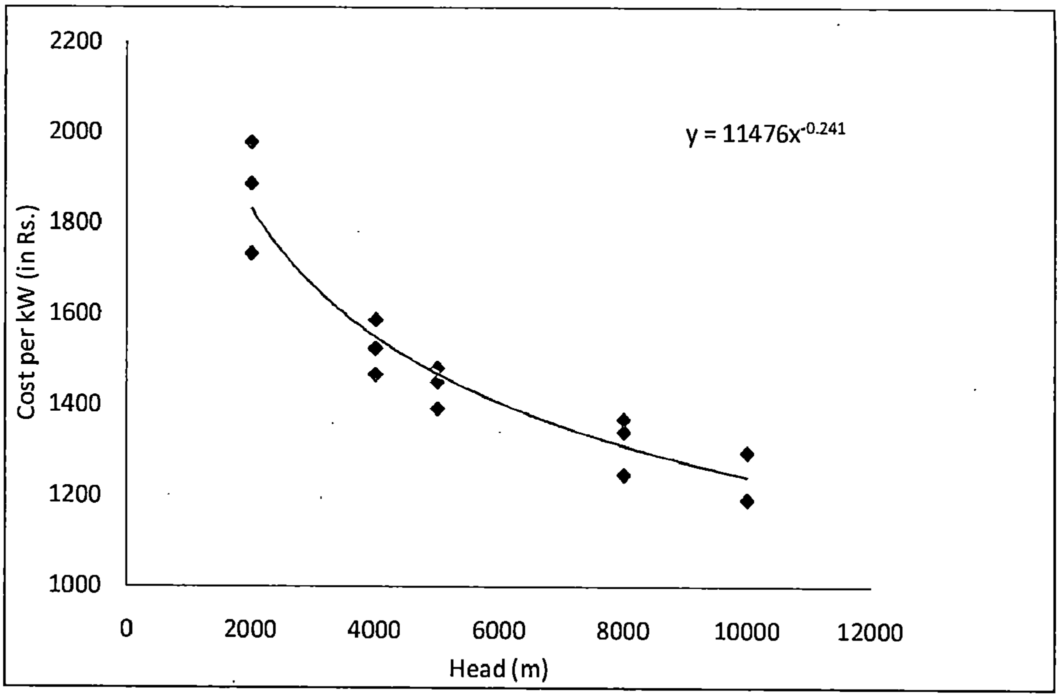



*Figure 4.3 Plot of Cost per kW of Power Channel with Capacity*

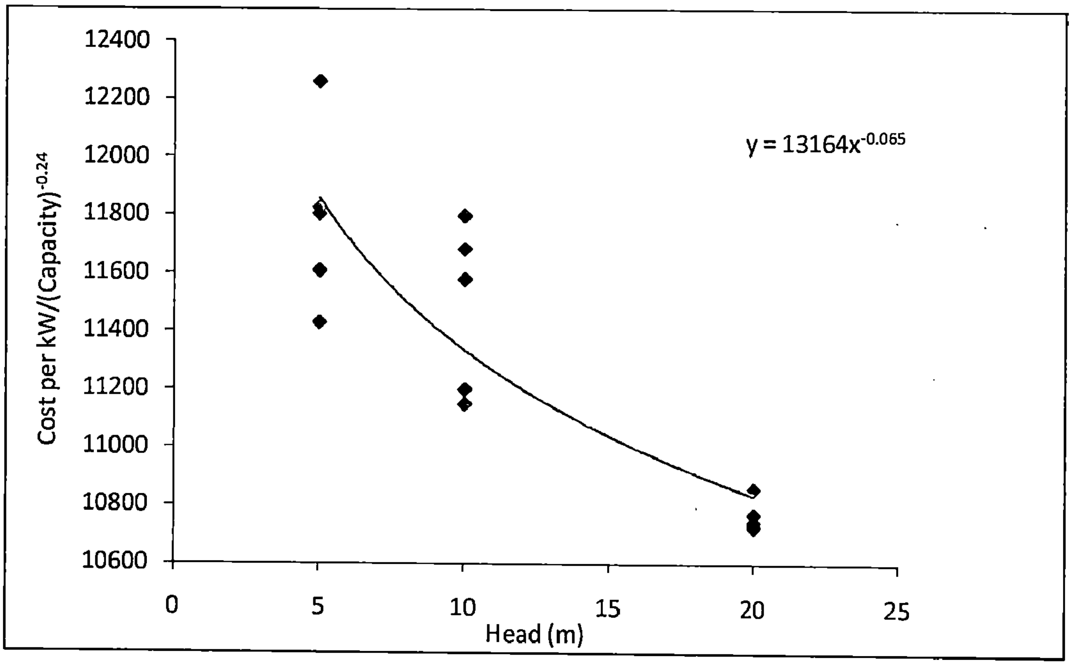



*Figure 4.4 Plot of Cost per kW of Power Channel/ (Capacity)^-0.24 with Head*

### 3. DESILTING TANK

Correlation for cost of Desilting tank has been developed. Figure 4.5 and 4.6 shows steps involved in regression analysis for a typical case. Correlation for Cost per kW of Desilting Tank ($C_3$) is expressed by equation no. 4.5.

$$C_3 = a_3 P^{x3} H^{y3} \qquad \text{................ (4.5)}$$

$a_3$= 17549
$x_3$= -0.23
$y_3$= -0.03

*Table 4.3 Cost of Desilting tank for canal based schemes at different combination of head and capacity*

| Head(m) | Capacity(kW) | Total | Total per kW | Cost per kW of Desilting tank/ Capacity^-0.06 with Head |
|---|---|---|---|---|
| | | | | |
| 5 | 2000 | 6276117 | 3139 | 1040371 |
| 10 | 2000 | 6026717 | 3009 | 997364 |
| 20 | 2000 | 5787227 | 2891 | 958111 |
| 5 | 4000 | 10664706 | 2661 | 1497278 |
| 10 | 4000 | 10240911 | 2557 | 1438754 |
| 20 | 4000 | 9833958 | 2454 | 1380810 |
| 5 | 5000 | 12649558 | 2508 | 1673014 |
| 10 | 5000 | 12146890 | 2408 | 1606369 |
| 20 | 5000 | 11664197 | 2331 | 1555525 |
| 5 | 8000 | 18122020 | 2163 | 2065692 |
| 10 | 8000 | 17401893 | 2206 | 2107006 |
| 20 | 8000 | 16710377 | 2129 | 2033231 |
| 5 | 10000 | 21494790 | 2089 | 2366569 |
| 10 | 10000 | 20640630 | 2108 | 2387564 |
| 20 | 10000 | 19633639 | 2045 | 2316415 |

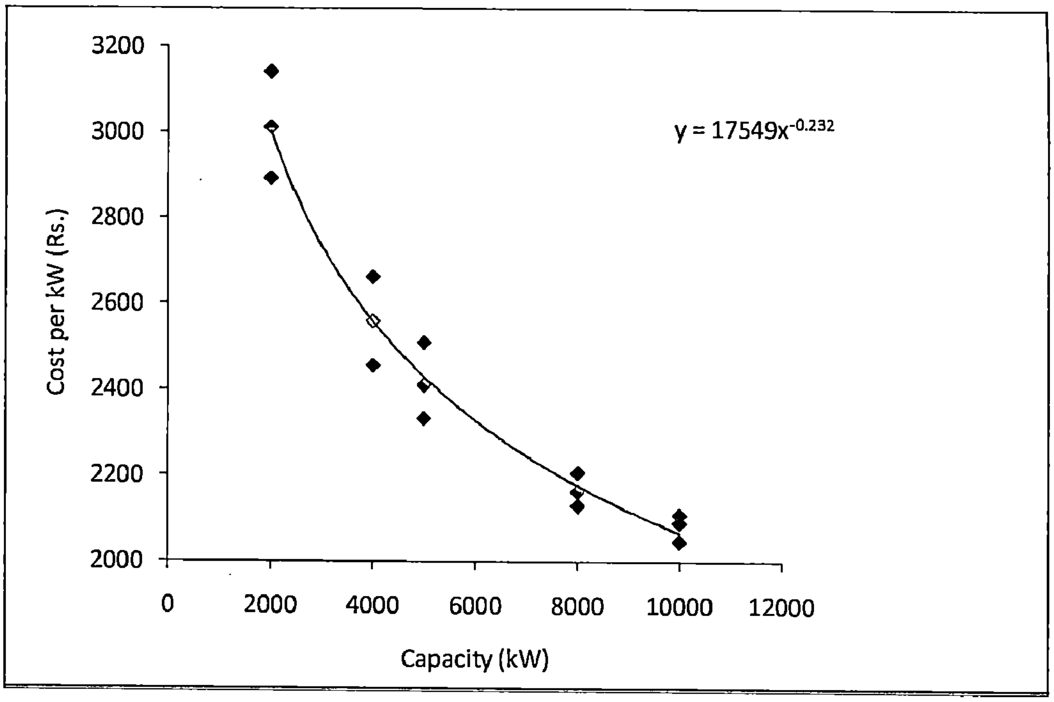



*Figure 4.5 Plot of Cost per kW of Desilting tank with Capacity*

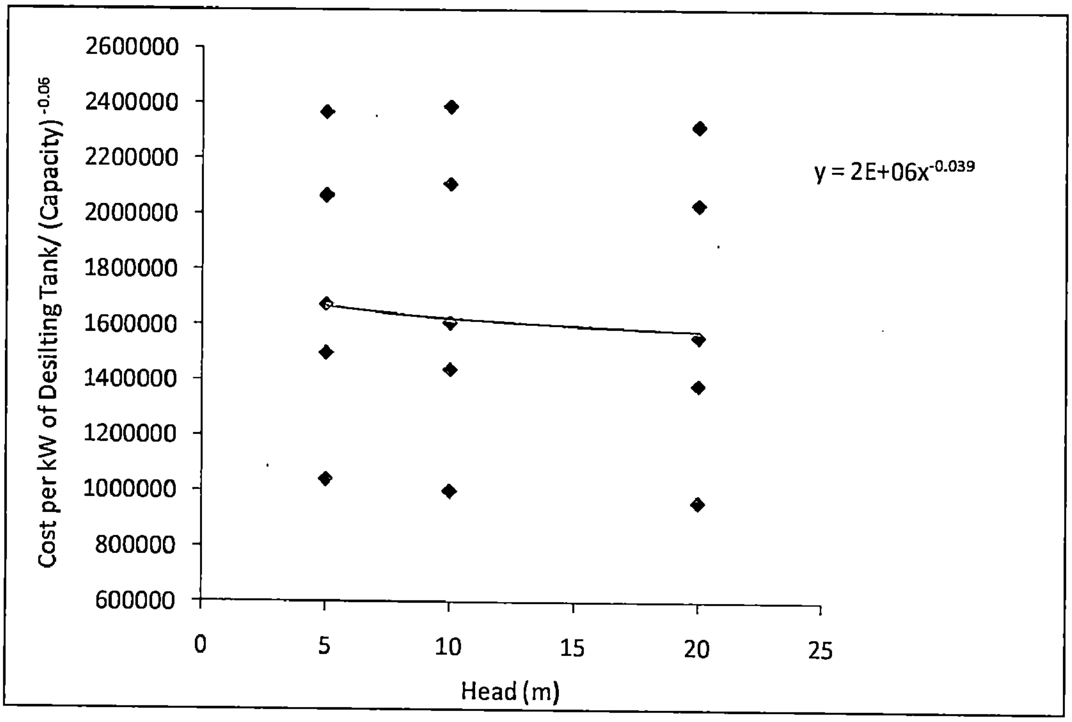



*Figure 4.6 Plot of Cost per kW of Desilting tank/ capacity^-0.06 with head*

## 4. FOREBAY

In order to develop correlation for cost of Forebay data for costs, determined in Fig. 4.7 and 4.8 show the regression analysis steps for development of correlation for a typical case. Table 4.4 gives the values of constants and exponents obtained from different alternatives.

$$C_4 = a_4 P^{x}{}_4 H^{y}{}_4 \qquad \text{............ (4.6)}$$

$a_4 = 24868$
$x_4 = -0.23$
$y_4 = -0.05$

*Table 4.4 Cost of Forebay for canal based schemes at different combination of head and capacity*

| Head(m) | Capacity(kW) | Total Cost | Total Cost per kW | Cost per kW of Forebay/ Capacity^-0.23 with head |
|---|---|---|---|---|
| | | | | |
| 5 | 2000 | 7935487 | 3967 | 22790 |
| 10 | 2000 | 7620147 | 3809 | 21885 |
| 20 | 2000 | 7317337 | 3658 | 21015 |
| 5 | 4000 | 13484394 | 3371 | 22712 |
| 10 | 4000 | 1295131 | 3237 | 21809 |
| 20 | 4000 | 12434002 | 3108 | 20942 |
| 5 | 5000 | 15994030 | 3199 | 22688 |
| 10 | 5000 | 14998448 | 2999 | 21271 |
| 20 | 5000 | 14748145 | 2949 | 20920 |
| 5 | 8000 | 22913387 | 2864 | 22633 |
| 10 | 8000 | 22002854 | 2750 | 21729 |
| 20 | 8000 | 21128505 | 2640 | 20867 |
| 5 | 10000 | 27177891 | 2718 | 22608 |
| 10 | 10000 | 25486145 | 2548 | 21195 |
| 20 | 10000 | 24987311 | 2498 | 20783 |

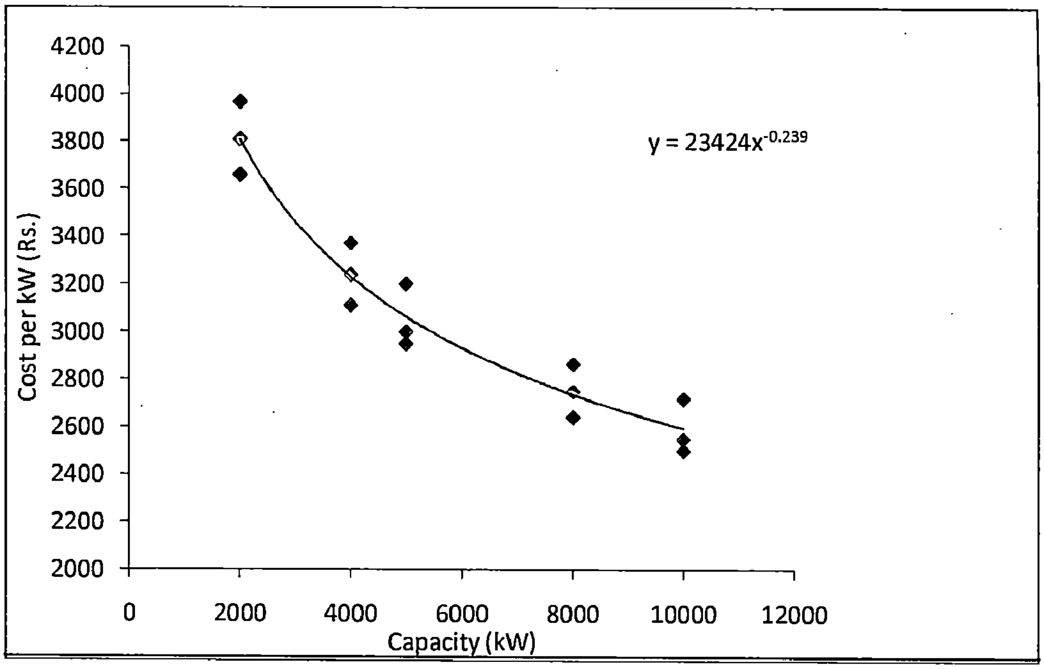



*Figure 4.7 Plot of Cost per kW of Forebay with Capacity*

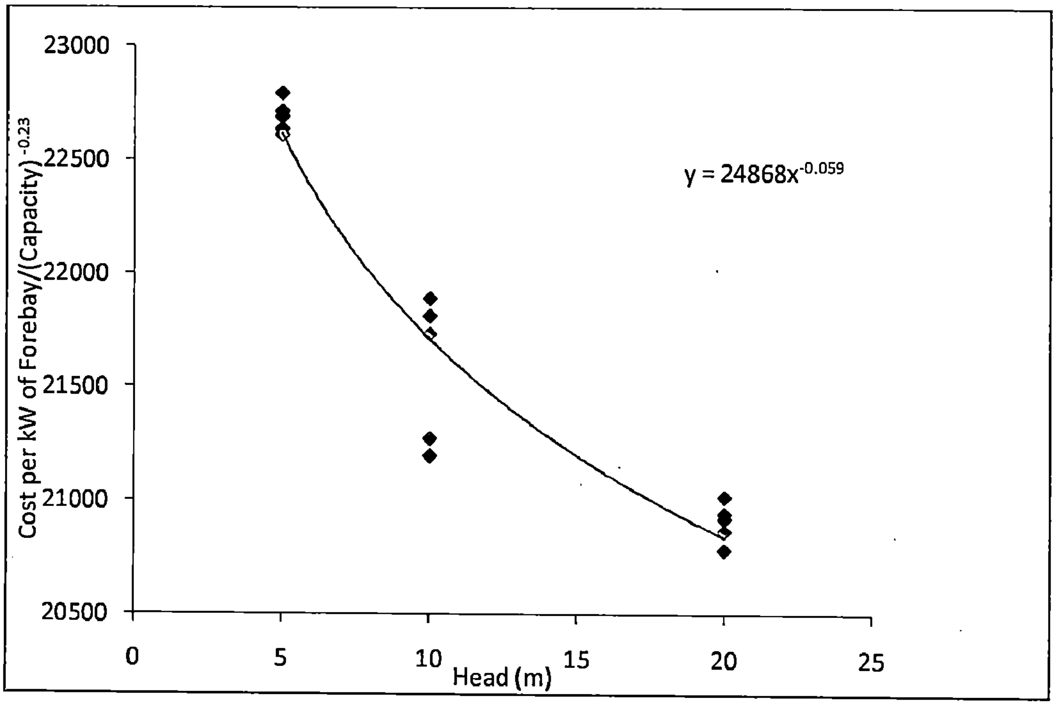



*Figure 4.8 Plot of Cost per kW of Forebay/ Capacity^-0.23 with head*

## 5. PENSTOCK

Similarly, regression analysis steps for development of correlation for a Penstock is determined as shown in figure 4.9 and 4.10. Different values of constants and exponents obtained are as follows.

$$C_5 = a_5 P^{x_5} H^{y_5} \quad \text{............ (4.7)}$$

$a_5 = 7141$
$x_5 = -0.38$
$y_5 = 0.468$

*Table 4.5 Cost of Penstock for canal based schemes at different combination of head and capacity*

| Head(m) | Capacity(kW) | Total cost | Total per kW | Cost per kW of Penstock/ Capacity^-0.38 with head |
|---|---|---|---|---|
| | | | | |
| 5 | 2000 | 1658219 | 829 | 14894 |
| 10 | 2000 | 2158658 | 1079 | 25233 |
| 20 | 2000 | 2810126 | 1404 | 32842 |
| 5 | 4000 | 2547592 | 636 | 14880 |
| 10 | 4000 | 3316438 | 829 | 21098 |
| 20 | 4000 | 4317316 | 1070 | 27466 |
| 5 | 5000 | 2925269 | 585 | 14886 |
| 10 | 5000 | 3808094 | 761 | 23155 |
| 20 | 5000 | 4957350 | 991 | 30174 |
| 5 | 8000 | 3913976 | 489 | 14883 |
| 10 | 8000 | 5095186 | 636 | 21077 |
| 20 | 8000 | 6632877 | 829 | 27455 |
| 5 | 10000 | 4494215 | 449 | 14870 |
| 10 | 10000 | 5850538 | 585 | 19372 |
| 20 | 10000 | 7616188 | 761 | 25204 |

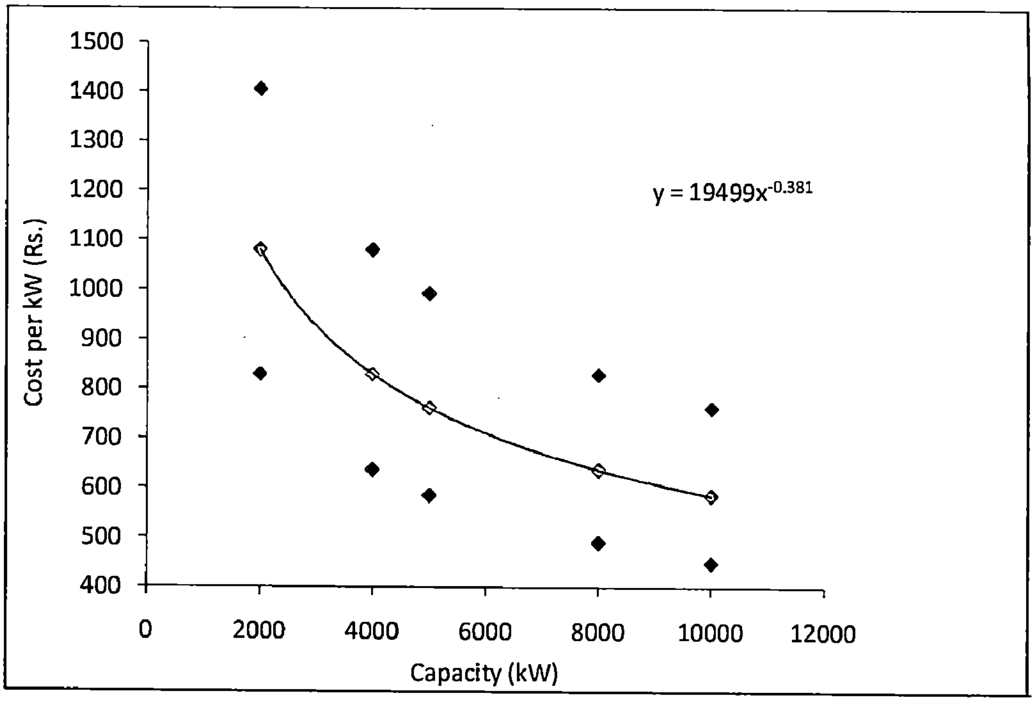



*Figure 4.9 Plot of Cost per kW of Penstock with Capacity*

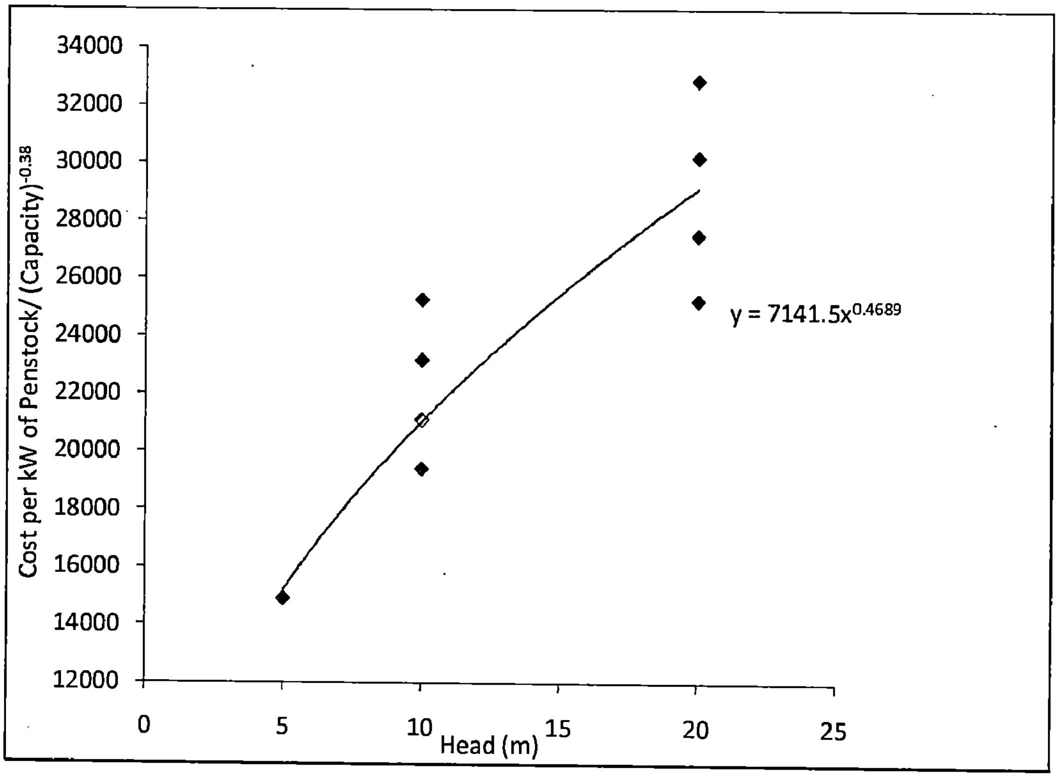



*Figure 4.10 Plot of Cost per kW of Penstock/ Capacity^-0.38 with head*

## 6. POWER HOUSE

The parameters which affect the cost of power house are as follows:

i) Location of PowerStation

ii) Soil Condition

iii) Type of Turbine

iv) Number of Generating Units

The developed correlations for Cost per kW of Powerhouse ($C_6$) are given below.

$$C_6 = a_6 P^{x_6} H^{y_6} \qquad \text{.......... (4.8)}$$

$a_6 = 91338$
$x_6 = -0.23$
$y_6 = -0.05$

*Table 4.6 Cost of Power House canal based schemes at different combination of head and capacity*

| Head(m) | Capacity(kW) | Total Cost | Cost per kW | Cost per kW/Capacity^-0.23 |
|---|---|---|---|---|
| 5 | 2000 | 29079392 | 14539 | 83519 |
| 10 | 2000 | 27923835 | 13961 | 80199 |
| 20 | 2000 | 26814199 | 13407 | 77016 |
| 5 | 4000 | 49413222 | 12353 | 83229 |
| 10 | 4000 | 47449640 | 11862 | 79919 |
| 20 | 4000 | 45564086 | 11390 | 76740 |
| 5 | 5000 | 58609720 | 11722 | 83134 |
| 10 | 5000 | 56280687 | 11255 | 79825 |
| 20 | 5000 | 54044205 | 10808 | 76655 |
| 5 | 8000 | 83965527 | 10495 | 82931 |
| 10 | 8000 | 80628905 | 10078 | 79635 |
| 20 | 8000 | 77424874 | 9677 | 76469 |
| 5 | 10000 | 99592696 | 9959 | 82835 |
| 10 | 10000 | 9563508 | 9563 | 79546 |
| 20 | 10000 | 93393337 | 9183 | 76384 |

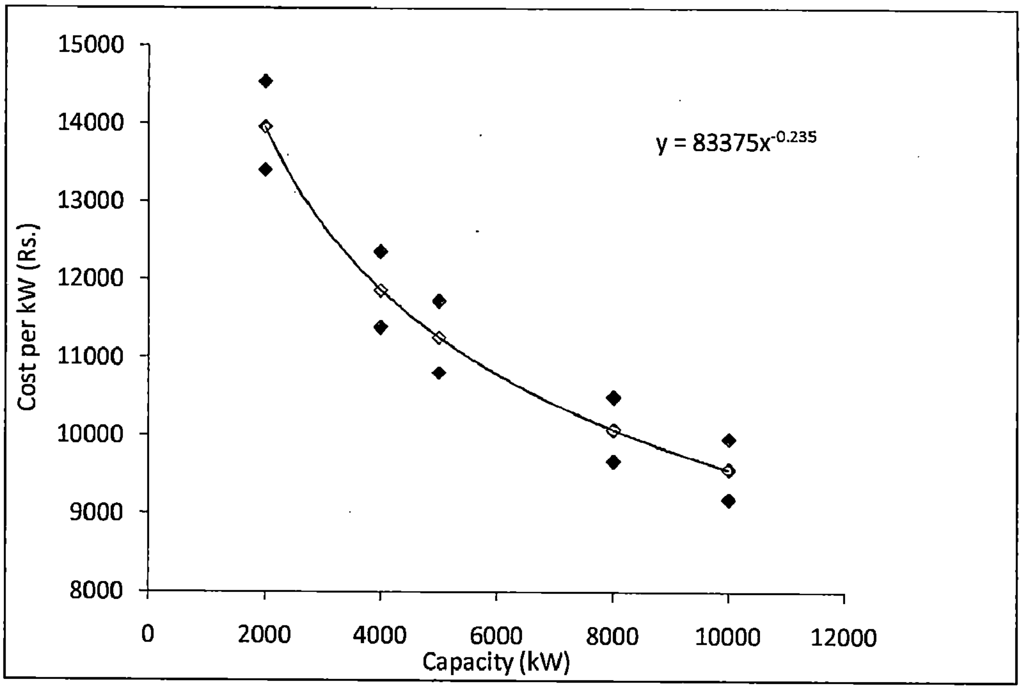



*Figure 4.11 Plot of Cost per kW of Power House with Capacity*

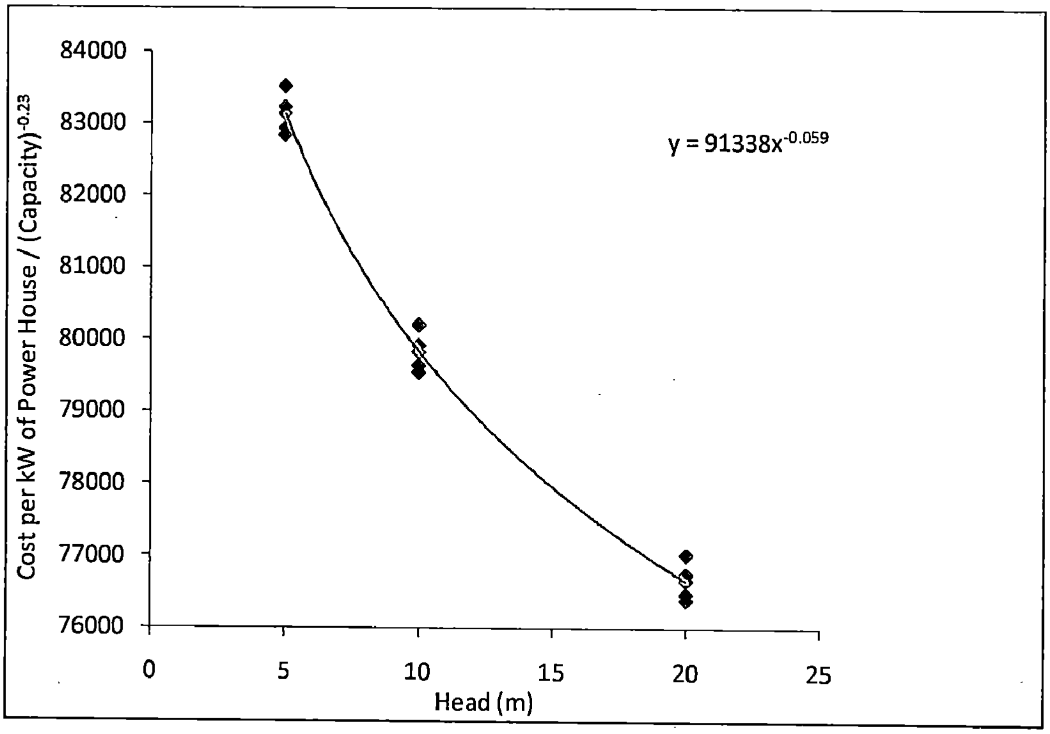



*Figure 4.12 Plot of Cost per kW of Power House/ Capacity^-0.23 with head*

## 7. TAILRACE CHANNEL

Using the cost data generated in Table 4.7 the regression analysis steps for development of correlation for a typical case are shown in Figure 4.13 and 4.14. And different values of constants and exponents obtained from are given in below.

$$C_7 = a_7 P^{x7} H^{y7} \qquad \text{................ (4.9)}$$

$a_7$= 28996
$x_7$= -0.37
$y_7$= -0.62

*Table 4.7 Cost of Tailrace Channel for canal based schemes at different combination of head and capacity*

| Head(m) | Capacity(kW) | Total | Total per kW | Cost per kW of Tailrace/ Capacity^-0.376 with head |
|---|---|---|---|---|
| | | | | |
| 5 | 2000 | 1219589 | 609 | 10625 |
| 10 | 2000 | 791354 | 395 | 6892 |
| 20 | 2000 | 513484 | 256 | 4469 |
| 5 | 4000 | 1879559 | 469 | 10621 |
| 10 | 4000 | 1219589 | 304 | 6894 |
| 20 | 4000 | 791354 | 197 | 4472 |
| 5 | 5000 | 2160369 | 431 | 10613 |
| 10 | 5000 | 1401797 | 280 | 6889 |
| 20 | 5000 | 909583 | 182 | 4483 |
| 5 | 8000 | 2896669 | 362 | 10640 |
| 10 | 8000 | 1879559 | 234 | 6891 |
| 20 | 8000 | 1219589 | 152 | 4473 |
| 5 | 10000 | 3329435 | 332 | 10617 |
| 10 | 10000 | 2160369 | 216 | 6903 |
| 20 | 10000 | 1401797 | 140 | 4470 |

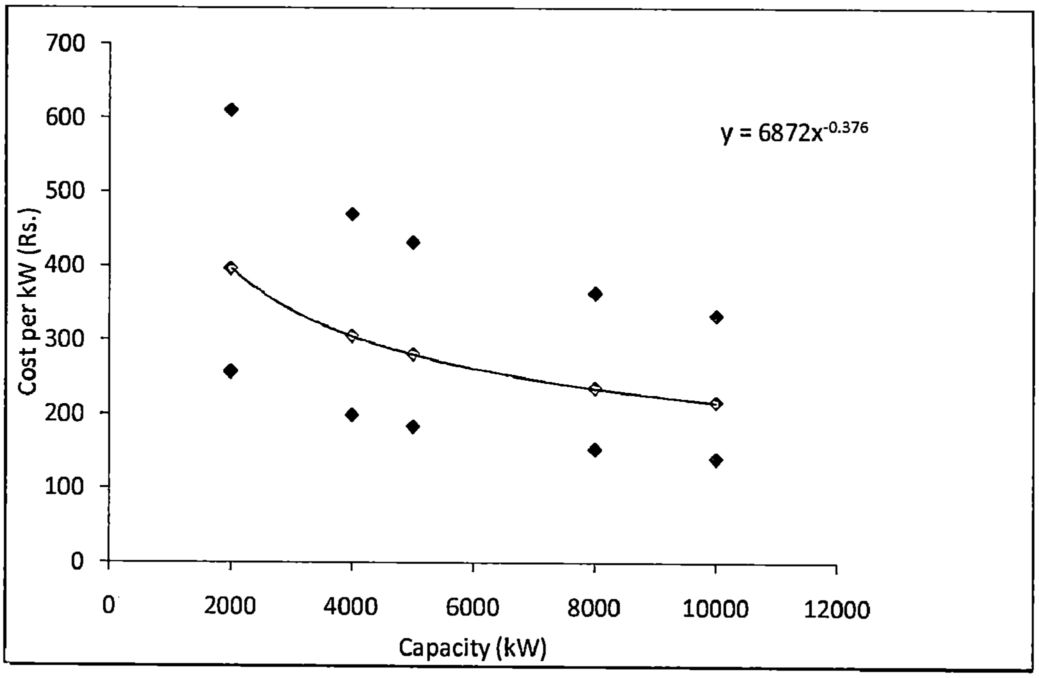



*Figure 4.13 Plot of Cost per kW of Tailrace Channel with Capacity*

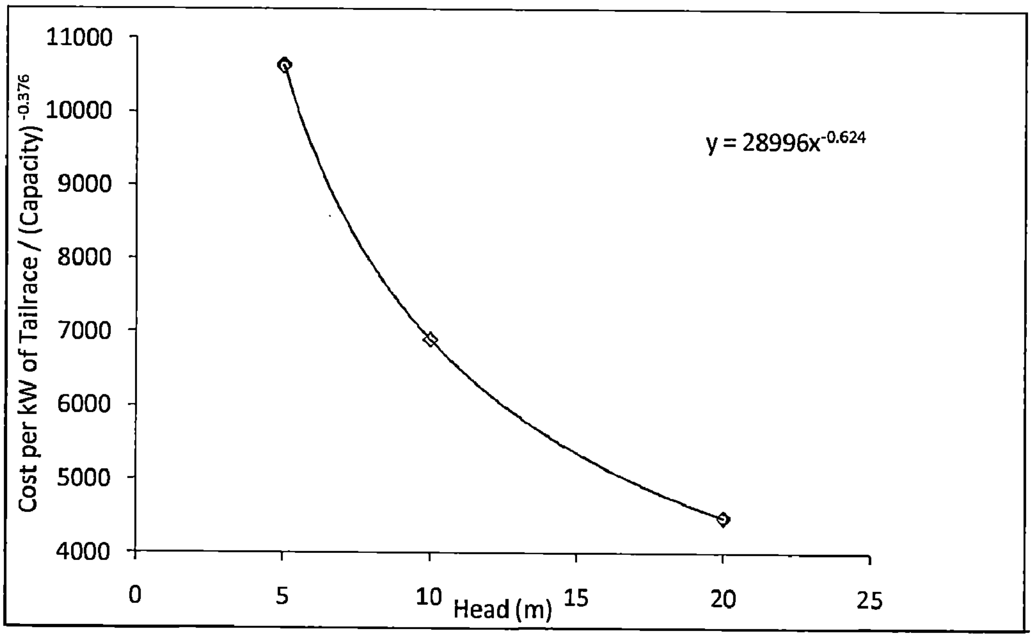



*Figure 4.14 Plot of cost per kW of Tailrace/ Capacity^-0.376 with head*

## 8. ELECTRO-MECHANICAL EQUIPMENT

Electro-mechanical equipments depend on the head and unit capacity. Therefore, components under equipment are similar for all three types of SHP schemes. Various alternatives such as turbines, type of generator and numbers of generating units have been considered for cost estimates of the components. Correlation has been developed by regression analysis considering Head and Capacity as cost sensitive parameters.

$$C_8 = a_8 P^{x8} H^{y8} \qquad \text{.............} (4.10)$$

$a_8$= 195226
$x_8$= -0.18
$y_8$= -0.19

*Table 4.8 Cost of E & M work for canal based schemes at different combination of head and capacity*

| Head(m) | Capacity(kW) | Total | Total per kW | Cost per kW of E & M Work/ Capacity^-0.199 with head |
|---|---|---|---|---|
| | | | | |
| 5 | 2,000 | 70,040,000 | 35,020 | 137,563 |
| 10 | 2,000 | 60,162,300 | 30,080 | 118,158 |
| 20 | 2,000 | 53,560,000 | 26,780 | 105,195 |
| 5 | 4,000 | 123,600,000 | 30,900 | 137,508 |
| 10 | 4,000 | 107,120,000 | 26,780 | 119,174 |
| 20 | 4,000 | 93,524,000 | 23,380 | 104,044 |
| 5 | 5,000 | 149,350,000 | 29,870 | 138,373 |
| 10 | 5,000 | 128,750,000 | 25,750 | 119,287 |
| 20 | 5,000 | 112,270,000 | 22,450 | 103,999 |
| 5 | 8,000 | 218,360,000 | 27,290 | 137,581 |
| 10 | 8,000 | 189,520,000 | 23,690 | 119,432 |
| 20 | 8,000 | 164,800,000 | 20,600 | 103,854 |
| 5 | 10,000 | 257,500,000 | 25,750 | 135,138 |
| 10 | 10,000 | 226,600,000 | 22,660 | 118,921 |
| 20 | 10,000 | 197,760,000 | 19,770 | 103,754 |

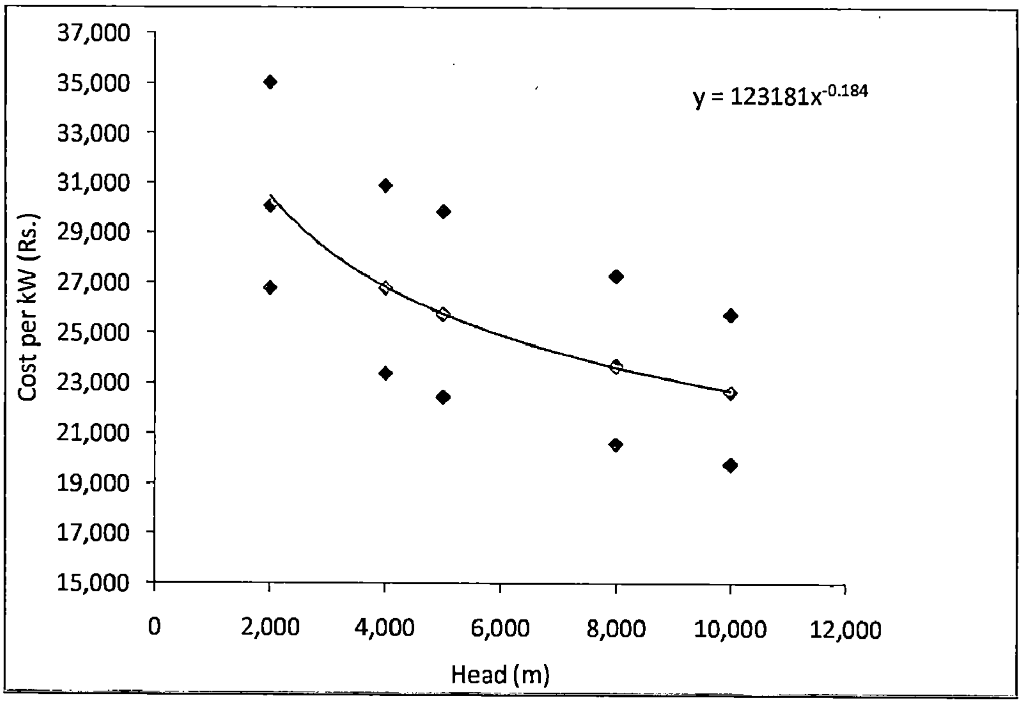



*Figure 4.15 Plot of Cost per kW of E & M Work with Capacity*

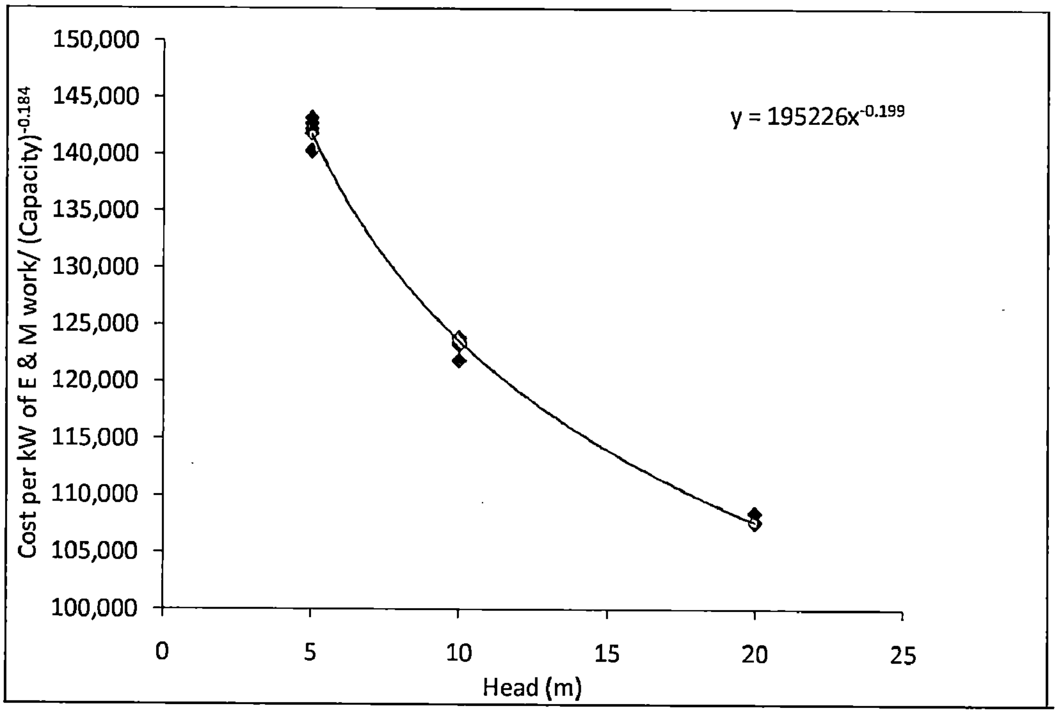



*Figure 4.16 Plot of Cost per kW of E & M Work/ Capacity^-0.184 with head*

## 4.3 TOTAL INSTALLATION COST

Based on the correlation developed for the cost of different components under different run-of-river SHP schemes, installation cost has been worked for various layouts. The total project cost includes cost of civil works, cost of electromechanical equipment, cost of other miscellaneous items and other indirect costs; Establishment related cost including designs, audit and account, indirect charges, tools and plants communication expenses, preliminary expenses on report preparation, survey and investigation and cost of lands were considered under miscellaneous and indirect costs. 13% of civil works and electromechanical equipment has been taken on account of this cost.

### 4.3.1 RUN-OF-RIVER SCHEME

Under civil works the cost of diversion weir and intake, power channel, desilting tank, forebay, spillway, penstock and power house have been correlated as function of capacity and head. Electromechanical equipment consisting of turbine, generator, auxiliaries' and transformer has been considered and the cost of these correlated. Based on the cost computed as per correlation developed, the cost of the run of river project is given in Table 4.10. for a layout of 5000kW capacity at 5m head.

*Table 4.9 Cost of Run-of-River Project calculated from correlations developed at 5m head and 5000kW installed capacity*

| Items | Cost (Rs.) | % distributed cost |
|---|---|---|
| Diversion weir & intake | 14968409 | 3.04 |
| Power channel | 7738679 | 1.57 |
| Desilting tank | 11789358 | 2.40 |
| Forebay | 16177047 | 3.29 |
| Penstock | 3780151 | 0.77 |
| Power House | 99916887 | 20.32 |
| Tail Race Channel | 3287324 | 0.67 |
| E & M Equipments | 269995060 | 54.90 |
| Sub Total(Civil Works) | 157657855 | 32.06 |
| Total (civil works & E&M Equipments) | 427652915 | 86.96 |
| Other Expenses | 64147937 | 13.04 |
| **Total Cost** | **491800852** | **100.00** |

The cost of component as determined based on developed correlation has been shown in fig.4.19

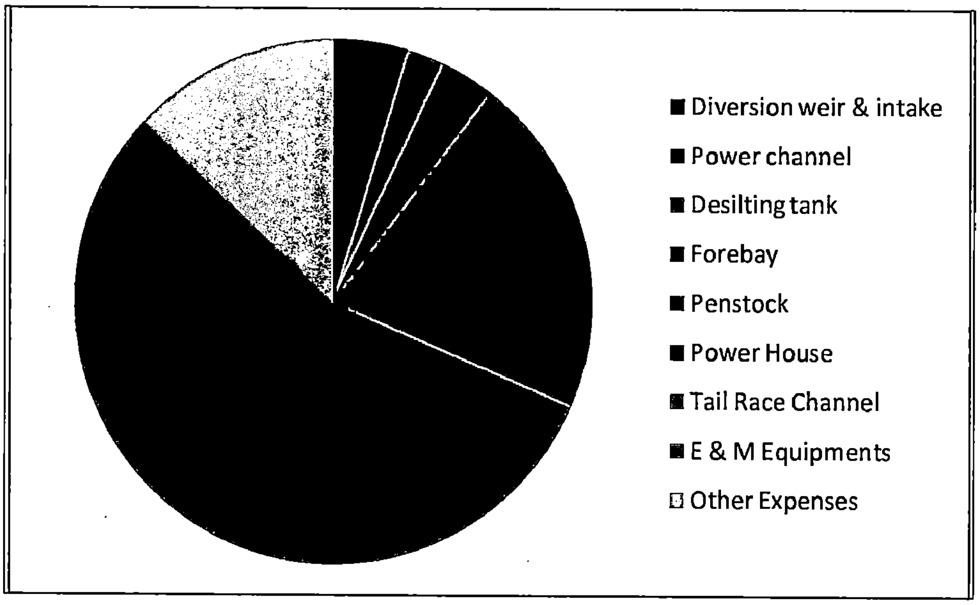



*Figure 4.17 Cost distribution for SHP of 5m*5000kW*

It has been found that major contribution of cost is from power house building which depends upon the type of turbine. Thus a computer program has been developed to determine the cost of power house based on type of turbine. The size of the turbine is represented by runner diameter.

## 4.4 DEVELOPMENT OF SOFTWARE FOR COST DETERMINATION BASED ON TYPE OF TURBINE SELECTION

The Civil works of small hydro power projects constitutes a major portion of the total cost. This is more so for the low head power plants. Efforts to minimize costs are therefore required in this area.

At various stages within the project development process, prior to project design and construction different levels of economic assessment are required. When screening a large number of potential sites for possible viable projects. Only a cursory evaluation is required. A rough determination of project cost and expected output is usually sufficient.

The need for software which can do this evaluation for low head small hydro sites is obvious. This can render the arduous task of lengthy computations very easy. It is in pursuit of this end. This program has been developed.

**About Program of Power House Cost**

It gives the possible layout and turbine configuration for a potential site along with the prediction cost for power house cost for each option. One can select the most economical option. The program prompts the user to input available Head (H), Powerhouse Capacity (P), Number of Units to be installed (U) and the turbine rotational speed (N).

It then checks the turbine types/layouts available under these conditions. Calculate the cost of powerhouse for each case and presents this information in tabular form.

It asks the user if he want to check same information for a different value of N and again performs the same option for this new value. This is done repeatedly as long as the user wants.

This way one can know about the different options of turbine and powerhouse layouts from this software in a very easy manner.

## 4.5 PROGRAM FOR CALCULATING POWER HOUSE COST

The program code is given below.

```
/*Program  for Calculating Power House Cost*/

   # include <stdio.h>
   # include <math.h>
   # include <conio.h>

   int H;     //Rated Head(in m)
   int U;     //Number of Units
   float N;   //Turbine Speed (rpm)
   float P;   //Power (in MW)

   float Ns,Df,Dp,Q;

   float C1,C2,C3,C4,C5,C6;

//-----------------------------------------------------------------------------------------------
   //Inputting the Turbine Data
   int Input()
   {
      // Enter Rated Head upto 20 m
      printf("\n\n  Enter Rated Head (in m)  : ");
      scanf("%d",&H);

      while(H>20)
      {
            printf("\n\n  SORRY! THIS PROGRAM IS FOR H LESS THAN 20 m. PLEASE TRY AGAIN !");
            printf("\n\n\tEnter Rated Head (in m)  : ");
            scanf("%d",&H);
      }
```

```
    // Enter Power upto 15000 kW
    printf("\n\n  Enter Power (in kW)      : ");
    scanf("%f",&P);

    while(P>15000)
    {
          printf("\n\n  SORRY! THIS PROGRAM IS FOR P UPTO 15000 kW. PLEASE TRY AGAIN !");

          printf("\n\n\tEnter Power (in kW)     : ");
          scanf("%f",&P);
    }

    //convert power in MW
    P=P/1000;

    //Enter Number of Units
    printf("\n\n  Enter Number of units    : ");
    scanf("%d",&U);

    return(0);
  }

//-----------------------------------------------------------------------------------------------------------

  //calculate Ns,Df,Dp from formulae
  int Calculate_Initial()
  {
    Ns = (float)(0.2626*(N * (pow(P,0.5)/pow(H,1.25))));

    Df = 84.6 * (0.0211 * (pow(Ns,(0.666)))) * (pow(H,0.5))/N;
```

```
    Dp = 84.6 * (0.0223 *  pow(Ns,(0.666)) ) * (pow(H,0.5))/N;

    printf("\n\n  Running Diamter Ns       = %f", (float)Ns);
    printf("\n\n  Running Diameter Df      = %f",(float)Df);
    printf("\n\n  Running Diameter Dp      = %f\n\n",(float)Dp);

    return (0);
  }

//-----------------------------------------------------------------------------------------------------------
  // Calculations for single unit
  int SUCalculate()
  {
    //Vertical Kaplan
    C1 = 142.82* pow(Dp,2) + 491.94* Dp + 95.21;

    //Vertical Francis
    C2 = (-17.05) * pow(Dp,3) + 167.45 * pow(Df,2) - 20.16 * Df + 145.74;

    //Semi kaplan tubular
    C3 = 52.32 * pow(Dp,2) - 22.63 * Dp + 101.25;

    //Tubular
    C4 = 48.56 * pow(Dp,2) + 197.43 * Dp + 106.72;

    //Bulb
    C5 = 116.1 * pow(Dp,2) + 103.2 * Dp + 94.6;

    //Rim
    C6 = 72 * pow(Dp,2) + 10.94 * Dp + 118.08;

    return (0);
  }
```

```
//----------------------------------------------------------------------------------------------------------

//Calculations for more than one unit
int MUCalculate()
{
  //Vertical Kaplan
  C1 = U *( (-15.87) * pow(Dp,3) + 277.71 * pow(Dp,2) + 47.61* Dp + 301.51);

  //Vertical Francis
  C2 = U * ( (-26.05) * pow(Dp,3) + 232.57 * pow(Df,2) - 192.25 * Df + 173.65 );

  //Semi Kaplan tubular
  C3 = U * ( 41.86 * pow(Dp,2) - 13.6 * Dp + 83.71 );

  //Tubular
  C4 = U * ( 30.95 * pow(Dp,2)+ 240.12 * Dp +17.07 );

  //Bulb
  C5 = U * ( 111.8 * pow(Dp,2) + 68.8 * Dp + 68.8);

  //Rim
  C6 = U * ( 66.24 * pow(Dp,2)- 1.9* Dp + 103.68);

  return (0);
}

//----------------------------------------------------------------------------------------------------------

// Display the calculated results
int Show_Result()
{
  printf("\n--------------------------------------------------------------------------------");
```

```
printf("\n  LAYOUT \t\t\t\t\t\t\tPOWERHOUSE COST\n\t\t\t\t\t\t\t(in million Rs)");

printf("\n----------------------------------------------------------------------------------------");

printf("\n\n\n  Discharge (m3/s) \t\t\t\t\t\t  %f",Q);


 printf("\n\n\n  Vertical Kaplan Turbine \t\t\t\t\t");
 printf("  %f",C1);

 printf("\n\n\n  Vertical Francis Turbine \t\t\t\t\t");
 printf("  %f",C2);

 printf("\n\n\n  Semi Kaplan Tubular \t\t\t\t\t\t");
 printf("  %f",C3);

 printf("\n\n\n  Tubular Kaplan Turbine \t\t\t\t\t");
 printf("  %f",C4);

 printf("\n\n\n  Bulb Turbine \t\t\t\t\t\t\t");
 printf("  %f",C5);

 printf("\n\n\n  Rim Turbine \t\t\t\t\t\t\t");
 printf("  %f",C6);


printf("\n\n\n------------------------------------------------------------------------------------------");

return (0);
}

//------------------------------------------------------------------------------------------------------------------------
```

```
int Calculate()
{
    //calculate Ns,Df,Dp from formulae
    Calculate_Initial();

    //if unit is single
    if( U == 1 )
    {
        SUCalculate();
    }
    //otherwise for more than one unit
    else
    {
        MUCalculate();
    }
    Q = (P *1000)/(9.81 * H * 0.85);

}

//-----------------------------------------------------------------------------------------------

int main()
{
  int n=1;

  //Take User Input
  Input();

  while( n == 1 )
  {
      //Enter Turbine Speed
      printf("\n\n  Enter Turbine Speed (rpm) : ");
```

```
        scanf("%f",&N);

        //Calculate Data
        Calculate();

        //Display Result
        printf("\n\n\n  For this Condition, following options are available : \n");
        Show_Result();

        //Continue?
        printf("DO YOU WANT TO CHECK WITH ANOTHER N ? Yes/No (1/0) : ");
        scanf("%d",&n);

    }

    return(0);
}
```

## 4.6 OUTPUT

Based on the software developed, cost of power house building having different layout on type of turbine has been computed as shown in Table 4.11.

*Table 4.10 Powerhouse Cost for different type of turbines*

| LAYOUT | POWERHOUSE COST (in million Rs.) Rated Heat = 5 m Power = 5000 kW |
|---|---|
| | |
| Discharge (m3/s) | 119.923 |
| Vertical Kaplan Turbine | 134.007 |
| Vertical Francis Turbine | 145.152 |
| Semi Kaplan Tubular | 99.815 |
| Tubular Turbine | 122.238 |
| Bulb Turbine | 103.251 |
| Rim Turbine | 119.352 |

For 5000kW capacity & 5m head the cost has been calculated using software program. Graphical representation of that cost of different types of turbine for 5m*5000kW is as shown in figure 4.18.

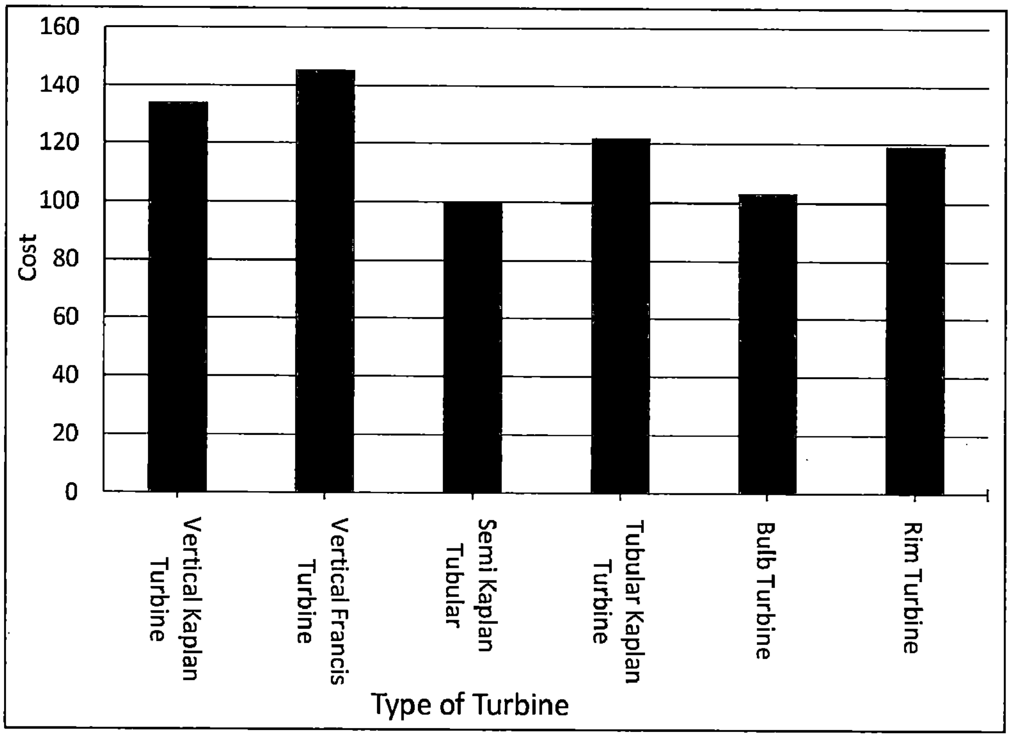



*Figure 4.18 Cost of plant w.r.t type of turbine*

From the table 4.10 and figure 4.18, it is seen that the layout with semi Kaplan tubular turbine is the cost effective layout, in the low head range while layout with vertical Francis turbine gives maximum cost.

# CHAPTER 5
# SPILLWAY DESIGN

In dam based project the components are intake, penstock, power house building, and tailrace channel. The components penstock, power house and tailrace are similar to the components in run-of river scheme. However the spillway is different. Selection and Design of Spillways is discussed in this chapter.

## 5.1 CAPACITY OF SPILLWAY

The capacity of a spillway is usually worked out on the basis of a flood routing study. As such, the capacity of a spillway is seen to depend upon the following major factors:

- The Inflow Flood
- The Volume of Storage Provided by the Reservoir
- Crest Height of the Spillway
- Gated or Un-Gated

According to the Bureau of Indian Standards guideline *IS: 11223-1985 "Guidelines for fixing spillway capacity"*, the following values of inflow design floods (IDF) should be taken for the design of spillway. [23]:

- For large dams (defined as those with gross storage capacity greater than 60 million $m^3$ or hydraulic head greater than 60 million $m^3$ or hydraulic head between (2m and 30m), IDF should be based on the Standard Project Flood (SPF).
- For intermediate dams those with gross storage between 10 and 60 million $m^3$ or hydraulic head between (2m and 30m), IDF should be based on the Standard Project Flood (SPF).
- For small dams (gross storage between 0.5 to 10 million $m^3$ or hydraulic head between 7.5m to 12m), IDF may be taken as the 100 year return period flood.

The volume of the reservoir corresponding to various elevation levels as well as the elevation of the crest also affects the spillway capacity.

## 5.2 SELECTION OF SPILLWAYS

The Bureau of Indian Standards code IS: 10137-1982 *"Guidelines for selection of spillways and energy dissipaters"* provide guidelines in choosing the appropriate type of spillway for the specific purpose of the project. The general considerations that provide the basic guidelines are as follows:

### 1- Safety Considerations Consistent with Economy

Spillway structures add substantially to the cost of a dam. In selecting a type of spillway for a dam, economy in cost should not be the only criterion. The cost of spillway must be weighed in the light of safety required below the dam.

### 2- Hydrological and Site Conditions

The type of spillway to be chosen shall depend on:

a) Inflow flood;

b) Availability of tail channel, its capacity and flow hydraulics;

c) Power house, tail race and other structures downstream; and

d) Topography

### 3- Type of Dam

This is one of the main factors in deciding the type of spillway. For earth and rockfill dams, chute and ogee spillways are commonly provided, whereas for an arch dam a free fall or morning glory or chute or tunnel spillway is more appropriate. Gravity dams are mostly provided with ogee spillways.

**4- Purpose of Dam and Operating Conditions**

The purpose of the dam mainly determines whether the dam is to be provided with a gated spillway or a non-gated one. A diversion dam can have a fixed level crest, that is, non-gated crest.

**5- Conditions Downstream of a Dam**

The rise in the downstream level in heavy floods and its consequences need careful consideration. Certain spillways alter greatly the shape of the hydrograph downstream of a dam. The discharges from a siphon spillway may have surges and break-ups as priming and depriming occurs. This gives rise to the wave travelling downstream in the river, which may be detrimental to navigation and fishing and may also cause damage to population and developed areas downstream.

**6- Nature and Amount of Solid Materials Brought by the River**

Trees, floating debris, sediment in suspension, etc, affect the type of spillway to be provided. A siphon spillway cannot be successful if the inflow brings too much of floating materials. Where big trees come as floating materials, the chute or ogee spillway remains the common choice.

Apart from the above, each spillway can be shown as having certain specific advantages under particular site conditions. These are listed below which might be helpful to decide which spillway to choose for a particular project.

### 5.2.1 Ogee Spillway [24]

It is most commonly used with gravity dams. However, it is also used with earth and rockfill dams with a separate gravity structure; the ogee crest can be used as control in almost all types of spillways; and it has got the advantage over other spillways for its high discharging efficiency.

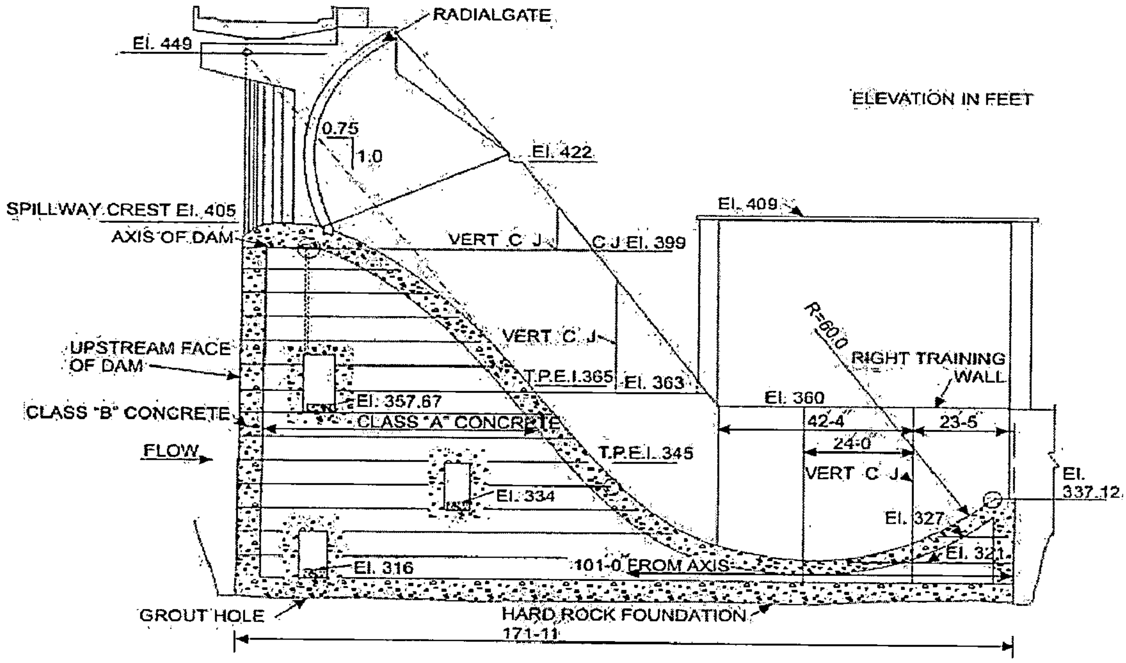



FIGURE 5. Typical overflow (ogee) spillway .Example of Panchet Dam on River Damodar

*Figure 5.1 Typical Overflow (Ogee) Spillway*

### 5.2.2 Chute Spillway

A Chute spillway, variously called as open channel or trough spillway, is one whose discharge is covered from the reservoir to the downstream river level through an open channel, placed either along a dam abutment or through a saddle. The control structure for the chute spillway need not necessarily be an overflow crest, and may be the side channel type.

Generally, the chute spillway has been mostly used in conjunction with embankment dams, like the Tehri dam, for example. Chute spillways are simple to design and construct and have been constructed successfully on all types of foundation materials, ranging from solid from solid to soft clay.

Chute spillways ordinarily consist of an entrance channel, a control structure, a discharge channel, a terminal structure, and an outlet channel. Often, the axis of the entrance channel or that of the discharge channel must be curved to fit the topography.

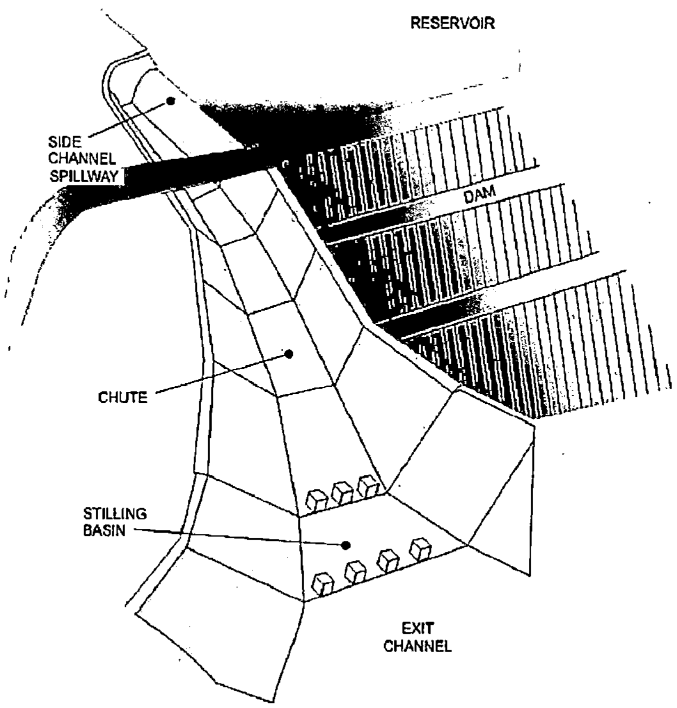



*Figure 5.2 Chute Spillway*

### 5.2.3 Side Channel Spillways

A Side Channel Spillway is one in which the control weir is placed approximately parallel to the upper portion of the discharge channel, as may be seen from the figure 5.3. The flow over crest falls into a narrow though opposite to the weir, turns an approximate right angle, and then continues into the main discharge channel. The side channel design is concerned only with the hydraulic action in the upstream reach of the discharge channel and is more or less independent of the details selected for the other spillway components. Flow from the side channel can be directed into an open discharge channel. Flow into the side channel might enter on one side of the trough in the case of a steep hill side location or on both sides and over the end of the trough if it is located on a knoll or gently sloping abutment.

Discharge characteristics of a Side Channel Spillway are similar to those of an ordinary Overflow Spillway and are dependent on the selected profile of the weir crest. Although the side channel is not hydraulically efficient, nor in expensive, it has advantage which make it adoptable to spillways where a long overflow crest is required in order to limit the afflux(surcharge held to cause flow) and the abutment are steep and precipitous.

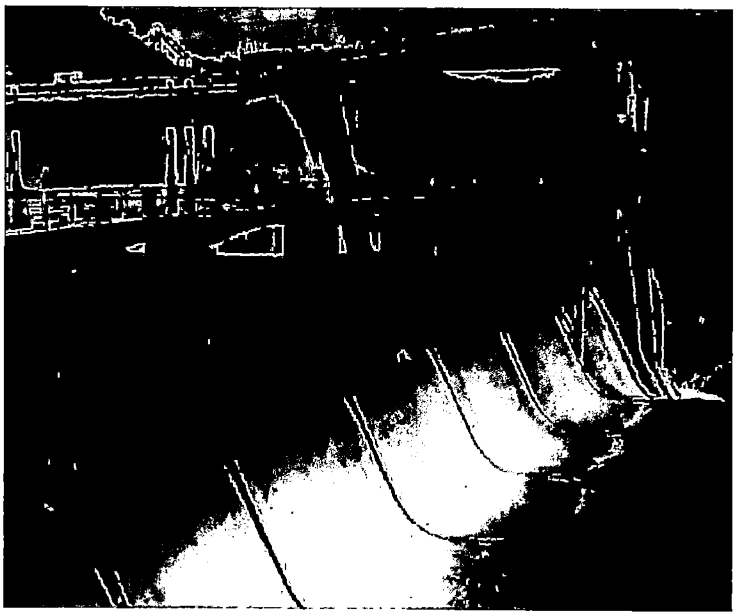


*Figure 5.3 Side Channel Spillway*

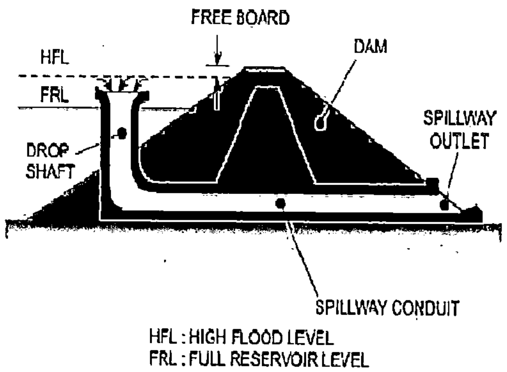
### 5.2.4 Shaft Spillways (Morning Glory Spillway)

a) This type of Spillways can be adopted very advantageously in dam sites in narrow canyons,

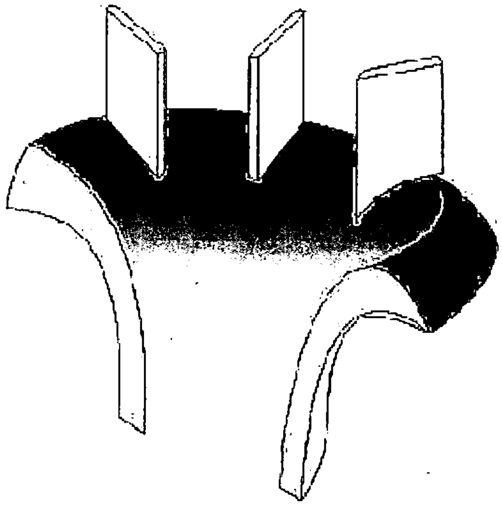
b) And Minimum discharging capacity is attained at relatively low heads.

This characteristic makes the spillway ideal where the maximum spillway outflow is to be limited. This characteristic becomes undesirable where a discharge more than the design capacity is to be passed. So, it can be used as a service spillway in conjunction with an emergency spillway. The structure may be considered as being made up of three elements, namely, an overflow control weir, a vertical transition, and a closed discharge channel. When the inlet is funnel shaped, the structure is called a Morning glory Spillway.

The factor limiting its adoption is the difficulty of air-entrainment in a shaft, which may escape in bursts causing an undesirable surging.




*(a) Section through Shaft Spillway*



*(b)Morning Glory Spillway with Anti-Vortex Piers*

*Figure 5.4*

### 5.2.5 Siphon Spillway

A siphon spillway is a closed conduit system formed in the shape of an inverted U, positioned so that the inside of the bend of the upper passageway is at normal reservoir storage level. Siphon spillways can be used to discharge full capacity discharges, at relatively low heads, and great advantage of this type of spillway is its positive and automatic operation without mechanical devices and moving parts. Siphon spillway comprise usually of five components, which include an inlet, an upper leg, a throat or control section, a lower leg and outlet.

The following factors limit the adoption of a siphon spillway:

It is difficult to handle flows materially greater than designed capacity, even if the reservoir head exceeds the design level; Siphon spillways cannot pass debris, ice, etc; There is possibility of clogging of the siphon passage way and breaking of siphon vents with logs and debris; In cold climates, there can be freezing inside the inlet and air vents of the siphon; When sudden surges occur and outflow stops; The structure is subject to heavy vibrations during its operation needing strong foundations; and Siphons cannot be normally used for vacuum heads higher than 8 m and there is danger of cavitations damage.

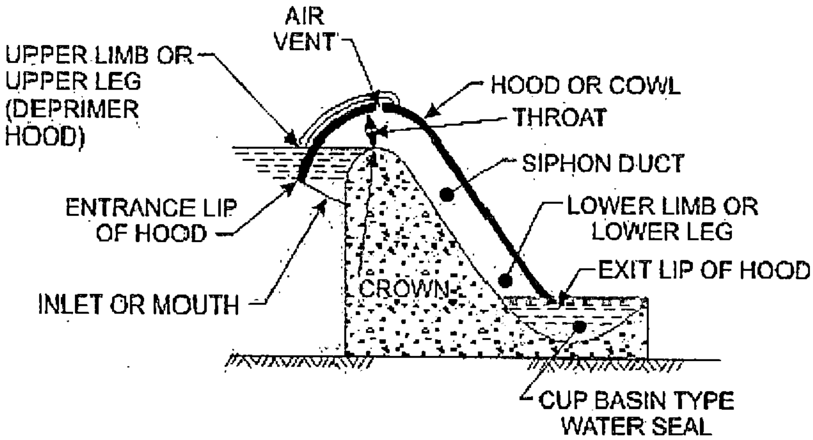
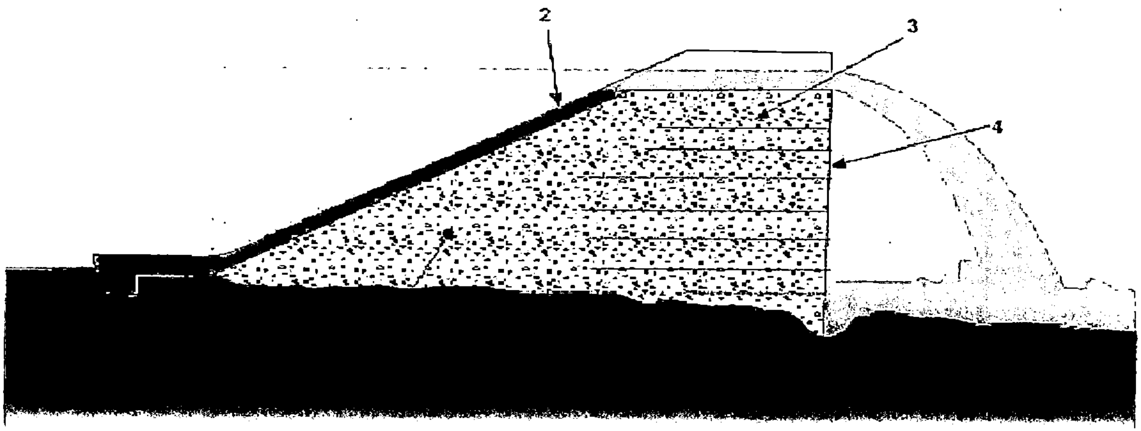



*Figure 5.5 Siphon Spillway*

### 5.2.6 Overfall or Free Fall Spillway

This is suitable for arch dams or dams with downstream vertical faces; and this is suitable for small drops and for passing any occasional flood.




LEGEND
1. RANDOM FILL 2. WATERTIGHT MEMBRANE 3. STEEL TENDONS 4. CONCRETE SLABS (1.5 M X 1.5 M).

*Figure 5.6 Free overall Spillway for a decked embankment Dam*

### 5.2.7 Tunnel or Conduit Spillway

This type is generally suitable for dams in narrow valleys, where overflow spillways cannot be located without risk and good sites are not available for a saddle spillway. In such cases, diversion tunnels used for construction can be modified to work as tunnel spillways. In case of embankment dams, diversion tunnels used during construction may usefully be adopted. Where there is danger to open channels from snow or rock slides, tunnel spillways are useful.

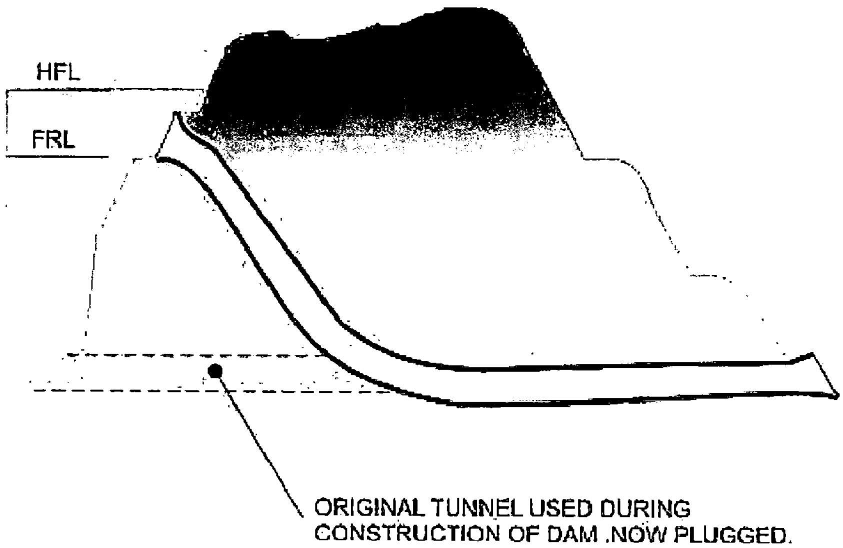



*Figure 5.7 Bell Mouth entry of a Tunnel Spillway*

## 5.3 HYDRAULIC DESIGN OF HIGH OGEE OVERFLOW SPILLWAYS

### 5.3.1 Ogee Spillway [25]

#### 5.3.1.1 Shape of the Profile

The ogee profile is divided into three groups as follows:

a. Spillway with vertical upstream face.
b. Spillway with sloping upstream face, and
c. Spillway with crest offsets and risers.

#### 5.3.1.2 Spillways with Vertical Upstream Face

a. Upstream profile

The upstream quadrant of the crest may conform to ellipse:

$$\frac{X^2}{A^2}+\frac{Y^2}{B^2}=1$$

Magnitudes of A and B are determined with reference to the parameter P/Hd from the graph. (IS: 6934)

b. Downstream profile

The downstream profile of the crest may conform to the equation:

$$X^{1.85} = K . H_d^{0.85} . Y$$

The magnitude of K is determined with reference to the parameter P/Hd from graph(fig2 IS code)

d. Upstream Face

In case of sloping upstream face, the desired inclination of the face is fitted tangential to the elliptical profile.

### 5.3.1.3 Discharge Computation

The charge over the spillway may be computed from the basic equation;

$$Q= 2/3\sqrt{2g}.\ C. L' H^{3/2}$$

$$C = 1.80\text{-}2.21 \text{ (S.I. Units)}$$

### 5.3.1.4 Effective length of overflow Crest

$$L= L' -2 H (N. K_p+ K_n)$$

## 5.4 CALCULATIONS FOR OGEE UNGATED SPILLWAY DESIGN

| Input data | |
|---|---|
| *Dam top* | *2601* |
| *Frl* | *2600* |
| *Mddl* | *2599* |
| *Spillway crest* | *2598* |
| *Tail water* | *2587* |
| *Bed level* | *2585* |
| *Q* | *150* |
| P | 15 |
| Hd | 2 |
| P/Hd | 7.5 |
| Cb | 2 |

Cb values has been taken from IS Code 6934

*Table 5.1 Dimensions for hydraulic design of Ungated Spillway*

| 'Hd | He | P/Hd | V^2/(2g) | Cb | L | N | Kp | Ka | L' |
|---|---|---|---|---|---|---|---|---|---|
| 1 | 2 | 7.5 | 0 | 2 | 26.5165 | 10 | 0.01 | 0.1 | 27.3165 |
| .05 | 2.1 | 7.5 | 0.1 | 2 | 24.6452 | 10 | 0.01 | 0.1 | 25.4852 |
| 1.1 | 2.2 | 7.5 | 0.2 | 2 | 22.98409 | 10 | 0.01 | 0.1 | 23.8640 |
| .15 | 2.3 | 7.5 | 0.3 | 2 | 21.50154 | 10 | 0.01 | 0.1 | 22.4215 |
| 1.2 | 2.4 | 7.5 | 0.4 | 2 | 20.17179 | 10 | 0.01 | 0.1 | 21.1317 |
| .25 | 2.5 | 7.5 | 0.5 | 2 | 18.97367 | 10 | 0.01 | 0.1 | 19.9736 |
| 1.3 | 2.6 | 7.5 | 0.6 | 2 | 17.88963 | 10 | 0.01 | 0.1 | 18.9296 |

**Profile of Spillway**

*Table 5.2 Profile Coordinates for Ogee Ungated Spillway.*

| U/S curve | |
|---|---|
| **A/Hd** | **0.28** |
| **A** | **0.56** |
| **B/Hd** | **0.165** |
| **B** | **0.33** |
| **X** | **Y** |
| 0 | 0 |
| -0.296363017 | 0.05 |
| -0.401576325 | 0.1 |
| -0.469358627 | 0.15 |
| -0.514716394 | 0.2 |
| -0.54329539 | 0.25 |
| -0.557681149 | 0.3 |
| -0.56 | 0.33 |

| U/S Face | |
|---|---|
| **X** | **Y** |
| **-0.56** | 0.33 |
| **-0.56** | 13 |

| D/S curve | |
|---|---|
| K2 | 2 |
| X | Y |
| 0 | 0 |
| 0.272302496 | 0.025 |
| 0.396068714 | 0.05 |
| 0.493122709 | 0.075 |
| 0.576088831 | 0.1 |
| 0.64993997 | 0.125 |
| 0.717255553 | 0.15 |
| 0.779581229 | 0.175 |
| 0.83793122 | 0.2 |
| 0.893014239 | 0.225 |
| 0.945348986 | 0.25 |
| 0.995328837 | 0.275 |
| 1.043260672 | 0.3 |
| 1.089389447 | 0.325 |
| 1.13391445 | 0.35 |
| 1.177000445 | 0.375 |
| 1.218785527 | 0.4 |
| 1.259386829 | 0.425 |
| 1.298904736 | 0.45 |
| 1.33742607 | 0.475 |
| 1.375026535 | 0.5 |
| 1.411772618 | 0.525 |
| 1.447723098 | 0.55 |
| 1.482930244 | 0.575 |
| 1.517440785 | 0.6 |
| 1.551296705 | 0.625 |
| 1.584535889 | 0.65 |
| 1.617192672 | 0.675 |
| 1.649298282 | 0.7 |
| 1.680881225 | 0.725 |
| 1.711967609 | 0.75 |
| 1.742581416 | 0.775 |
| 1.772744738 | 0.8 |
| 1.80247798 | 0.825 |
| 1.831800038 | 0.85 |
| 1.860728447 | 0.875 |
| 1.889279521 | 0.9 |
| 1.917468462 | 0.925 |
| 1.945309471 | 0.95 |
| 1.972815833 | 0.975 |
| 2 | 1 |
| 2.026873664 | 1.025 |
| 2.053447817 | 1.05 |
| 2.079732813 | 1.075 |
| 2.105738415 | 1.1 |
| 2.131473845 | 1.125 |
| 2.156947821 | 1.15 |
| 2.182168598 | 1.175 |
| **2.192114583** | **1.184926802** |

| D/S Face | |
|---|---|
| A | 1 |
| X | Y |
| 2.192114583 | 1.184926802 |
| 11.8150732 | 13 |

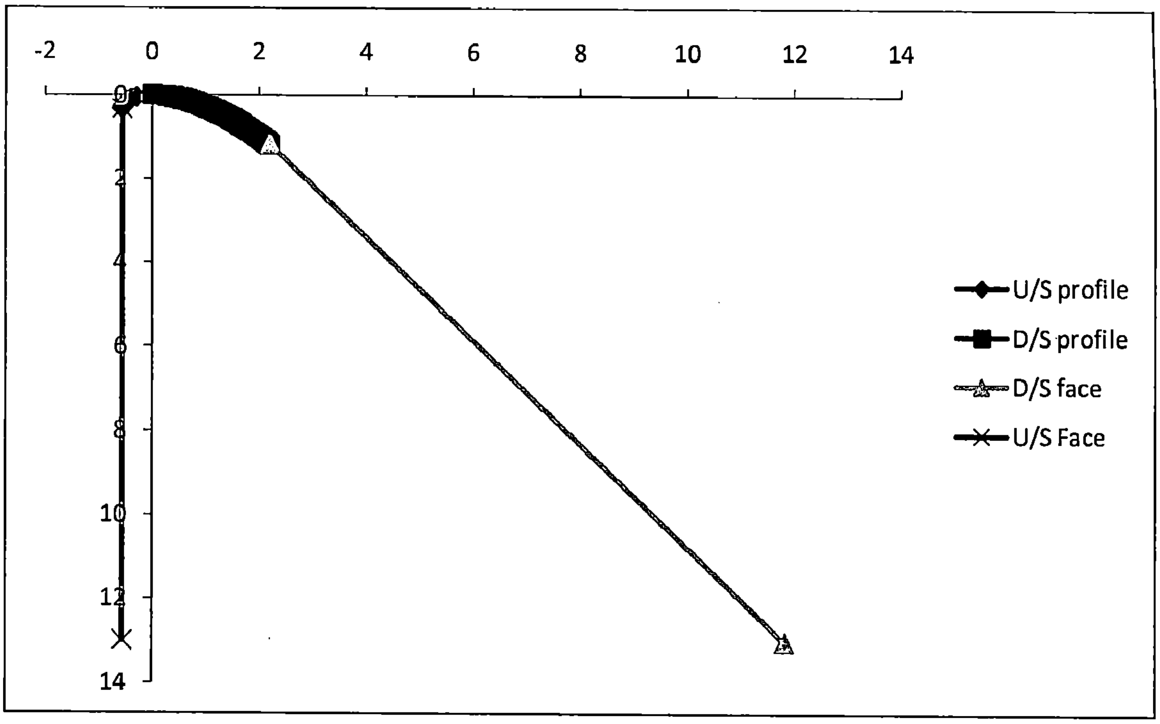



*Figure 5.8 Profile of Ogee Ungated Spillway*

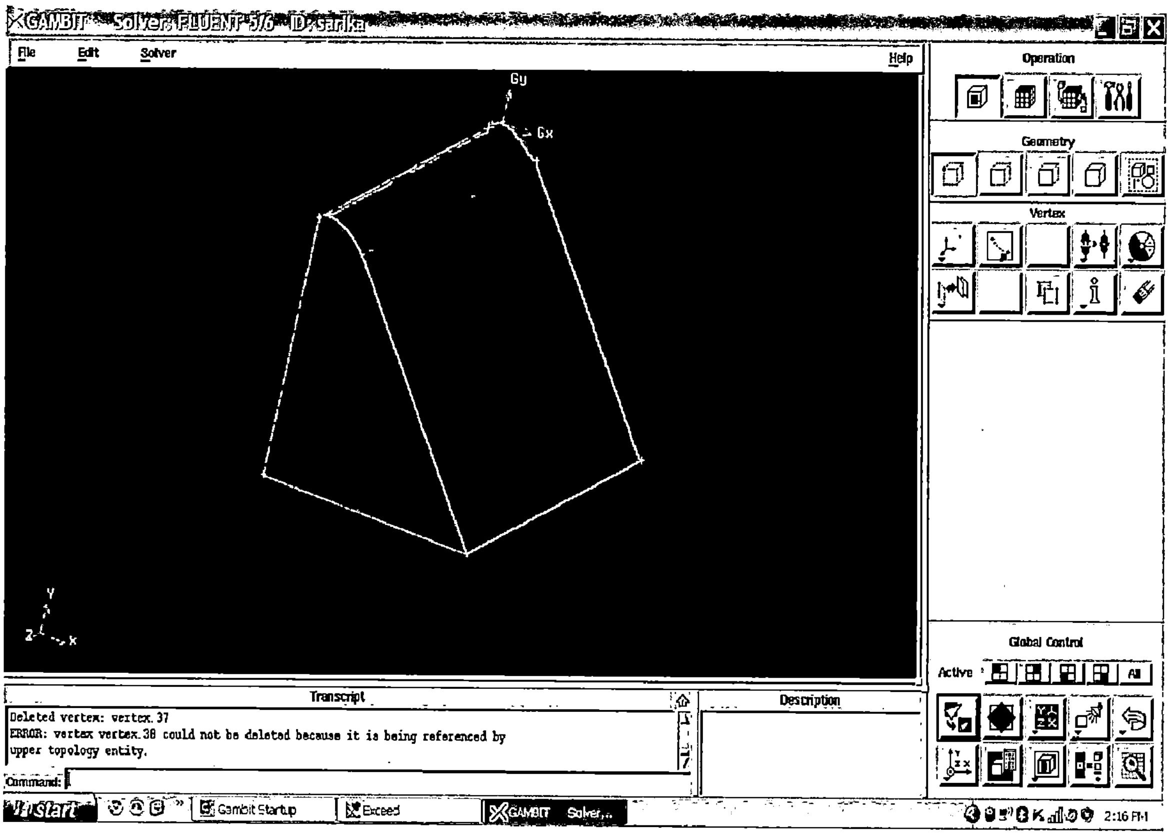



*Figure 5.9 3-D View of Ogee Ungated Spillway*

**RESULTS**

The Area under Curve has been calculated using GAMBIT Software.

| | |
|---|---|
| Area | 87.8317 $m^2$ |
| Volume | 2095.678 $m^3$ |
| Rate of Concrete [25] | Rs. 3629 per $m^3$ |
| Cost of Ogee Ungated Spillway | Rs. 7605215.4 |

## 5.5 CALCULATIONS FOR OGEE GATED SPILLWAY DESIGN

### 5.5.1 Shape of the profile

When spillways are equipped with gates, discharges for partial gate opening will occur as orifice flow. With full head on the gate and with gate partially opened the jet emerging from the gate will be in the form of a trajectory conforming to a parabola.

$$X^2 = 4HY$$

### 5.5.2 Discharge Computation

$$Q = C_g G_0 L \sqrt{2g} H_c$$

**Input data**

| | |
|---|---|
| Dam top | 2601 |
| Frl | 2600 |
| Mddl | 2599 |
| Spillway crest | 2598 |
| Tail water | 2587 |
| Bed level | 2585 |
| Q | 150 |
| P | 15 |
| Hc | 1.5 |
| G0 | 1 |

*Table 5.3 Dimensions for hydraulic design of Gated Ogee Spillway.*

| B | Cg | Hc | L | N | Kp | Ka | L' | Gate |
|---|---|---|---|---|---|---|---|---|
| 50 | 0.67 | 1.5 | 33.6957 | 10 | 0.01 | 0.1 | 34.2957 | 3.42957 |
| 60 | 0.68 | 1.5 | 33.2002 | 10 | 0.01 | 0.1 | 33.8002 | 3.38002 |
| 70 | 0.685 | 1.5 | 32.9579 | 10 | 0.01 | 0.1 | 33.5579 | 3.35579 |
| 80 | 0.69 | 1.5 | 32.7191 | 10 | 0.01 | 0.1 | 33.3191 | 3.33191 |
| 90 | 0.71 | 1.5 | 31.7974 | 10 | 0.01 | 0.1 | 32.3974 | 3.23974 |
| 100 | 0.74 | 1.5 | 30.5083 | 10 | 0.01 | 0.1 | 31.1083 | 3.11083 |

**Profile of Spillway**

*Table 5.4 Profile Coordinates for Ogee Gated Spillway.*

| U/S Face | |
|---|---|
| **X** | **Y** |
| **-0.42** | **0.2475** |
| **-0.42** | **13** |

| U/S curve | |
|---|---|
| **P/Hc** | **10** |
| **A/Hd** | **0.28** |
| **A** | **0.42** |
| **B/Hd** | **0.165** |
| **B** | **0.2475** |
| **X** | **Y** |
| 0 | 0 |
| -0.253127363 | 0.05 |
| -0.337266057 | 0.1 |
| -0.386037295 | 0.15 |
| -0.412192507 | 0.2 |
| -0.42 | 0.2475 |

| D/S curve | |
|---|---|
| **X** | **Y** |
| 0 | 0 |
| 0.547722558 | 0.05 |
| 0.774596669 | 0.1 |
| 0.948683298 | 0.15 |
| 1.095445115 | 0.2 |
| 1.224744871 | 0.25 |
| 1.341640786 | 0.3 |
| 1.449137675 | 0.35 |
| 1.549193338 | 0.4 |
| 1.643167673 | 0.45 |
| 1.732050808 | 0.5 |
| 1.816590212 | 0.55 |
| 1.897366596 | 0.6 |
| 1.974841766 | 0.65 |
| 2.049390153 | 0.7 |
| 2.121320344 | 0.75 |
| 2.19089023 | 0.8 |
| 2.258317958 | 0.85 |
| 2.323790008 | 0.9 |
| 2.387467277 | 0.95 |
| 2.449489743 | 1 |
| 2.50998008 | 1.05 |
| 2.569046516 | 1.1 |
| 2.626785107 | 1.15 |
| 2.683281573 | 1.2 |
| 2.738612788 | 1.25 |
| 2.792848009 | 1.3 |
| 2.846049894 | 1.35 |
| 2.898275349 | 1.4 |
| 2.949576241 | 1.45 |
| 3 | 1.5 |

| D/S Face | |
|---|---|
| **A** | **1** |
| **X** | **Y** |
| **3** | **1.5** |
| **11.5** | **13** |

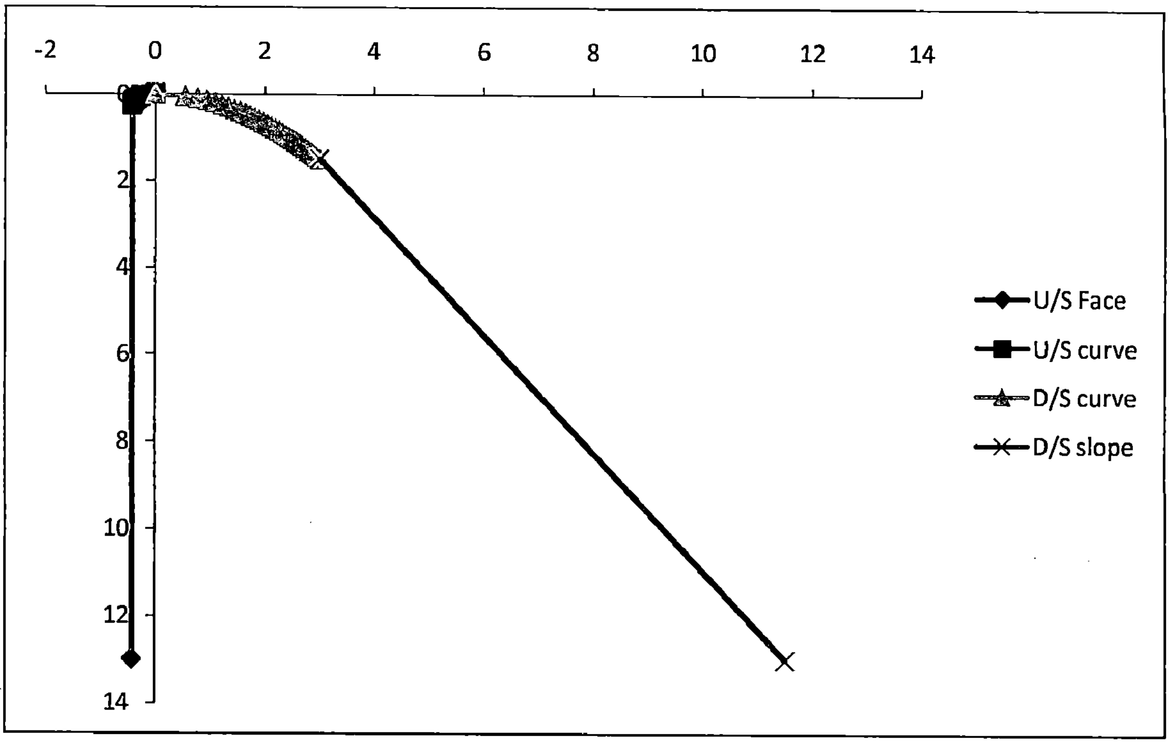



*Figure 5.10 Profile of Ogee Gated Spillway*

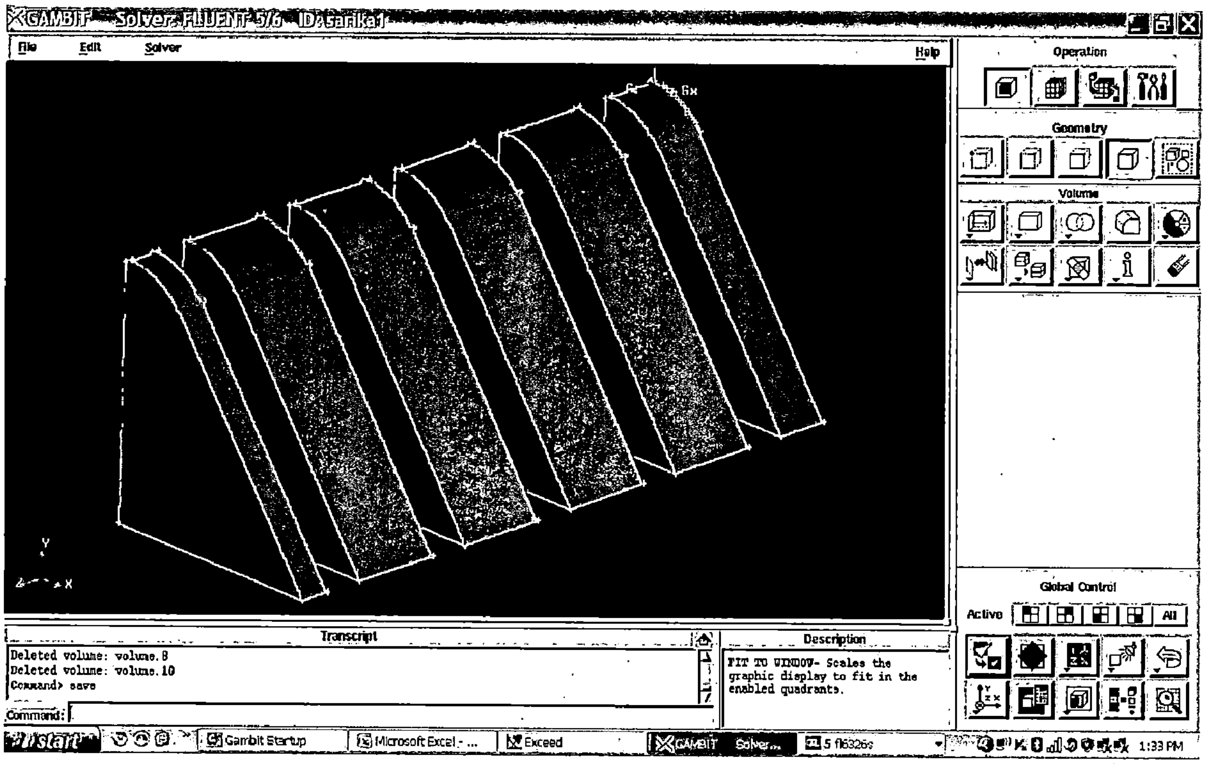



*Figure 5.11 3-D view of Ogee Gated Spillway*

**RESULTS**

| | |
|---|---|
| Area | 91.794693 $m^2$ |
| Volume | 3003.4397 $m^3$ |
| Total Gates 5 | 2m Width |
| Area of gate | 20 $m^2$ |
| Volume of Gate | 60m3 |
| Area of Concrete | 71.794693 $m^2$ |
| Volume of Concrete | 2369.224 $m^3$ |
| Rate of Concrete | Rs. 3629 per m3 |
| Rate of Steel | Rs. 40 per kg |
| Density of steel | 7860 $kg/m^3$ |
| Cost of Ogee Gated Spillway | Rs. 27461986.48 |
| | |

## 5.6 CALCULATIONS FOR OGEE WITH BREAST WALL SPILLWAY DESIGN

Spillways are sometimes provided with breast wall from various considerations such as increasing the regulating storage of flood discharge, reducing the height of gate, minimizing the cost of gate operating mechanism, etc

For the spillway with breast wall, the following parameters are required to be determined:

a) Profile of the spillway crest including the upstream and downstream quadrants,
b) Profile of the bottom surface of the breast wall, and
c) Estimation of discharge efficiency of the spillway.

### 5.6.1 Ogee Profile- upstream Profile

$$\frac{X^2}{A^2}+\frac{Y^2}{B^2}=1$$

$$A = 0.541\, D\, (H_d/D)^{0.32}$$

$$B = 0.3693\, D\, (H_d/D)^{0.04}$$

### 5.6.2 Downstream Profile

$$X^n = K \cdot H_d^{n-1} \cdot Y$$

Where,

$$K = 0.44 - 0.025\left(\frac{Hd}{D}\right) \text{ and}$$

$$n = 1.782 - 0.0099\left(\frac{Hd}{D} - 1\right)$$

### 5.6.3 Bottom Profile

The bottom profile of the breast wall may conform to the equation:

$$X = \frac{K}{n^{2.4}} Y^{2.4}$$

Where

$$K = 0.541\ D\ (H_d/D)^{0.32}, \text{ and}$$

$$n = 0.4\ D$$

### 5.6.4 Discharge Computation

The discharge through the breast wall spillway may be estimated by the equation:

$$Q = Cb.\ L.\ D.\ \{2g\ (H_c + V^2/2g)\}^{0.5}$$

**Input data**

| | |
|---|---|
| Dam top | 2601 |
| Frl | 2600 |
| Mddl | 2599 |
| Spillway crest | 2598 |
| Tail water | 2587 |
| Bed level | 2585 |
| Q | 150 |
| P | 15 |
| Hd | 2 |
| P/hd | 7.5 |
| D | 1 |

*Table 5.5 Dimensions for hydraulic design of Ogee Spillway with breast wall.*

| [d | He | P/Hd | V^2/(2g) | Cb | Hc | L | N | Kp | Ka | L' |
|---|---|---|---|---|---|---|---|---|---|---|
| | 2 | 7.5 | 0 | 0.767698 | 1.5 | 36.01685 | 10 | 0.01 | 0.1 | 36.816 |
| [illegible] | 2.1 | 7.5 | 0.1 | 0.781524 | 1.5 | 34.25623 | 10 | 0.01 | 0.1 | 35.096 |
| | 2.2 | 7.5 | 0.2 | 0.793719 | 1.5 | 32.72282 | 10 | 0.01 | 0.1 | 33.602 |
| [illegible] | 2.3 | 7.5 | 0.3 | 0.804282 | 1.5 | 31.3832 | 10 | 0.01 | 0.1 | 32.303 |
| | 2.4 | 7.5 | 0.4 | 0.813214 | 1.5 | 30.21064 | 10 | 0.01 | 0.1 | 31.170 |
| [illegible] | 2.5 | 7.5 | 0.5 | 0.820515 | 1.5 | 29.18368 | 10 | 0.01 | 0.1 | 30.183 |
| | 2.6 | 7.5 | 0.6 | 0.826185 | 1.5 | 28.2849 | 10 | 0.01 | 0.1 | 29.324 |

**Profile of Spillway**

*Table 5.6 Profile Coordinates for Ogee with Breast Wall Spillway*

| U/S Face | |
|---|---|
| **X** | **Y** |
| **-0.675346827** | **0.379682436** |
| **-0.675346827** | **13** |

| U/S curve | |
|---|---|
| **A** | **0.675346827** |
| **B** | **0.379682436** |
| **X** | **Y** |
| 0 | 0 |
| -0.334985646 | 0.05 |
| -0.456740223 | 0.1 |
| -0.537762722 | 0.15 |
| -0.594934219 | 0.2 |
| -0.634732676 | 0.25 |
| -0.660306959 | 0.3 |
| -0.673279924 | 0.35 |
| -0.675346827 | 0.379682436 |

| D/S curve | |
|---|---|
| **K** | **0.39** |
| **n** | **1.7721** |
| **X** | **Y** |
| 0 | 0 |
| 0.146629311 | 0.05 |
| 0.216816683 | 0.1 |
| 0.27255954 | 0.15 |
| 0.281549845 | 0.158879208 |
| | |

| D/S Face | |
|---|---|
| **A** | **1** |
| **X** | **Y** |
| **0.281549845** | **0.158879208** |
| **12.84112079** | **13** |

**Breast Wall**

| | |
|---|---|
| K1 | 0.675346827 |
| n1 | 0.4 |

| X | Y | Y' |
|---|---|---|
| 0 | 0 | -1 |
| -0.004593153 | 0.05 | -1.05 |
| -0.024242806 | 0.1 | -1.1 |
| -0.064150775 | 0.15 | -1.15 |
| -0.127954297 | 0.2 | -1.2 |
| -0.218594336 | 0.25 | -1.25 |
| -0.338589819 | 0.3 | -1.3 |
| -0.490169394 | 0.35 | -1.35 |
| -0.675346827 | 0.4 | -1.4 |
| -0.675346827 | 2 | -3 |

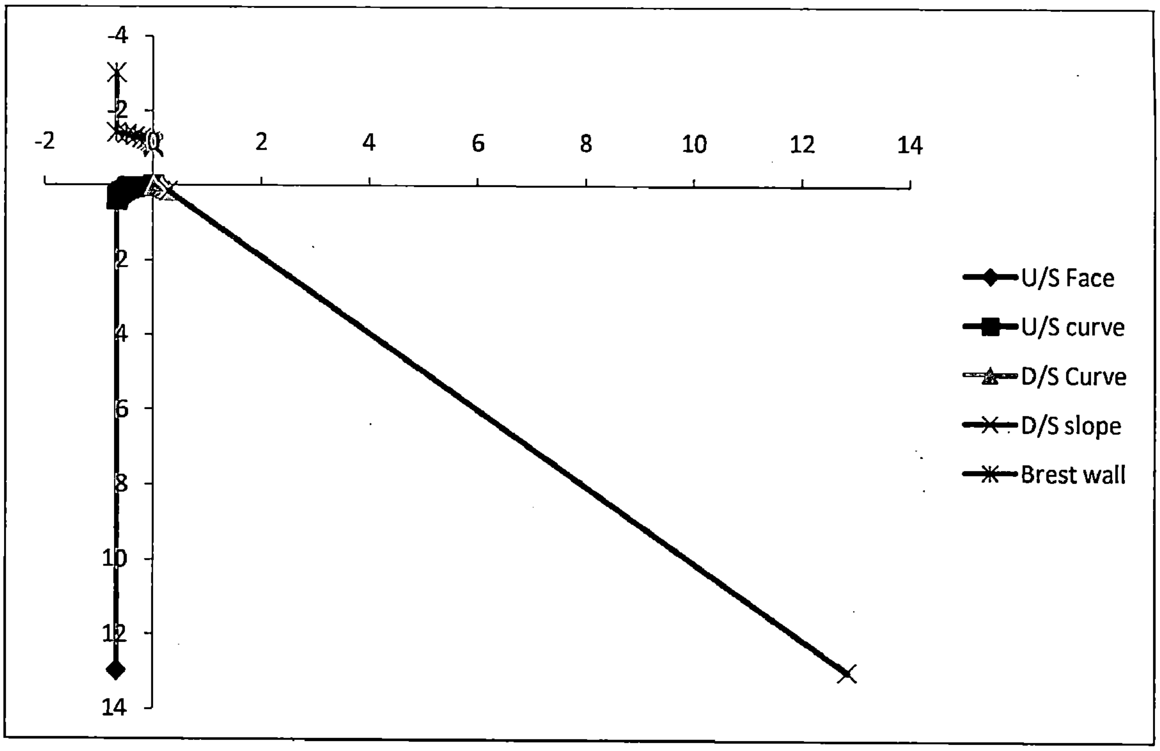



*Figure 5.12 Profile of Ogee with Breast Wall Spillway*

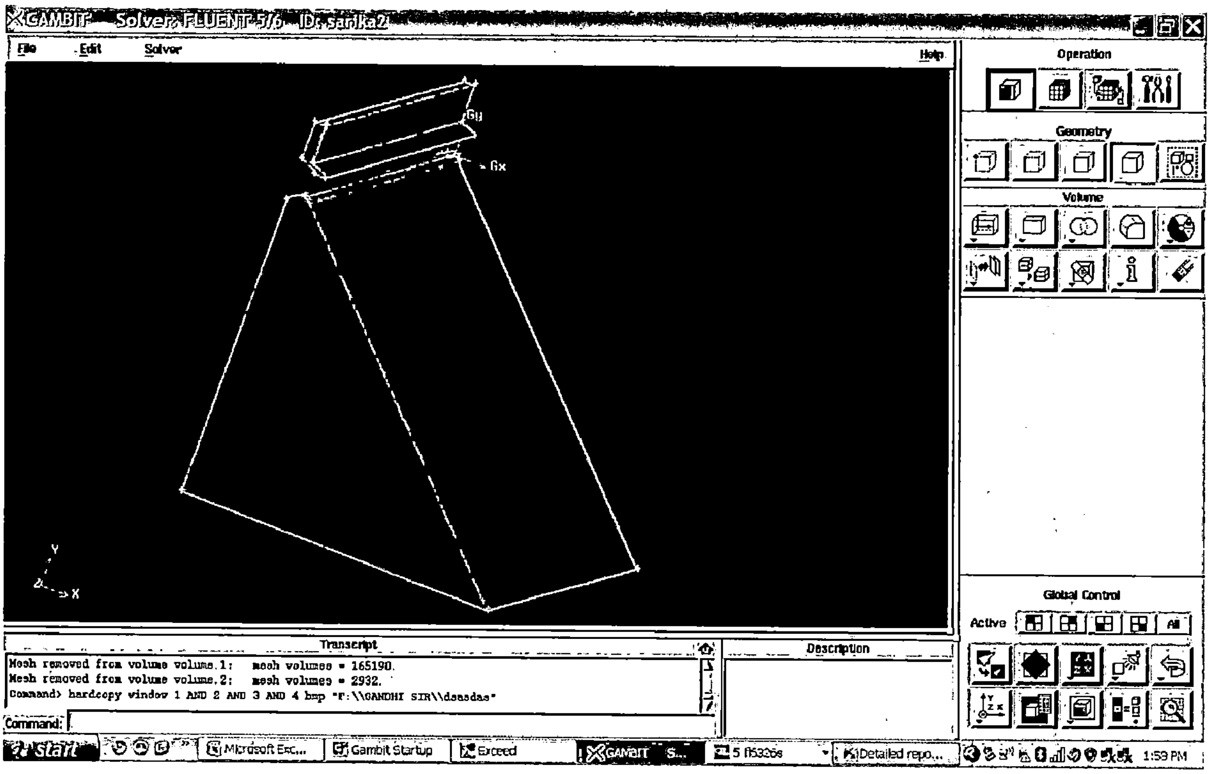



*Figure 5.13 3-D view of Ogee Spillway with Breast Wall*

**RESULTS**

| | |
|---|---|
| Area | 93.975945 $m^2$ |
| Volume | 3007.2075 $m^3$ |
| The Rate of Concrete Structure | Rs. 3629 per $m^3$ |
| Cost of Ogee Spillway with Breast Wall | Rs. 10913156.02 |

Graphical representation cost of different types of spillways is as shown in fallowing figure 5.14 from this graph it is conclude that the cost of ungated ogee spillway is less than cost of ogee spillway with breast wall and cost of ogee gated spillway. The cost of Ogee gated spillway is higher because it involves the cost of steel material which is required for gate structure.

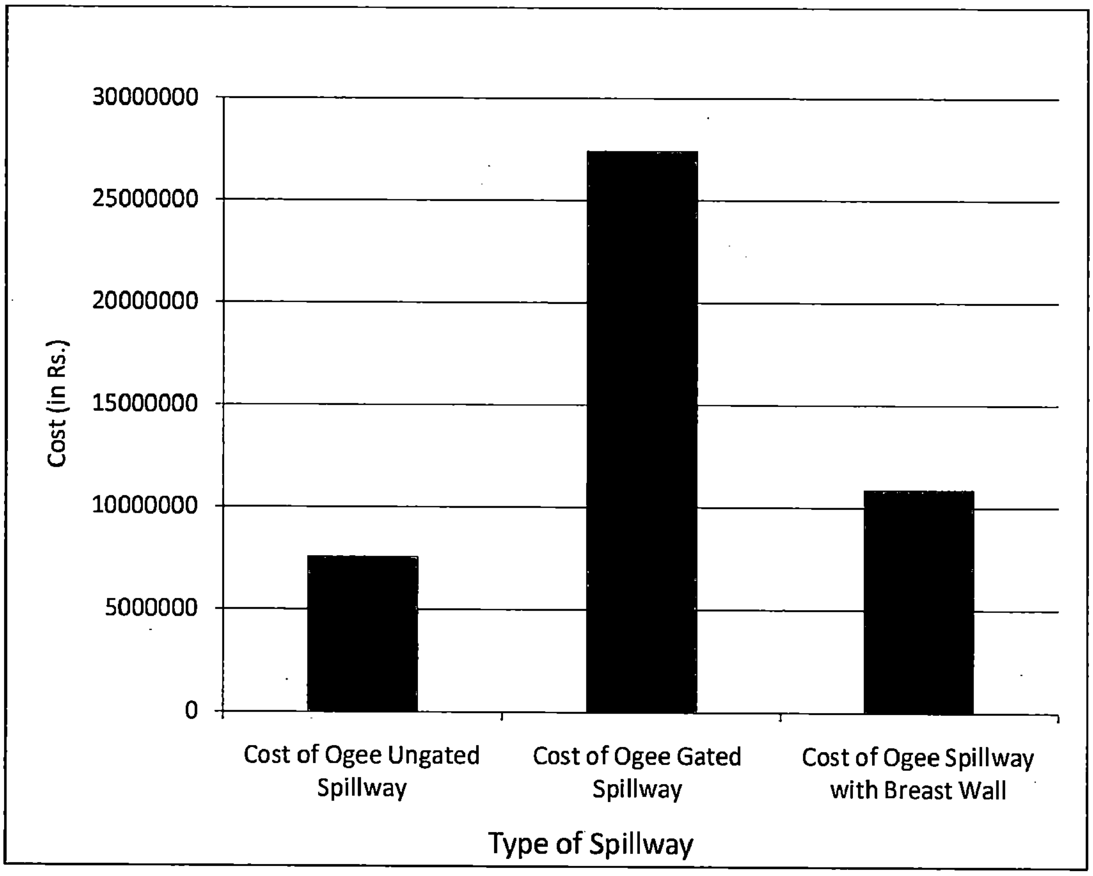



*Figure 5.14 Cost of Spillway (in Rs.)*

# CHAPTER 6

# CONCLUSION

Cost effectiveness of Small Hydro Power schemes depends on proper selection of site, optimum planning of the scheme, standard designs, use of appropriate construction techniques and equipments suitable to small hydro power and efficient execution.

For different head and capacity combinations various layouts were studied. Following are the main conclusions drawn from the study.

1. The installation cost of small hydro project mainly depends on the civil work components as well as electromechanical equipment, being large physical sizes.
2. The sizing of civil works has been determined based on hydraulic design of the components. Computer software in 'C' program has been developed for sizing the components which will be useful to quick design of components.
3. It has been found that power house building contribute maximum with cost of the project. In a typical layout of 5000 kW capacity at 5m head, contribution of power house such has been found 20.32 % of the total cost.
4. To determine cost of power house building, computer program has been developed based on type of turbine, head, capacity, specific speed and number of turbines. This can be utilized to planning of new projects for cost estimation.
5. It has been found that the layout of a run-of-river scheme with semi Kaplan tubular type gives minimum cost. The cost comes out to Rs 99.81 millions for power house building in a layout of 5000kW at 5m head.
6. In dam toe scheme, spillway is an important component. The design of spillway has been carried out using IS Code and cost has been computed using GAMBIT software.

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