TRIBHUVAN UNIVERSITY
INSTITUTE OF ENGINEERING
ADVANCED COLLEGE OF ENGINEERING & MANAGEMENT
Kupandole, Lalitpur

A
PROJECT REPORT
ON
HEWA KHOLA-B
SMALL HYDROPOWER PROJECT

SUBMITTED TO
DEPARTMENT OF CIVIL ENGINEERING
ADVANCED COLLEGE OF ENGINEERING AND MANAGEMENT

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PREFACE

To introduce the students with the real civil engineering practice and to give them confidence, ability to tackle problems related to civil engineering and idea of practical working in professional field with the application of theoretical knowledge gained during the whole four years, there is a provision of project work in the syllabus of TU.IOE on the final semester of bachelor’s degree program. This project entitled “Pre-feasibility Study of Hewa Khola-B small hydropower project” is the one prepared by a group of six students in partial fulfillment of the requirement for the Bachelor’s degree in Civil Engineering subject entitled "CIVIL ENGINEERING PROJECT (EG777CE)" in Second Semester, Fourth Year.

Hydropower engineering includes great diversified nature of work from meteorological analysis to geological study, civil engineering structures, electromechanical installation, operation etc. In order to complete this project, the period of one semester inclusive of the regular classes and timely assessments is very difficult. However every effort has been made to collect the most reliable data, past reports and relevant design information.

From the very beginning of the project, from the hydrological analysis to hydraulic design and then to electro-mechanical components design every attempt have been made to cover all the parts of a hydropower plant. This project group is sure that this report will be beneficial for the detail investigation and design of the Hewa Khola-B Small Hydropower Project. The group will also be delighted for any feedback and suggestion to upgrade this report.

NOVEMBER, 2011 PROJECT GROUP

ACKNOWLEDGEMENTS

It is our great pleasure to submit this report on "Pre-feasibility Study on Hewa Khola-B Small Hydro Power Project" to the Department of Civil Engineering in partial fulfillment of the requirement for the Bachelor's degree in Civil Engineering.

We would like to express our sincere gratitude and thanks to our esteemed supervisor Er. British Singh for providing his valuable, remarkable support and kind cooperation along with positive suggestions and guidance.

We would like to extend our deep appreciation and acknowledgement to Project Coordinator Er. Rajesh Khadka, Head of Department Er. Laxmi Bhakta Maharjan, Principal, Associate Professor Dr. Prem Chandra Jha and others who are related to us for their kind support and precious knowledge with cooperation.

Likewise, we would like to extend special thanks to our admired friends in our class and others for their remarkable help and kind support. We are also obliged to all our family, friends and colleagues for their encouragements to complete this Project Work.

Finally, last but not the least, we express our sincere thanks to all of our well wishers.

NOVEMBER, 2011 PROJECT GROUP

EXECUTIVE SUMMARY

Hewa Khola-B project encompasses the Bharpa and Nagin VDC of Panchathar District, Mechi Zone of Nepal. Hewa Khola is a snow fed river and the hydropower project is a run-off river type.The elevation of the headworks will be about 670m from the mean sea level.Gravel trap will locate adjacent to the intake. 60m run from headworks, settling basin will be constructed. After two kilometer run, forebay pond will be made. With 65.45m gross head and 55.63m net head power house will be located in 605m elevation from the mean sea level. Hewa Khola-B small hydropower project will have installed capacity of **3.8 MW** and will produce total energy as **16.08 GWh** (Dry energy **0.48 GWh** and Wet energy **15.6 GWh**) from two Francis turbines of **1.9 MW** capacities each.The structure can pass flood discharge of 100 years return period (372 Cumecs) safely. About 2.7 Km of access road is necessary to upgrade for the transportation facility.

The project will have base period of 3 years and payback period of 7.5 years. The B/C ratio and IRR of the project will have 1.03 and 13% respectively with total project cost 7,877,000 US$. With respect to the economic analysis the proposed project is technically feasible, economically viable and environmentally acceptable.

CONTENTS

Salient Features

The salient features of the Hewa Khola is presented herein

SN.	Description	Parameters
1	**Project Name**	Hewa Khola-B Hydropower Project
2	**Location**	
	Latitude	27º 10' 40" to 27º 09' 39"
	Longitude	87º 47' 42" to 87º 46' 10"
	VDC	Bharpa and Nagin VDCs
	District	Panchathar
3	**Type of power plant**	
	Type	Snow Fed type Run off river
4	**Hydrology**	
	Catchment area at intake site	221 km^2
	annual average flow	11.87 m^3/s (WECS)
	average minimum 1 in 2 year flow(monthly)	2.17 m^3/s (WECS)
	design flood at intake (1 in 100 yrs)	372 m^3/s (WECS)
5	**Diversion weir**	
	Type	Semi-Permanent Boulder lining diversion weir
	Crest level	668m
	Length	16.25 m
	Height	3 m above natural bed level
6	**Intake**	
	Type	side intake fitted with 2 numbers of mechanised gate
	Size of opening	2×1m clear opening
	Intake invert level	666 m
7	**Approach Canal**	
	Type	Concrete lined rectangular open channel
	Length	60 m
	Width	2.5 m
	Height	1.8 m
	Bed slope	1:750
8	**Settling Basin**	
	No of bays	2 nos
	Nominal size of trapped particles	0.2 mm, 90% of the particle size to be settled
	Length	
	Inlet and outlet transition	36.5 m, 15m
	Uniform sections	66.5 m
	Average depth	5 m
	Invert slope	1:80
	Width	17.2m
	Flushing channel	0.5×0.5 m
9	**Forebay**	
	Surface area	21m×15m×4.15m (L×B×H)
	Depth	4 m
	Lining type	Reinforced concrete lining
	Flushing	Gated flushing arrangement
	Normal operating level	661.9 m

10	**Penstock**	
	Type	Surface type
	Material	Steel pipe
	Numbers	1
	Diameter	1970 mm
	Thickness of pipe	8 mm thick, welded metal strap
	Length	133 m
	Anchor blocks	4X6X3.75 m
11	**Powerhouse**	
	Type	Surface
	size	18mX6mX11.5m (LXBXH)
	Gross head	65.45 m
	Net head	55.63m
	Design flow	7.8m^3/sec
	Capacity	3.8 MW
12	**Tailrace canal**	
	Shape	Rectangular
	Length	≈ 25 m
	Cross-section area	1.5X2.5 m^2
	Bed slope	1:500
13	**Turbines**	
	Type	Horizontal Francis type
	Number of units	2 nos each of 1.9 MW capacity
14	**Generators**	
	Type	Synchronous
	capacity	4.75 MVA
	Voltage	6.6 KV
15	**Transmission line**	
	Length	≈ 2 Km
	Voltage	132 KV (Upper Hewa)
16	**Transformer**	
	type	3 phase, oil immersed
	Rating	5 MVA
	Power factor	0.8
	Frequency	50 Hz
17	**Energy generation**	
	Mean annual energy per year	16.08 GWH
	Dry energy	0.48 GWH
	Wet energy	15.60 GWH
18	**Access road**	
	Availability	4 Km from the Mechi Rajmarga
	Proposed road length	≈ 800 m
	Type	Gravel road single lane
19	**Construction period**	
	Construction period from award of civil contract	3 years
20	**Economic indicators**	
	Project cost	NRs. 59,07,78,930
	Cost per KW	NRs. 1,55,468
	Internal rate of return (IRR)	13%
	B/C ratio at discounted rate of 10%	1.03
	Payback period	7.5 years.

1.0 INTRODUCTION

1.1 BACKGROUND

Hydropower is the source of renewable energy formed by the movement of flowing mass of water on the surface of the earth with the help of positional difference. Water resource is a major source for the economic development of the country through the development of hydropower and other multipurpose projects.

Nepal has 83000MW total hydropower potential out of which 44000 MW is technically feasible and about 42000MW is economically viable. The advent of small hydropower development in Nepal was Pharping Hydropower station in 1911 B.S with an installed capacity of 500KW as a first station in Nepal knowing immense importance of hydropower to fulfill the energy crisis, Nepal has established several programs related to energy and power under government and private sectors such as Nepal Electricity Authority (NEA), Water and Energy Commission Secretariats (WECS), Ministry of Energy, Department of Electricity Development (DoED), Alternative Energy Promotion Centre (AEPC) etc.

1.2 HYDROPOWER DEVELOPMENT IN NEPAL

Nepal, being a developing country, is facing a lot of challenges to raise its economic status. To achieve the sustainable development of any country, it is necessary to use its available natural resources. Nepal is endowed with rich hydropower resources which is the major source of renewable energy. Hence the major achievements in the socio-economic development of Nepal could be possible through power harnessing of the water resource.

First approach in hydropower development in Nepal was the power generation from the construction of Pharping Hydropower station (500 KW) in 1911. But the progressive development was gradual only after the Sundarijal (600 KW) and Panauti (2400 KW) Hydropower Stations came into operation after long interval of 23 and 29 years.

The completion of Dhankuta Hydropower station (240 KW) in 1971 was regarded as the bench mark of small hydel development of Nepal. The establishment of small hydel development board in 1975 was another milestone under which several small hydro schemes such as Jhupra (345 KW), Doti (200 KW), Jumla (200 KW) etc. were made during 1975 to 1985. Nepal Electricity Authority (NEA), established 1985, responsible for generation, transmission and distribution of electric power brought the revolution in hydropower development. Many potential sites for hydropower generation had identified by private consultancies and companies in collaboration with NEA.

Prior to 1960, all the hydropower stations were constructed through grant aid from friendly countries like the USSR (Panauti), India (Trishuli, Devighat, Gandak, Surajpura- Koshi) and China (Sunkoshi). Since 1970, hydropower development took a new turn with the availability of bilateral and multilateral funding sources.

From 1990s, subsequent to the adoption of the policy of economic liberalization, hydropower development took yet another turn with the private sector entering the arena. After formulating Hydropower Development Policy – 1992 by government of Nepal, many private sectors are involving towards power development. In order to encompass projects of various scales intended for domestic consumption as well as to export hydropower, the former policy was replaced by the **Hydropower Development Policy 2001** to provide further impetus to active participation of private sectors.

Development of hydropower in Nepal is a very complex task as it faces numerous challenges and obstacles. Some of the factors attributed to the low level of hydropower development are lack of capital, high cost of technology, political instability, and lower load factors due to lower level of productive end-use of electricity and high technical and non technical losses.

Legends for the Power Development in Nepal

Major Hydropower Plants

Name	Capacity(MW)	Name	Capacity MW)
Trishuli	24.00	Gandak	15.00
Sunkoshi	10.05	Devighat	14.10
Kulekhani 1	60.00	Khulekhani- 2	32.00
Marsyandi	69.00	Upper Modi (Gitec)	14.00
Khimiti Khola (HPL)	60.00	Jhimruk (BPC)	12.30
Botekoshi (BPKC)	36.00	Kaligandaki (A)	144.00
Chilime (CPC)	20.00		

Some small project plants

Name	Capacity(MW)	Name	Capacity MW)
Tatopani, Myagdi	2.00	Panauti	2.40
Seti, Pokhara	1.50	Phewa, Pokhara	1.088
Hewa, Butwal	1.024	Chatara	3.20
Andhikhola(BPC)	5.10	Indrawati (NHPC)	7.50
Piluwa Khola(AVHP)	3.00	Sunkoshi (Sanima)	2.60

Planned & Proposed

Name	Capacity(MW)	Name	Capacity MW)
Rawa Khola	2.30	Molung Khola	1.20
Naugargad (Darchula)	1.80	Gandigad (Doti)	1.80
Khudi (KHL)	3.50	Mailung (MPC)	5.00
Daram Khola (GHP)	5.00	Upper Khimti	4.00
Chaku Khola (A. Power)	1.50	Lower Indrawati (SH)	4.60
Thoppal Khola	1.40	Mardi Khola	1.40
Lower Nayagdi (BHN)	4.50		

1.3 HYDROPOWER POTENTIAL OF NEPAL

The kingdom of Nepal, lying between India and China against the impressive Himalayas, comprises of the most diverse climatic ranges and physical environment in the world. From the Gangetic plains at about 70m altitude, to the Mt. Everest at 8,848 m altitude, there is only the distance of about 170 km. These slopes are the steepest slopes in the world resulting high hydropower potential. Because of the existence of snow feed perennial rivers, several tributaries and countless streams, Nepal, is considered as the World's 2^{nd} richest country in the gross hydropower potential.

Gross hydropower potential of Nepal is 83,000 MW out of which about 42,000 MW is assessed to be economically feasible and 44,000 is technically feasible. Approximately 6000 big and small rivers have been identified in Nepal's territory carrying about $174\times10^9 m^3$ of surface run-off annually (0.5% of total surface run off of the world)

Hydropower Potential of Nepal (in million KW) Source: Water Resources in Nepal, C. K. Sharma

S.N.	River Basins	Theoretically feasible	Technically feasible	Economical feasible
1	Saptakoshi	22.35	11.40	10.48
2	Karnali	34.60	24.36	24.00
3	Gandaki	17.95	6.73	6.27
4	Mahakali	1.58	1.13	1.13
5	Others	3.07	0.98	0.98
	Total	**83.29**	**44.60**	**42.15**

1.4 CLASSIFICATION OF HYDROPOWER PROJECTS

As per Nepal Electricity Authoricity (NEA) hydropower projects are catagorised as follows;

i. Micro Hydro Power Plant : Less than 100 KW
ii. Mini Hydro Power Plant : 100 KW – 1MW
iii. Small Hydro Power Plant : 1MW – 10 MW
iv. Medium Hydro Power Plant : 10 MW – 300 MW
v. Large Hydro Power Plant : More than 300 MW

Based on the above classifications Hewa Khola-B is small hydropower project since its installed capacity is 3.2MW. A small hydropower plant is found to be most feasible than both the micro hydro and large hydropower in context of Nepal. For small hydropower project head and discharge is easily available than the other hydro electric project. Investment required for small hydro is affordable to the countries like Nepal.

1.5 POWER SITUATION IN NEPAL

The total energy consumption in Nepal is about 7008GWh. Out of which about 65 % is produced from NEA hydro power plants, about 0.2% is produced from NEA thermal power plants and 6% is borrowed from Indian State Electricity. Alternative sources of energy, like, solar power is also contributing but to smaller scale. After the formulation of Hydropower policy 1992, private sectors are also allowed to participate in the development of new hydropower plants, 28% energy is being produced from the private sectors. Total system installed capacity is now 615 MW.

While analyzing regional balance of the power projects, most of the hydropower projects are in western region while power demand and transmission lines are in eastern region of Nepal.

1.6 DOMESTIC POWER DEMAND AND SUPPLY

Traditional sources of energy of domestic purpose were forests, which are declining rapidly these days to provide food and shelter for the increasing population. On the one hand, power demand is increasing corresponding to the population and industrialization and on the other hand, traditional sources of energy are declining day by day. Again availability of commercially viable petroleum deposits or other minerals to meet the increasing power demand are not known yet.

So, demand of hydropower, a renewable source of energy is increasingly day by day. Power demand forecast for years to come is present in the table listed

Table: Energy Demand and Peak load forecast in Nepal (Source-NEA)

Year	Energy (GWh)		Peak Demand (MW)	
	Base Case	Planning Target	Base Case	Planning Target
2005	2502	2722	571	622
2010	3637	4266	831	974
2015	5185	6848	1184	1563
2020	7244	9973	1654	2277

These, days power supply in national grid has been improved. Private sectors have been encouraged for hydropower generation after the formulation of National Hydropower Policy 1992. This approach has supported NEA to avoid other costly power generating systems like diesel power plant etc. In year 1998, NEA purchased of 210.29 GWh from India and 83.47 GWh from Butwal Power Company (BPC). It is expected that power production from existing project like: Puwa Khola Hydropower Project (6 MW), Modi Khola Hydropower Project (14 MW), Kali-Gandaki A Hydropower Project (144 MW), Chilime Hydropower Project (20 MW),Khimiti Hydropower Project (60 MW), Bhotekoshi Hydropower Project (36 MW), Indrawati Hydropower Project (5 MW) etc. cannot meet the power demand on up coming years.

1.7 LOAD FORECAST

The load forecast for Integrated National Power System (INPS) made by NEA according to the power system master plan studies is presented here under table. The load has been forecasted considering the country's macro- economic indicators and rural electrification expansion programs. The forecast revealed that the energy and peak demand is expected to grow more than three times between 2005 and 2020.

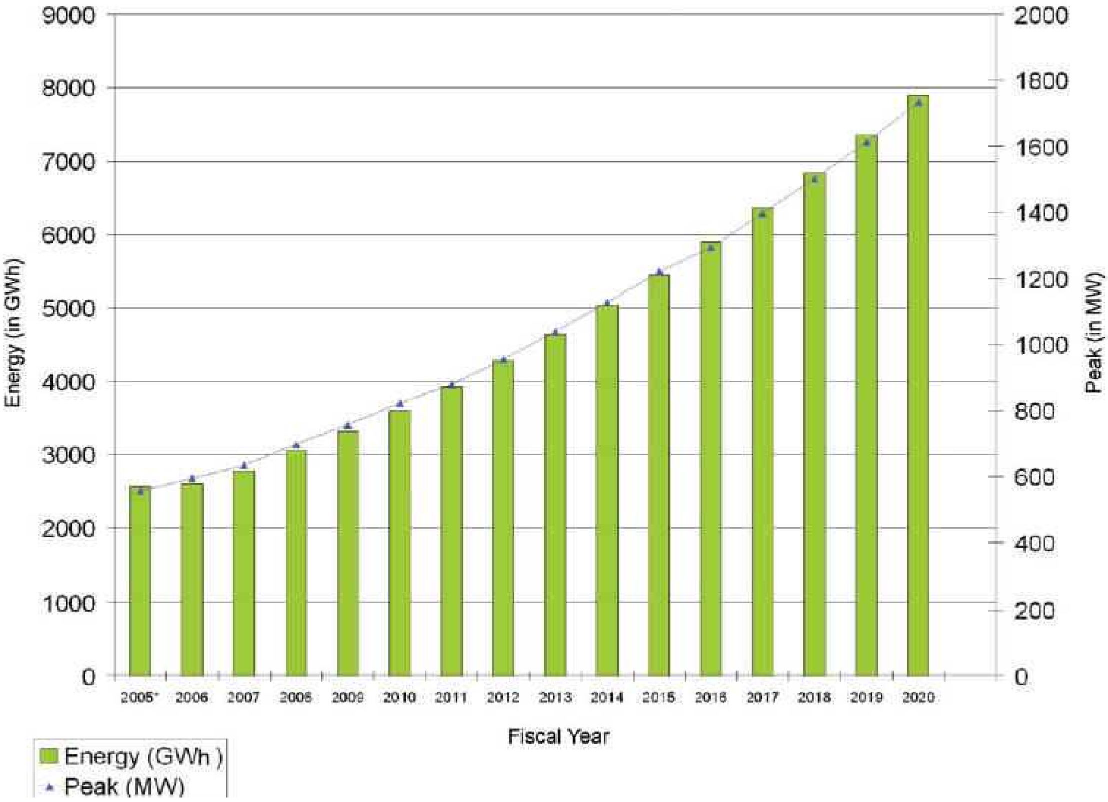
Load forecast for INPS (NEA, 2003/4)

LOAD FORECAST

For Integrated Nepal Power System (INPS)

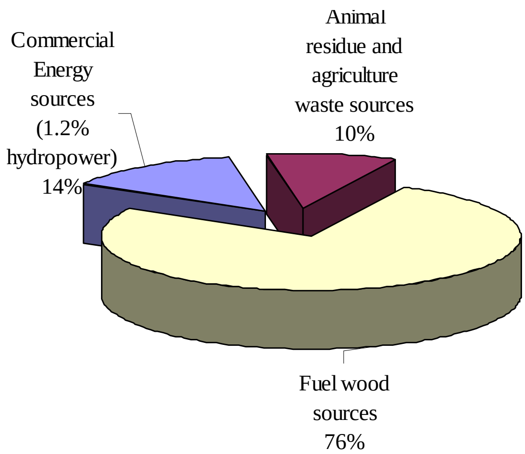
Year	Energy (GWh)	Growth(%)	Peak (MW)	Growth(%)
2005*	2565.8		556.3	
2006	2600.1	1.3	593.6	6.7
2007	2777.6	6.8	634.2	6.8
2008	3055.9	10.0	697.7	10.0
2009	3317.4	8.6	757.4	8.6
2010	3598.9	8.5	821.7	8.5
2011	3923.6	9.0	878.2	6.9
2012	4271.1	8.9	956	8.9
2013	4640.4	8.6	1038.7	8.7
2014	5032.9	8.5	1126.5	8.5
2015	5450.3	8.3	1220	8.3
2016	5894.5	8.2	1294	6.1
2017	6367.4	8.0	1397.8	8.0
2018	6842.3	7.5	1502.1	7.5
2019	7350.4	7.4	1613.6	7.4
2020	7894	7.4	1733	7.4
Average Growth		7.80		7.87

*Actual



1.8 ENERGY CONSUMPTION PATTERN OF NEPAL

In Nepal traditional energy sources are the biggest contributors having share of 86% in the total energy. These sources comprise of fuel wood (76 %), agricultural residues and animal wastes (10 %). Commercial energy sources share 13.64 % having rest to other non conventional sources. Electricity contributes about 1.2 % of the total energy needs.



1.9POWER DISTRIBUTION PLAN

The need to extend distribution over the country is reflected from the fact that 85% population of the country is not getting electricity as a source of energy. So, the distribution of electricity should be done strategically. NEA has taken systematic studies of carrying out rural electrification and distribution system reinforcement (DSR) feasibility on district-wise basis. NEA intends to undertake these works with multi-source financing. Also, Nepal Government contributes to rural electrification scheme on an annual basis with an increasing magnitude in the year, 1999/2000, outlay being approximately 4.5 million US dollar. NEA and Nepal Government are jointly working for the electrification of rural areas. To cope with this objective, micro and small hydropower are the better options in the present scenario. The total capital investment in distribution system expansion and reinforcement for the fiscal year 1999/2000 to 2007 is estimated at 9,349.2 million NRS.

1.10 LEGAL PROVISIONS FOR INVESTMENT

Hydropower industry is one of the major industries with wider scope in Nepal. For an industry to prosper there should be support of government policies and legal provisions. Only the potential cannot do the development of a nation if the policies cannot be harnessed. Clearly defined conditions and attractive policy are always essential to harness the innumerous resources. Realizing this fact, Nepal Government has developed certain policies.

a. Why to invest in Nepal?

- ❖ Attractive Investment Features
- ❖ One-Window Policy
- ❖ Repatriation of Foreign Exchange
- ❖ Income Tax Incentives
- ❖ Fixed Royalty Payments
- ❖ Import Concessions

- ❖ Export Opportunities
- ❖ No Nationalization of Projects

b. Policies, Act and Regulations:

- ❖ Hydropower Development Policy-1992
- ❖ Industrial Policy- 1992
- ❖ Foreign Investment and One Window Policy- 1992
- ❖ Electricity Act- 1992
- ❖ Industrial Enterprises Act-1992
- ❖ Foreign Investment and Technology Transfer Act -1992
- ❖ Environment Conservation Act – 1996
- ❖ National Environment Impact Assessment Guidelines – 1993

c. Legal Framework:

- ❖ Survey License issued within 30 days
- ❖ Survey License Period up to 5 years
- ❖ Project License issued within 120 days
- ❖ Project License period up to 50 years
- ❖ Exclusive Water Rights
- ❖ Public Consultation before issuance of Project License
- ❖ Government land available on lease

d. Institutional Framework for Electricity Development as "One Window"

- ❖ Issuance of Survey & Survey licenses
- ❖ Provision of tax concessions & incentives
- ❖ Assistance in importing goods, land permits, approvals etc.
- ❖ Regulation and monitoring of projects

e. Incentive Income Tax

- ❖ Generation :- 15 years tax holiday
- ❖ Transmission:- 10 years tax holiday
- ❖ O & M Contracts:- 5 year tax holiday
- ❖ After tax holiday:- 10 percent less than period prevailing
- ❖ Foreign Lenders:- 50 percent capital cost allowance
- ❖ Equity Investors:- No tax on interest earned
- ❖ No tax on dividend

f. Import Concessions:-

- ❖ Plant and Equipment including Construction Equipment
  - ~ 1% Custom Duty
  - ~ No import License Fee
  - ~ No sales Tax etc

g. Repatriation of Foreign Exchange

- ❖ Principal and interest on debt
- ❖ Return on equity
- ❖ Sale of share equity
- ❖ Prevailing Market rates

h. Royalty Payments:

- ❖ For year from 1 to 15 year
  - ~ On Install Capacity- NRs. 100/KW
  - ~ On Energy Generated – 2% of Average Tariff/Kwh

- ❖ For Year after 15 years
  - ~ On Install Capacity- NRs. 1000/KW
  - ~ On Energy Generated- 10% of Average Tariff/KWh

i. Market:

- ❖ Domestic: Nepal Electricity Authority (NEA)
- ❖ Foreign: India
  - ~ Under Power Exchange Agreement
  - ~ Under Power Trade Agreement between two countries
- ❖ Regional: Government
  - ~ Probably under the Regional Cooperation especially quadrangle concept within SAARC

j. Nepal Government/ NEA Policy on Purchases from Small Project

The private sectors should do the Power Purchase Agreement (PPA) with NEA to sell the energy produced. To promote the private sectors in national level and to provide the opportunity to invest in the hydropower sectors for the Nepalese people, NEA has the provision to purchase the energy of small hydropower plants with first priority.

k. Export Opportunities:

- ❖ Existing Power Trade Agreement between Nepal and India
- ❖ Existing Interconnection Facilities with India
- ❖ Power Deficit in India
- ❖ Oriented Projects in Nepal

2.0 GENERAL DESCRIPTION OF PROJECT

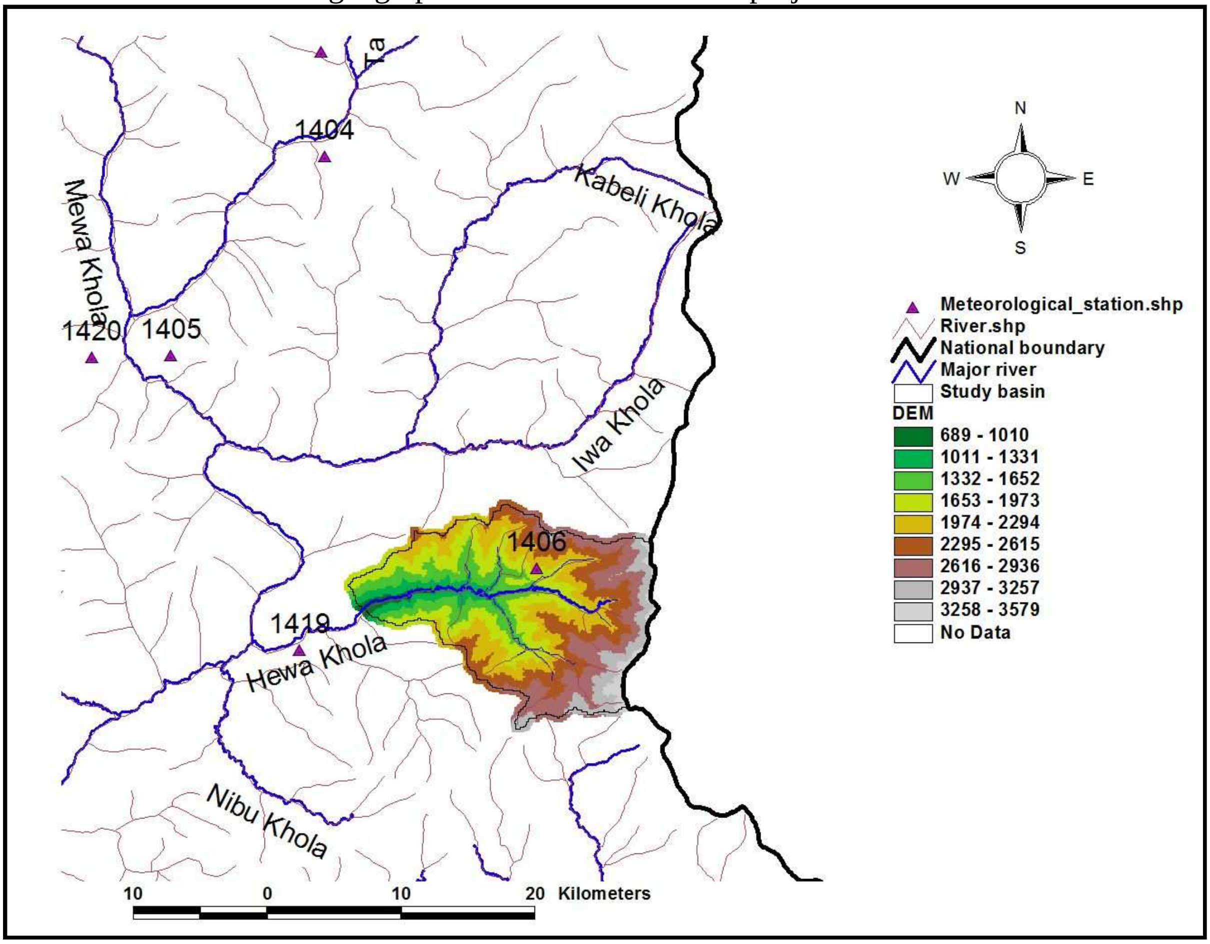
The proposed Hewa Khola Small Hydropower Project is located at Sundaradevi, Bharpa VDCs in Panchthar District, eastern development region of Nepal. It is a run off river snow fed type of plant. Hewa khola is a tributary of the Tamor River which meets the Saptakosi River at the Triben. The Saptakosi drains central and eastern part of Nepal. The catchment area of the project site is found to be 221 km^2 with the help of topographical map drawn at 1:25000 scales. The installed capacity of the plant is 3.8MW.

2.1 OBJECTIVE OF THE PREFEASIBILITY STUDY

The objective of this study is to know the technical feasibility, economical viability and environmental acceptability of the project. In this study surface geology, topography, hydrological study and environmental and social datas are collected. This report reflects the necessacity of further study or termination.

2.2 LOCATION OF PROJECT SITE

Hewa Khola Small Hydropower project is located in Bharpa and Nagani VDC in Panchthar District, eastern development region of Nepal. The project area lies on the left bank of the Iwa Khola . The location and geographical co-ordinates of the project area are shown below.



Geographical co-ordinates of the project site

Description	Latitude, N	Longitude, E
Project area boundary	27˚10'40" to 27˚09'39"	87˚47'42" to 87˚46'10"

2.3 ACCESSIBILITY

The project is accessible partly through earthen road and partly through Gravel road. Project site such as headworks and powerhouse is not accessible and thus requires construction of access road. It will be about 200 m and 500 m to headworks and powerhouse respectively.

2.3 BASIN PHYSIOGRAPHY

Watershed of Hewa khola is a tributary of the Tamor River which meets the Saptakosi River at the Tribeni the Saptakosi drains central and eastern part of Nepal. The study river is a rainfed river while the Tamor River is a snow fed river having large drainage basin compared to the study basin. The nearest hydrometric stations from the study basin with long published flow records and in rain fed river is the 728 gauging station in Maikhola at Rajdwali. Hence the reference hydrological analysis for the project was made with respect to the Maikhola River gauging station 728. The basin lies within latitude of 27º10’40”and 27º 09’ 39” N and longitude of 87º47º 42’ and 87º46º 10’ E. The total catchment at the proposed intake is 221 km^2. The total length of Hewa Khola upto the confluence to Tamor River is about 35 km. The Hewa Khola flows with an average river slope of about 1 in 30 average. However, it is about 1 in 20 in the project corridor. It has elevation ranging from 600 m to above 3573 m.

The project area is mostly covered by alluvial soils. The project area is occupied with rocks belonging to Kunchha Group such as bedded schist; phyllites and meta-sandstone with few quartzite bands occupy the project area. No active faults and landslides are present in the project area

2.4 GEOLOGY OF PROJECT SITE

Hewa Khola Project site is located in Lesser Himalayan zone. Geology of the project site is sound with respect to surface geological study. Surface geology of the site define the design type, quality etc. of any structure on or below the surface of earth. For the detailed study of the project geology of the project site should be identified.

3.0 HYDROMETEOROLOGY

3.1 INTRODUCTION

Several activities were carried out under the hydrological and sedimentation studies to achieve the following objectives:

- Determination of long term mean monthly discharges available for power generation
- Preparation of Flow Duration Curve for determination of installed capacity
- Estimation of the magnitude of design flood and diversion flood for the design of spill way and diversion facilities during the construction period
- Assessment of sediment transport load at the head work site of Hewa khola HPP based on regional approach and sediment data observed at the gauging site

3.2 SCOPE OF WORK

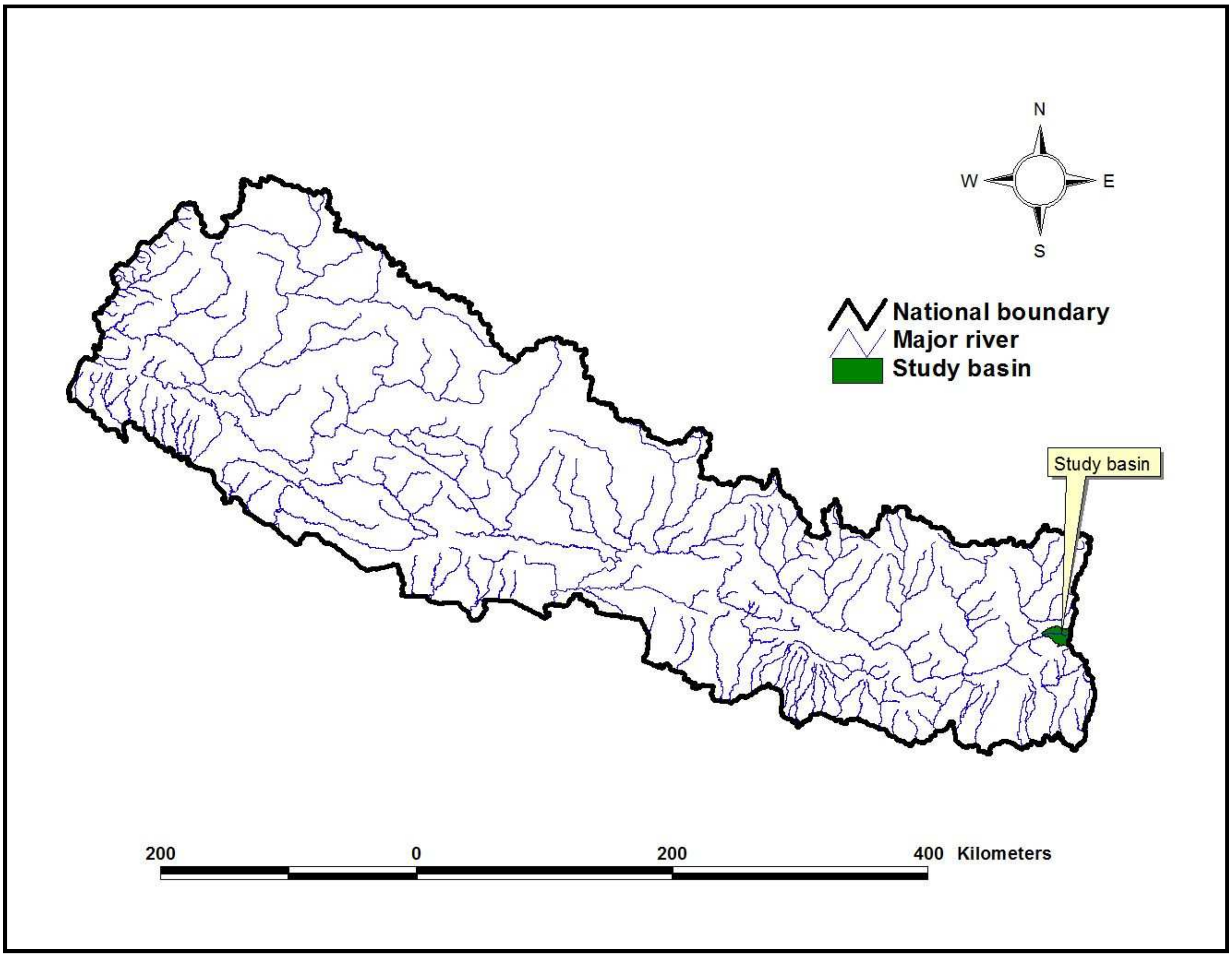
The main works under the hydrological sediment studies are listed below.

- ➢ Collection of available meteorological data
- ➢ Collection of available hydrological data
- ➢ Analyze the available data to estimate pertinent (which is appropriate to a particular situation) hydrological parameters like Design flood, Diversion flood, Flow Duration curve, and long term mean monthly flow, etc.
- ➢ Collect discharge at the dam site for checking estimation of flow
- ➢ Collect available sediment data and map
- ➢ Estimate sediment load at dam site

3.3 HYDROLOGICAL INVESTIGATIONS

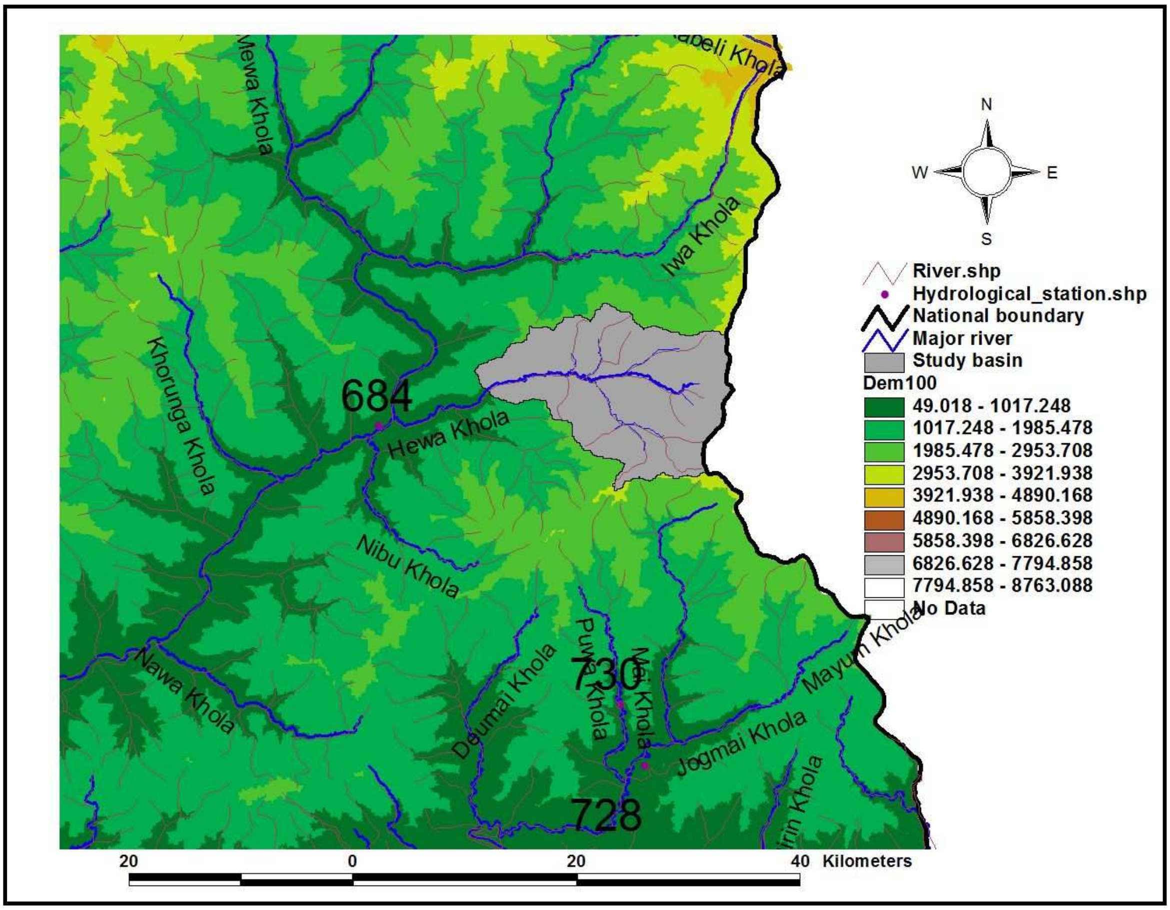
Hydrological investigations were carried out to collect hydrological data and to estimate the pertinent hydrological parameters like design flood flow, long term average flow, low flow and flow duration curve. These parameters are basic and essential to complete the feasibility and IEE study of the hydropower project. The Hydrological investigations were done based on both primary and secondary data. The standards methods and instruments commonly used for hydrological data acquisition were applied to reduce errors at the source. Standard analysis techniques and software have been applied during the study.Collection of Available Meteorological and Hydrological Data

The Hewa khola watershed above the proposed intake and power house sites does not have any hydrometric stations. The nearest hydrometric station from the study Hewa khola waterhed is in the Tamor River in Majhitar with the station number 684 which is located about 500 m downstream after the confluence of Hewa khola with the Tamor river. DHM has published the flow records from 1996-2006 of the gauging station. The study watershed of Hewa khola is a tributary of the Tamor River which meets the Saptakosi River at the Tribeni. The Saptakosi drains central and eastern part of Nepal as shown in Location map given in Figure. The study basin lies in Panchtahar District of Mechi Zone in Eastern development region.

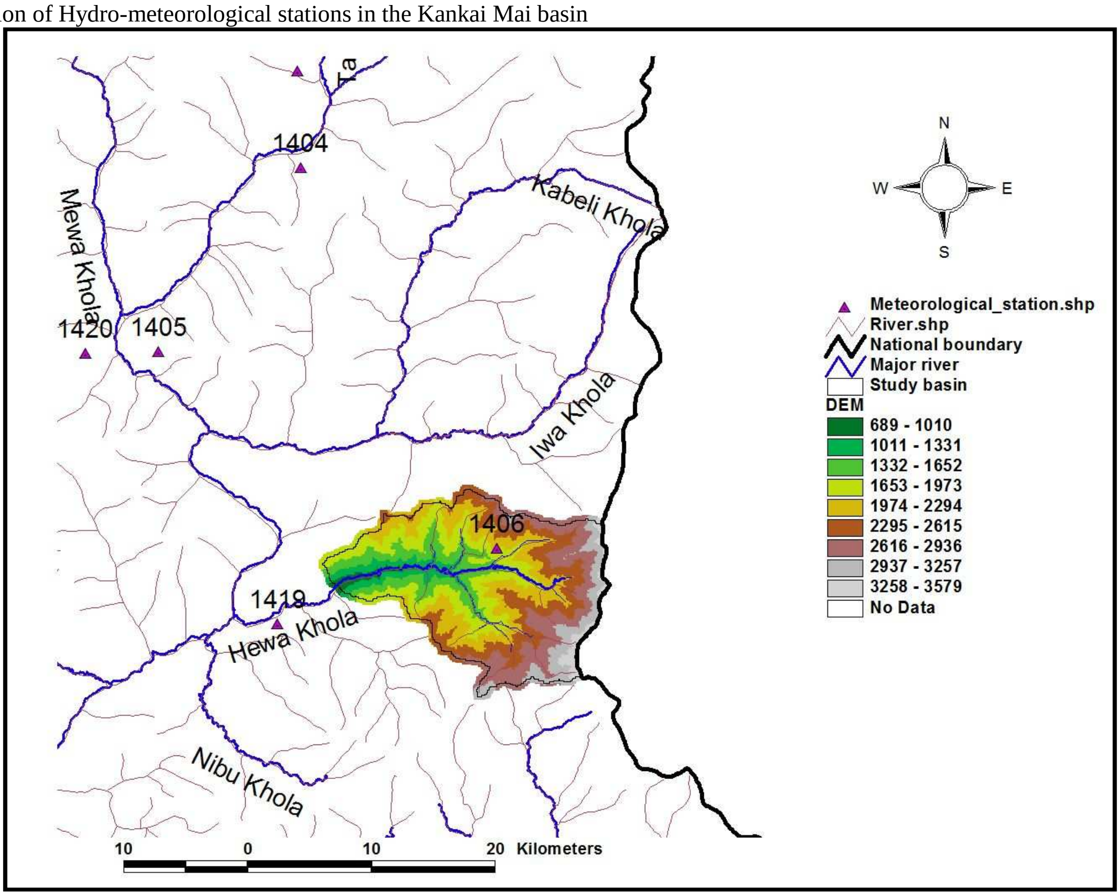


Location map of the Study Basin

The study river is a rain fed river while the Tamor River is a snow fed river having large drainage basin compared to the study basin. The nearest hydrometric stations from the study basin with long published flow records and in rain fed river is the 728 gauging station in Maikhola at Rajdwali. Hence the reference hydrological analysis for the project were made with respect to the Maikhola River gauging station 728. The gauging station 728 lies south east from the Hewa study basin. The gauging station 728 has published flow records from 1983 to 1995 and the monthly flow records including the extreme instantaneous maximum and minimum historical flow records were collected from the DHM. The flow data of the referenced stations were given in Data attachment section at the end of this report.



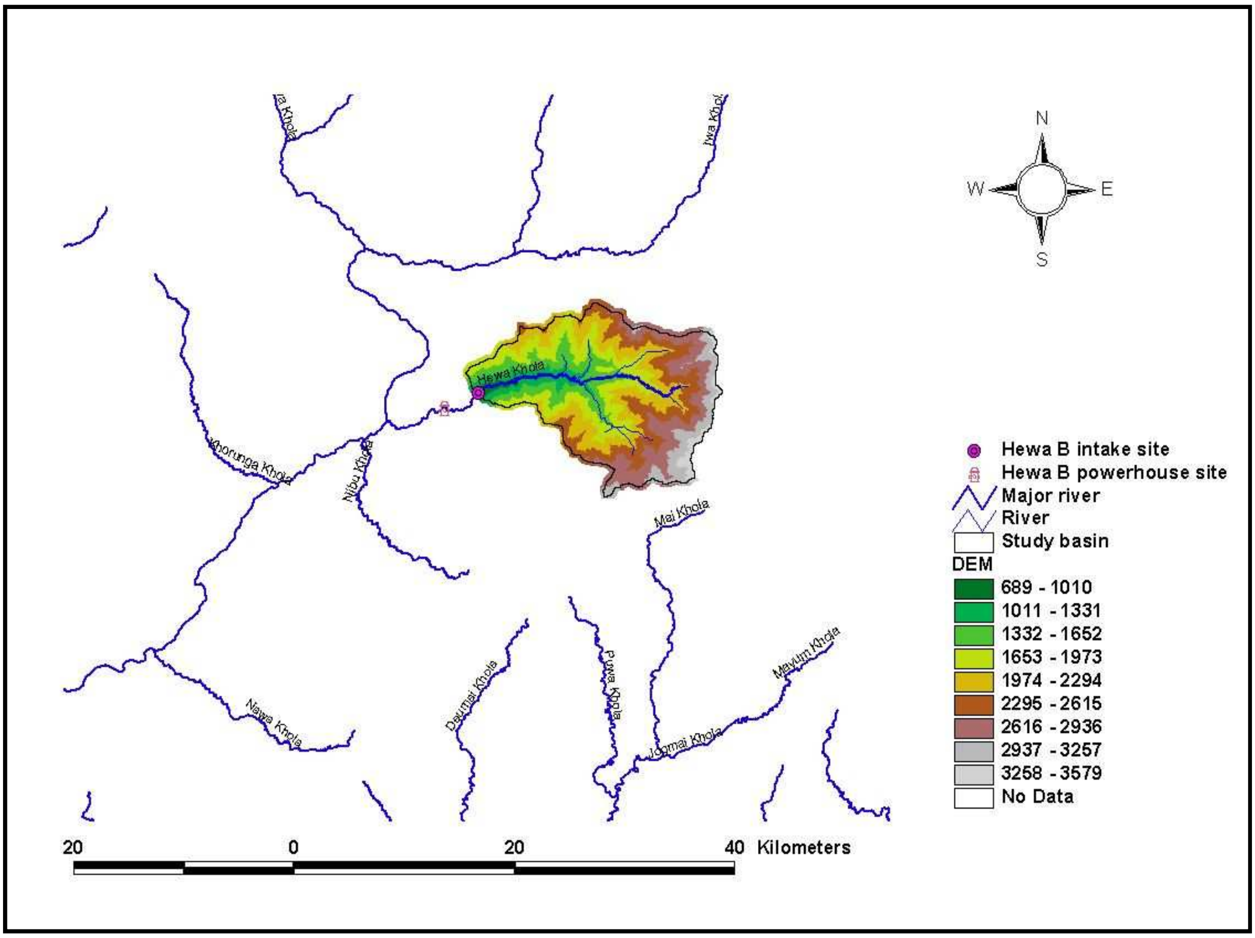
Location of Hydro-meteorological stations in the Kankai Mai basin



The location of meteorological stations around the study basin

3.4 HYDROLOGICAL STUDIES

The hydrological studies of the project mainly focus on the interpretation and analysis of collected relevant primary and secondary data and information as described in section 3 above by applying the standard and appropriate methodologies and procedures to determine important hydrological parameters such as long term mean monthly flows, flood flows, dry season flows, construction flood flows, flow duration pattern, and stage discharge relationship etc as per requirement for completion of the feasibility design and IEE study of the project.



Hewa Khola Study basin above the intake site

Table 4.1 shows the basic basin characteristics of the study basin above the intake and power house site. The basin characteristics of the reference river up to the hydrometric stations 728 were also extracted from the topographic map. The basin characteristics of the referenced river up to the hydrometric stations were listed below.

Basin characteristics of the study basins and reference river basins

S.N.	Description	unit	Dam Site (low Head weir site)	Powerhouse site	Gauging Site 728
1	Catchment Area	km^2	221.35	353.62	383.55
2	Perimeter	km	91.4	103.8	118

3	Catchment Area <5 km	km^2	221.35	353.62	383.55
4	Catchment Area <3 km	km^2	203.37	334.36	376.67
5	Catchment Area >5 km	km^2	0.00	0.00	0.00
6	Length of longest flow channel	km	28.56	32.743	38.398
7	US elevation	m	3546	3546	3578
8	DS elevation	m	689	565	424
9	Average slope of Longest flow channel	%	10.00	9.10	8.21
10	Length of flow up to centroid of the CA	km	13.79	20.05	19.28
11	Max elevation in basin	m	3579	3579	3600
12	Min elevation in basin	m	689	565	424
13	Mean elevation of basin	m	2190.62	2061.768	1715.086
14	Elevation of basin centroid	m	1418	1265	2337

The longest flow length of the Hewa khola from its origin to the proposed intake site is about 13.8 km. The drainage area of the river up to the intake site is 221.35 km^2.

Similarly the drainage area below 3000 m and 5000 m elevations above the proposed powerhouse site were reported to be 203.37 km^2 and 221km^2 respectively. These data were directly used in the hydrological analysis for determination of design discharge to determine installed capacity of this Hewa B cascade HPP project.

3.5 METHODOLOGY

Depending upon whether a river is gauged or ungauged, the method available for estimating the long term hydrological investigation as well as to estimate the low flow and flood flow can be broadly classified into two categories – direct method and indirect method. Direct method is used to estimate the flow using the data available at gauged station; indirect method is used for an ungauged station where no or very few data are available in the vicinity of the project area.

3.6 METHODOLOGIES FOR UNGAUGED CATCHMENTS

Though the catchment is a gauged catchment and thus long term flow is available. However, an attempt has been made to use various methods common to ungauged catchment so that it could be possible to check the flow data as generated by transforming the gauged data to the point of interest. Followings are widely adopted methods for most of the ungauged catchment in Nepal and are discussed hereunder.

3.6.1 MEDIUM IRRIGATION PROJECT (MIP) METHOD

The Medium Irrigation Project (MIP) method presents non-dimensional hydrographs of mean monthly flows for seven different physiographic regions of Nepal. This method is applicable to the ungauged sites. This method is applicable only if there are measured flow at least in the low flow season of a year. If the measured flow is directly used, the MIP averages the flow for the middle of the month and thus gives unrealistic flow information. It is thus necessary to adjust flow value if measured at the beginning or end of the month. The measured flow is used with regional non-dimensional hydrograph to synthesize an annual hydrograph for the site.

3.6.2 HYDEST METHOD

The method was developed by WECS/DHM in 1990 for determining the hydrological characteristic of ungauged catchment. This method is used to determine the instantaneous flood peak, long term and mean monthly flow by using computer software or manually. But in this our project we have used software. For the complete hydrological analysis by this approach, the catchments area and its distribution in altitude are essential along with Monsoon Wetness Index (MWI) of the catchments. The monsoon wetness index from the isoheytal map for the project area is taken as 1500 mm. The modified hydest is also used to analyze the hydrological parameters of the project.

3.6.3 MEDIUM HYDROPOWER STUDY PROJECT (MHSP) METHOD

The Medium Hydropower Study Project (MHSP) under NEA in 1997 developed a method to predict long-term flows, flood flows and flow duration curves at ungauged sites through regional regression technique. The MHSP method has been used to estimate mean monthly flow series at the proposed intake site. The input variables are similar to those used in WECS/DHM method. This approach uses both MWI and average precipitation of the area along with the catchment area of the River.

3.6.4 CATCHMENT CORRELATION METHOD

This method is used when there is unavailability of hydrological data at the headworks. Since there is no availability of hydrological data particularly at the headworks area and thus an attempt was made to correlate the flows with Station 728 located. This is simply because of the similarity of the catchment in many respects with the mother catchment. The discharge of the required basin is given by:

$$Q_2 = \left(\frac{A_2}{A_1}\right) Q_1$$

Where, Q_1= Known discharge of the basin 1.

Q_2= Required discharge of the basin 2.

A_1= Area of the basin 1.

A_2 = Area of the basin 2.

3.7 FLOW ANALYSIS

3.7.1 HIGH FLOOD ANALYSIS

Depending on whether a river is gauged or not, the methods available for estimating the flood discharge of rivers can be broadly classified into two categories – direct methods and indirect methods. Direct methods are used to estimate design floods for different return periods using the flow data or precipitation data available at gauged locations. Indirect methods are helpful in estimating floods for an ungauged basin, where no, or very few, data are available in the vicinity of the study area.

3.7.2 WECS/DHM METHOD

From WECS method long term flow is calculated by using following equation:

$$Q_{mean} = C \times (\text{Total basin area})^{A1} \times (\text{Basin area below 5000m} + 1)^{A2} \times (\text{Monsoon wetness index})^{A3}$$

Where, C, A1, A2, A3 are constants derived from the regression analysis.

A is the catchment area in Km^2.

Q is discharge in m^3/sec

The values of the constants for different months are different. The Monsoon Wetness Index for the catchment area is taken as 1500 mm.

The mean monthly flow using WECS/DHM is presented in Appendix.

3.7.3 MEDIUM IRRIGATION PROJECT (MIP) METHOD

The MIP Method for long term flow analysis, developed for the design of medium irrigation projects in Nepal, is based on regional non-dimensional hydrographs drawn up for seven regional groups of Nepal.

3.7.4 REGIONAL REGRESSION METHOD

The WECS/DHM Method was developed by WECS (1989) which estimates the hydrological characteristics of ungauged sites in Nepal using a frequency distribution parameter technique that is a variation of the multiple regression technique. In this method, the independent variable that is most significant in the regression analysis is the area of the basin below the 3,000 m elevation, i.e. the area of the basin influenced by monsoon precipitation. This method is not applicable to basins located entirely above 3,000 m, and its results for basins with a very small portion below the 3,000 m elevation are not particularly reliable.

The WECS/DHM Method uses regression equations for 2-year (median flood) and 100-year floods for both maximum daily and maximum instantaneous flood peaks of the form:

$$Q_{aby} = \alpha(A_{3000} + 1)^{\beta}$$

where Q_{aby} is the discharge in m^3/s, subscript a is either a daily or an instantaneous flood peak, subscript b is either a 2 year or a 100 year return period, A_{3000} is the catchment area below 3,000 m and α and β are coefficients and taken from reference book. Using this equation, floods of other return periods can be calculated simply by the plotting the 2 year and 100 year floods on log-normal probability paper, which results in a straight line. Alternatively, algebraic equation can be used for this purpose.

So Flood flow of 2 years and 100 years return period are predicated using following equations:

$$Q_{inst,2} = 1.876(A_{3000} + 1)^{0.8783}$$

$$Q_{inst,100} = 14.63(A_{3000} + 1)^{0.7343}$$

Where, Q is in m³/sec

Using the WECS/DHM Method, the daily and instantaneous floods for different return periods were calculated.

3.7.5 FLOOD FLOW USING GUMBEL'S METHOD

This extreme value distribution was introduced by Gumbel (1914) and is commonly known as Gumbel's distribution. It is one of the most widely used probability- distribution functions for extreme values in hydrologic and meteorological studies for prediction of flood peaks, maximum rainfalls, maximum wind speed, etc.

Gumbel defined a flood as the largest of the 365 daily flows and the annual series of flood flows constitute a series of largest values of flows. According to his theory of extreme events, the probability of occurrence of an event equal to or larger than a value of x_0

$$P(X \geq x_0) = 1 - e^{-e^{-y}}$$

In which y is a dimensionless variable given by

$$y = \alpha(x - a)$$

$$a = \bar{x} - 0.45005\sigma_x$$

Thus
$$y = \frac{1.2825(x - \bar{x})}{\sigma_x} + 0.577$$

Where $\bar{x}$ = mean and σ_x = standard deviation of the variate X. In practice it is the value of X for a given P that is required and the eqn. is transposed as

$$Y_p = -\ln[-\ln(1 - P)]$$

Noting that return period T=1/P and designating Y_T= the value of y, commonly called the reduced variate, for a given T,

$$Y_T = -\left[\ln \ln \frac{T}{T - 1}\right]$$

$$Y_T = -\left[0.834 + 2.303 \log \log \frac{T}{T - 1}\right]$$

So, the value of variate X with a return period T is

$$x_T = \bar{x} + K\sigma_x$$

$$where,\ K = \frac{(y_T - 0.577)}{1.2825}$$

The values obtained from Gumbel's Method are fitted on the best fit line obtained from plotting position method.

3.7.6 FLOOD FLOW USING LOG –PEARSON TYPE III DISTRIBUTION

In this method the variant is first transformed into logarithmic form (base10) and the transformed data is then analyzed. If X is variant of random hydrologic series, then the series of z variants Where,

$$z = \log x$$

For z series, for any recurrence interval T

$$z_T = \bar{z} + K_z \sigma_z$$

Where K_z = a frequency factor is function of recurrence interval T and coefficient of skew C_s

$\sigma_z =$ Standard deviation of the Z variant sample

$$\sigma_z = \sqrt{\frac{\sum (z - \bar{z})^2}{(N - 1)}}$$

$C_{s=}$ coefficient of skew of variant Z

$$C_s = \frac{N \sum (z - \bar{z})}{(N-1)(N-2)\sigma_z^{\ 3}}$$

The variations of K_z =f (C_s, T) is given in table.

The corresponding value of x_T=antilog (z_T)

3.8 REFERENCE HYDROLOGY AND STREAM FLOW ANALYSIS

Looking at the physiographic conditions and proximity of the gauging stations, it would be more appropriate to use the discharge data from the Maikhola observed at Rajdwali station 728 for deriving the discharge data at the intake site of the Hewa Khola. Both of these rivers are rain fed rivers and lying in eastern part of the Nepal. The station 728 is in South East of the Hewa khola basin having comparable drainage area. Since the elevation variations in the study basin and reference basin is in the same range and the drainage areas are in comparable, the result obtained from the reference hydrological analysis provide reliable and realistic data although it may produce slightly overestimate of the flow because of likelihood of having larger rainfall in the referenced basin compared to the study basin.

3.9 REVIEW OF DRAINAGE AREA

The drainage area is an important parameter in reference hydrological study as it gives the base for transformation of stream flow records observed at one location to other. Long Term Average Mean Monthly and Yearly Flow (m^3/s) at reference station is given below:

St^n no.	Jan	Feb	Mar	Apr	May	Jun	July	Aug	Sep	Oct	Nov	Dec	Yearly
728	5.72	4.83	4.61	5.86	10.02	25.68	52.52	49.81	47.39	20.55	10.16	6.88	21.39

The specific run off computed from the Maikhola river basin was found to be 0.0677 $m^3/s/km^2$. The specific run off depends upon many physiographic, land use, land cover and climatic factors. There are no different river catchments having perfectly similar hydrological catchments. Primary data collected from the gauging station at the dam site and powerhouse site are important and essential for precise estimation of hydrological parameters. At the present level of study, there is no sufficient primary data and hydrological estimations were carried out based on secondary data observed at the reference station having similar hydrological characteristics.

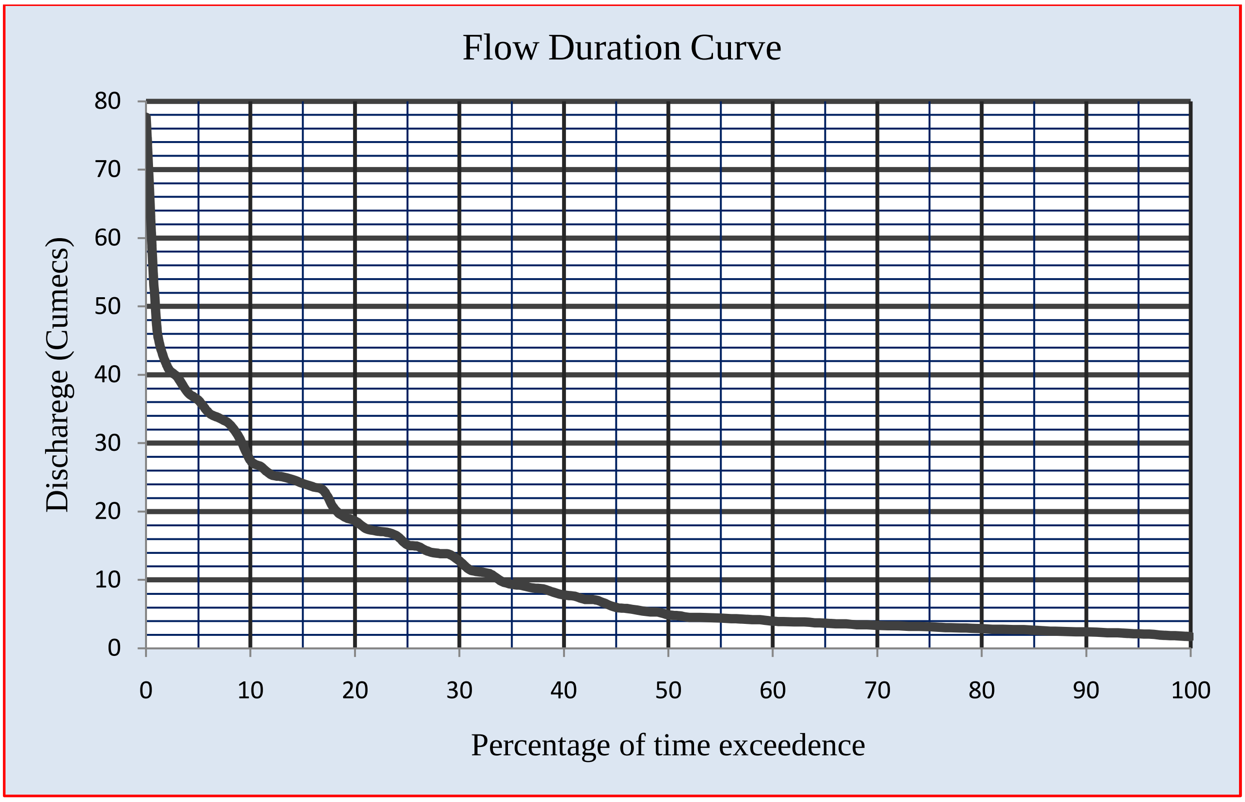
3.10 LONG-TERM STREAMFLOW SERIES AT THE INTAKE SITES

Long term mean monthly flow were calculated by applying Catchment Area Ratio method (CAR) method directly from the data observed at the reference stations 728. Beside this, the long term mean monthly flow were also estimated applying widely used regional approaches like WECS-DHM and Hydrological Estimation in Nepal (DHM 2004) methods.

3.11 FLOW DURATION CURVE ANALYSIS

If a run-off-river hydropower scheme requires flows greater than the minimum stream flow for power generation, it is useful to know the variation of flow over the year to select the most appropriate turbine configuration. For this purpose, information presented in the form of a flow-duration curve is most useful.

The average flow-duration curve is an exceedence probability-discharge curve showing the percentage of time when a particular flow is equaled or exceeded. The flow duration curve was prepared from the generated mean monthly flow data from 1983 to 1995 at the intake site from transformation of mean daily flow data of Maikhola observed at the 728 gauging station as described above. The generated mean monthly flow data for the intake site was arranged in descending order to find the flow corresponding to different probability of exceedence. The result of the flow duration curve has shown below. The flow duration curve analysis were also carried out using the ready made Microsoft Excel function “PERCENTILE ((Data array, (1-% of time exceedence)) on the same data in spread sheet for checking the result. The flow duration curve analysis was carried out using the long term average mean monthly flow generated at the intake site from the reference hydrology.



Estimated FDC at the dam site of Hewa khola

These values are recommended to use for preliminary design and analysis of the hydropower project. It is recommended to increase the flow measurement at the intake site more frequently to verify the results and apply necessary corrections if necessary depending upon the observed values.

Estimated available flow (m^3/s) at dam site for different percentile of Time Exceedence % of a year

Percent of time	Q(m^3/s) flow- 728 average mean monthly flow (AMM)	Q(m^3/s) flow average mean monthly flow (AMM)
0%	142	77.67
5%	66	36.3
10%	50	27.45
15%	44	24.12
20%	34	18.65
25%	28	15.15
30%	23	12.77
35%	17	9.38
40%	14	7.8
45%	11	5.98
50%	9	4.92

55%	8	4.43
60%	7	3.98
65%	7	3.7
70%	6	3.38
75%	6	3.17
80%	5	2.91
85%	5	2.67
90%	4	2.43
95%	4	2.13
100%	3	1.73

The 40% excedence flow of the Hewa Khola-B Hydropower Project taken in the design discharge is of 7.8 m^3/s while the flow values of 50 and 60% of time exceedence were 4.92 m^3/s and 3.98 m^3/s respectively as shown in above table.

3.12 FLOOD FLOW ESTIMATION

The objective of the flood analysis was to estimate the project inflow floods up to 1000 year return periods. These peak flood values are required to determine the spillway design flood. Owing to the negligible storage capability of the Hewa Khola Hydroelectric project, information on the shape of the flood hydrographs is of less importance.

It is emphasized that the Hewa Khola Hydroelectric Project will have a relatively low dam so that a potential hydrological failure would hardly cause catastrophic consequences in terms of human life and considerable loss of property. This is a fact which was taken into consideration when selecting the return period for spillway design flood.

3.13 FLOOD FREQUENCY ANALYSIS OF PROJECT

The annual monsoon and storm rainfalls in the months June through October cause sustained high flow conditions and floods in the Hewa Khola basin which generally reach their maxima during July to September.

Prior to initiating the flood frequency analysis, the maximum instantaneous discharges were extracted for the intake site from the observed historical flood data at referenced gauging stations 728 using CAR method. Generation of the extreme instantaneous maximum discharge at the intake sites were done for both the intake site and power house site considering the CAR of the whole drainage basin area lying below 3 km elevation.

Flood frequency analysis was performed using a computer spread sheet in Microsoft Excel program. The following types of frequency distribution functions were used in the flood frequency analysis on the generated annual maximum flood series data from 1983-1995 with reference to the 728 gauging station. These analysis were done separately for both the intake and the Powerhouse site.

- Gumbel Extreme Value (GEV)
- Log-Pearson Type III (LPIII)
- Three Parameter Lognormal (LN)

Comparative study of the distribution based on the fitting of observed and computed values; the LN distribution seems better fitted although others distributions are acceptable as there was very little differences were observed between the various distributions. The resulting flood discharges of the Hewa Khola at the proposed intake and powerhouse sites with the return periods are displayed.

3.14 DESIGN FLOOD

Design flood with a return period of 100 years is adopted as 372 m^3/s at the intake site of Hewa Khola B HPP.

3.15 LOW FLOW ANALYSIS

Information of low flow is needed to determine the maximum power that a run-of-river plant can generate during the peak of the dry season. The minimum usable flow in a stream determines the value of reliable firm power and then firm energy. Knowledge of minimum stream flow is essential also for determination of minimum water level that can goes down to the river at the intake. Therefore Low flow analysis is essential in the planning of hydropower in run off river and pondage run off river modes.

The duration curve of the long-term daily inflow series predicts the flow duration for an average hydrological year. Individual dry and wet years would display different flow duration characteristics. For a hydroelectric plant, sustained low flows experienced in the dry years are critical to the operation resulting in nil energy generation when the flow becomes less than the minimum permissible flow to avoid considerable cavitations.

In order to predict the likelihood of this occurring, a probabilistic low flow analysis was carried out by analyzing the mean daily project inflow time series (1983-1995) with reference to 728 gauging station) of the Hewa Khola at the intake sites using the minimum instantaneous flow series observed at the reference stations. The catchments area transformation methods were applied for the generation of the low flow series considering the area below 5 km elevation. In addition to the frequency analysis, regional approaches WECS-DHM and DHM-2004 were also applied to estimate the likely hood of the low flow values.

3.16 RIPARIAN RELEASE

The amount of riparian release that is made available in the downstream of the intake site is the most crucial factor to sustain aquatic ecosystem of the river during the operation of the hydroelectric project. The release is more important in the dry season, when it would be tendency to divert all the flow in to the power channel. It is mandatory for the project to release some portion of the flow to maintain aquatic environment to some extent at the reach between the intake and the tailrace site. It has been practiced that roughly that 10% of the minimum monthly average flow is required to sustain such activities. Hence, 0.25 m^3/s corresponding to about 10% of the minimum recommended mean monthly flow 2.54 m^3/s in February is needed to release downstream of the intake in Hewa khola as environmental compensation flow during dry seasons. The Environmental study will re-define the minimum requirement of the riparian release based on the project impacts on aquatic life and their nature.

4.0 POWER OUTPUT AND ENERGY GENERATION

Some definitions

Primary or Firm power

The power which is available throughout the year is known as firm power. It is the power which is always ensured to a consumer at any time of a day. This type of power may correspond to the minimum flow of river and is available for all the time.

Secondary or Surplus or Non-Firm Power

If the power is available intermittently for unpredictable time, the power is called the secondary power. It is the excess power available over the firm power during the off-peak hours or monsoon etc. In other words, it is surplus or non firm power other than the primary one and is useful in the interconnected system of power station i.e. grid.

Gross Head

It is the difference in WL elevation at the point of diversion and the point of return of water back to the river. The gross head obtained is 65.45m.

Net Head

It is the head obtained after the deducting the losses between the diversion point and axis of turbine from gross head. The net head obtained is 55.63m.

Overall Efficiency

$$\eta_o = \eta_h \times \eta_t$$

Where,

η_H = Hydraulic efficiency = 0.97

η_T = Turbine efficiency = 0.93

η_o = Overall efficiency = 0.90

4.1 INSTALLED CAPACITY OF PLANT

The installed capacity of a power plant is the maximum power which can be developed by the generators at the normal head with full flow.

$$N = 9.81 \times Q_d \times H_n \times \eta_o$$

Where,

N = Installed capacity i.e. power in KW

Q_d = Design discharge in cumecs

H_n = Net head in m

η_o = Overall Unit Efficiency

5.0 HYDRAULIC DESIGN

5.1 WEIR

A weir is a water diversion structure generally constructed across the run off river to supply sufficient water to the intake. There are different types of weir and the use of each type depends upon the topography, geology, discharge, river morphology etc. If the major part or the entire ponding of water is achieved by a raised crest and smaller part or nil part of it is achieved by the shutters then it is called weir. If most of the ponding is done by gates and smaller or nil part of it is done by the raised crest, then it is called Barrage or River Regulator.

5.1.1 DESIGN CONSIDERATION OF DIVERSION WEIR

The design of weir includes computing the elevation of weir crest, length of weir, computing the forces acting on the weir and checking the safety of the weir from all aspects like overturning, sliding, crushing etc. They all are explained in the following articles.

5.1.2 ELEVATION OF WEIR CREST

There are various factors that affect the elevation of the crest, but in our case, diversion of water is the purpose and the height should be sufficient to pond the water at a level that can facilitate design flow in the intake. The height of the weir is governed by the height of intake sill, depth of intake orifice and depth of the river at the intake site.

Four other important considerations to be considered for fixing the crest level of the weir are as follows:

- The height of the crest affects the discharge coefficient and consequently the water head above the weir as well as the back water curve.
- The elevation of the weir crest has to be fixed such that the design flood is safely discharged to the downstream without severe damage to the downstream.
- The elevation of the weir determines the head of the power production.
- The height of the weir crest affects the shape and location of the jump and the design of the basin.
- The height of the weir crest affects the discharge that can be diverted into the canal.

The bed level of the river at the headwork is 665 m. The crest level of weir provided is 668 m.

5.1.3 LENGTH OF WEIR

The length of the weir depends upon the width of the waterway at the intake site. Crest length should be taken as the average wetted width during the flood. The upstream and downstream should be properly examined for the protection consideration.

Rise in water level on the upstream of the structures after construction of the weir is called afflux. Fixation of afflux depends on the topographic and geomorphologic factors. A high afflux shortens the length of the weir but increases the cost of the river training and river protection works. For alluvial reaches it is generally restricted to 1m but for mountainous region it may be high. The water way must be sufficient to pass high floods with desired afflux. Generally, the waterway is calculated by Lacey's perimeter Formula: $P = 4.75\sqrt{Q}$ for alluvial channel. But for boulder reaches it may be taken just as 60 % of "P" calculated above. Minimum waterway is taken as actual width available between river banks. A weir with crest length smaller than the natural river width can severely interfere the natural regime of flow thus altering the hydraulic as well as the sediment carrying characteristics of the river.

5.1.4 FORCES ACTING ON WEIR

The main forces which are acting on the weir when it will be in operation are: Water Pressure, Uplift Pressure, Slit Pressure and Weight of the weir.

WATER PRESSURE

It is the major external force acting on the weir. This is called hydrostatic pressure force and acts perpendicular on the surface of the weir and its magnitude is given by: $P = 0.5 \times \gamma \times H^2 \times b$

Where, γ = Unit weight of water,

H = Depth of water,

b = Width of the Weir surface.

This pressure force acts on H/3 from the base.

UPLIFT PRESSURE

Water seeping through the pores, cracks and fissures of the foundation material, seeping through the weir body itself and seepage from the bottom joint between the weir and its foundation exerts an uplift pressure on the base of the weir. The uplift pressure virtually reduces the downward weight of the weir hence acts against the dam stability. The analysis of seepage is done using Khosla's Theory. Khosla's Theory is the mathematical solution of the Laplacian equation and it is easy and accurate method for seepage analysis.

According to the USBR, the uplift pressure intensity at the heal and toe should be taken equal to their respective hydrostatic pressure and joined by a straight line in between.

SILT PRESSURE

The silt gets deposited on the upstream of the weir and exerts the horizontal and vertical pressure as exerted by the water. So, flushing of the silt should be done regularly to reduce its effect of destabilizing the weir. It is done by the use of under sluice gate. The silt pressure is given by the relation:

$$P_{silt} = 0.5 \times \gamma_{sub} \times H^2 \times K_a .$$

Where, γ_{sub} = Submerged unit weight of silt

H = Depth of silt deposited and

K_a = Coefficient of Active earth pressure and is given by,

$$K_a = \frac{(1 - \sin\phi)}{(1 + \sin\phi)}, \phi = \text{Angle of internal friction of silt}$$

The silt pressure force also acts at a height of H/3 from the base.

But for practical consideration, Equivalent Liquid = Mix of silt and water

$\gamma_{liquid\,(v)} = 1950\,kgf\,/\,m^3$

$\gamma_{liduid\,(H)} = 1360\,kgf\,/\,m^3$

WEIGHT OF WEIR

The weight of weir and its foundation is the major stabilizing/ resisting force. While calculating the weight, the cross section is splited into rectangle and triangle. The weight of each along with their C.G. is determined. The resultant of all these forces will represent the total weight of dam acting at the C.G. of dam. Simply, when the sectional area of each part is multiplied by unit weight of concrete, weight of that part is obtained.

5.1.5 MODE OF FAILURE AND CHECK FOR STRUCTURAL STABILITY OF WEIR

OVERTURNING ABOUT THE TOE

If resultant of all the forces acting in the weir passes outside, the weir shall rotate and overturn about the toe. Practically, this condition will not arise because the weir will fail much earlier by compression. The ratio of resisting moment to the overturning moment about the toe is the factor of safety against overturning and it should be greater than 1.5.

COMPRESSION OR CRUSHING

While designing the weir section it should be so design that the resultant should pass through middle 3rd part of the section to avoid the possible tension on the weir section. The section should be totally in compression. So, weir should be checked against the failure by crushing of its material. If the actual compressive stress may exceed the allowable stress, the dam material may get crushed. The vertical combine stress at the base is given by: $\sigma_{max/min} = \frac{\sum V}{B}\left(1 \pm 6 \times \frac{e}{B}\right)$,

Where, $e = \frac{B}{2} - x,\ x = \frac{\sum M}{\sum V}$,

e = eccentricity of the resultant force from the centre of the base.

B = Base width of the weir.

SLIDING STABILITY

Sliding will occur when the net horizontal force above any plane in the weir or at the base of the weir exceed the frictional resistance developed at that level. Factor of safety against the sliding is measured as Shear Stability Factor (SSF) and is given by: $SSF = \frac{(\mu \times \sum V + Bq)}{\sum H}$

Where, μ = Coefficient of friction;

q = Average shear strength of the joint.

For safety against sliding, SSF should be greater than 3-5. To increase the value of SSF, attempts are always made to increase the magnitude of q, which is achieved by providing the stepped foundation, ensuring the better bond between the dam base and rock foundation etc.

5.1.6 PROTECTION WORK FOR WEIR STRUCTURE

The weir should be well protected from the flowing river to avoid creep effect. For this, the wing wall is essential to construct. It should be well anchored into the bed. Similarly, to protect the channel bed from being eroded, launching apron is used. To protect the weir body riprap is usually placed. In the site both the banks are vulnerable to erosion hence special protection structure shall be constructed. Gabion walls are used as protection works for the banks which ultimately protect the degradation of the weir. To prevent the seepage effect, sheet piles are inserted at the upstream and downstream.

5.2 INTAKE STRUCTURE

5.2.1 GENERAL

The intake structure is used to tap the required amount of water for the specific purpose with or without storing. An intake structure should ensure good quality of water in proper quantity and a control over the supply of water. For this purpose, arrangements of weir and intake structure must be chosen to evacuate necessary amount of water at any regime to the channel. The peak discharge must be safely evacuated without any damage. To achieve this, hydrological data must be collected and evaluated and the structures should be designed accordingly.

Prerequisites of the location of intake structure

- The course of the river should be relatively permanent at the intake site, i.e. the river should not change its course at the intake location at the time.
- The river should not have a large gradient at the intake site.
- As far as possible the intake should be placed at the side of rocky outcrop or large boulders for the stability and the strength.
- The intake should be on the concave bank of the bend for good performance. This limits sediment deposition at the intake area and also ensures the flow availability during the dry season.

Generally the intake is provided 2-10 m upstream of the diversion weir and the crest of the intake is raised 1-1.5 m bed of the diversion structure.

The intake structure is designed for 30% more than Design discharge 5% for loss and 25% for flushing, i.e. $Q_{design(intake)}=1.3Q_{design.}$ The intake is designed considering free flow submerged condition at normal flow and gated condition at flood discharge.

5.2.2 DESIGN CONSIDERATION OF INTAKE STRUCTURES

For small hydropower projects it is general practice to use 100 years return period from probabilistic analysis of flood. A simple and moderately priced construction should be used to minimize maintenance and repairs. For the small projects with no automation facilities, hydraulically controlled structures become more feasible than mechanically controlled units. In hydraulically controlled intake structure, usual practice is to construct skimmer wall to restrict the flood water entering in the canal, such that intake structure works as free flow weir at normal condition and as submerged orifice at high flood conditions. The excess water is allowed to flow in canal up to a suitable point downstream where it is returned back to river using escape structures.

There must be adequate provision to remove the suspended and bed load deposited upstream behind the weir. This may be done using intermittent flushing using sluice gates or allowing some water to flush it continuously. It has been found that entry of bed load towards diverted canal will be minimum if the intake is located just downstream of concave bank of the river bend. It not only restricts the bed load, but also ensures sufficient water depth even at low water condition.

Topography, geology, height of bank, ratio of water diverted to that available, channel width, routing of diversion canal, ease of diversion of river during construction, stability of river bank and sides, river protection works governs the selection of the intake location and type. For steeper gradients with straight reaches of river bottom rack intake is more suitable. But in rocky banks, winding river, considerable suspended load it is not desirable. The lateral side intake functions well in such case. Intake sill with 1- 1.5 m is used not to allow bed loads to enter the canals. Trash rack is used to prevent the entry of tree branches, leaves and other coarse materials in the canal. Head is extremely valuable in hydropower projects and design of trash rack should be such that the head loss should be minimum. Suitable factor of safety should be employed to design height of intake sill, to ensure sufficient withdrawal capacity in the future.

5.2.3 PROTECTION WORK

The skimmer wall is constructed to protect the entry of flood water in the canal at the time of high flood. Trash racks are used to prevent the entry of trash matters in the canal. To prevent adverse effect of seepage, sheet pile is used inside the ground below sill.

5.3 GRAVEL TRAP

5.3.1GENERAL

It is necessary to check or trap the particles incoming from the canal intake which would, otherwise, flow in the downstream side and reduce the discharge capacity of canal and ultimately cause the wearing and chocking of the turbine unit. The trap of coarse particle (>2mm) is achieved by means of a hydraulic structure known as gravel trap. During the high flood season, the river carries appreciable amount of gravel hence a gravel trap should be provided to trap the design size of gravel entering through intake.

5.3.2 DESIGN CONSIDERATIONS

Gravel trap should be located at a safe place but as close to the intake as possible or sometimes even within the intake so that debris is not carried a long distance into the waterways. Gravel is checked in gravel trap by allowing water to flow in a wide and deep channel at a slower velocity so as to reduce the capacity of water thereby causing deposition of particles towards bed. Flow velocity of water and settling velocity of the particles affect the settling of the particles. The flow velocity must not exceed the upper limit so as not to allow suspended particles being washed again. For construction easiness, depth is generally limited to 3m; width is calculated to satisfy the velocity of flow and length is calculated to ensure desirable efficiency of settling. As boundary friction is predominant for short width, the effective length may be taken only 85% of provided length.

The flushing of settled particles should be done to ensure proper working. Generally continuous flushing is adopted for gravel trap as the sediment load is high. Gates are used to control flow at flushing orifice at inlet. Sufficient bed slope and cross slope is required to make the flushing effective. Standard methods are used to design the gravel trap. The concentration approach, which is modern and rational approach, is used. Vetter's equation to calculate efficiency is used. Camp's formula is used to calculate the transit velocity and Newton's formula is used to calculate the settling velocity.

Continuous flushing system is used in the gravel trap which works continuously in the monsoon season and can work as intermittent flushing at the time of low flow. 10% water is used for flushing purpose. The flushing orifice is designed on the basis of the head to cause flow.

5.3.3 PROTECTION WORKS

Gates are used to control the flow across the gravel trap. Flushing gates are used to flush the settled matters. The flushing orifices are controlled using the flushing gates. Flushed water and the excess water are safely diverted to the river using open channel. The side protection works fencing etc. are carried out.

5.4 SETTLING BASIN

5.4.1 GENERAL

The suspended particles entered in a canal, if allowed to flow through penstock pipe and turbine, cause abrasion of such units and reduce efficiency as well as durability. In addition, problem of clogging is always present due to such particles in turbine units. There is also the possibility of siltation in canal. So, the finer particles escaped from gravel trap are to be removed before entering into penstock.

The severity of particles depends on effective head of water, hardness of particles, shape of particles and size of pipe, valves opening and turbine blades and opening. It is very difficult to trap all the particles. So, a particular size of particles is selected to make a design basis for Settling Basin. The basin design philosophy is similar to that of gravel trap. Selection of width and length depend on land available. For more reliable operation, more than one chamber is employed. It will not interrupt whole system when it is to be stopped for maintenance. To ensure uniform flow, transitions are provided at inlet and outlet. Both height and width vary gradually inlet transition and width varies in outlet transition.

Flushing of deposited matters is essential for smooth operation of settling basin. The lateral and longitudinal slope may be provided for this purpose. There must be control of flow in and from settling tank. For this purpose gates can be used. A continuous flushing system can operate continuously in wet season when there is sufficient water and excessive sediments. In dry season, when there is clear water in river and water is scare, it can work as intermittent flushing.

5.4.2 DESIGN CONSIDERATION

The settling basin is designed following standard practices. Concentration approach is used to design it. Trap efficiency is selected as 90% removal of 0.2mm sized sedimentary particles. Vetter's equation is used for efficiency calculation. Camp's equation and various charts are used to compute the transit velocity and the settling velocity.

5.4.3 PROTECTION WORKS

Gates are used to control the flow across the settling basin. Flushing gates are used to flush the settled matters. The flushing orifices are controlled using the flushing gates. Flushed water and the excess water are safely diverted to the river using open channel. The side protection works like fencing etc. are carried out.

5.5 FOREBAY

5.5.1 GENERAL

A forebay is a storage basin which is constructed at end of the headrace canal and beginning of the penstock. Its main function is to temporarily store water which is rejected by the plant due to reduced load during off-peak hours and also to meet the instantaneous increased demand when the ground profile changes form slightly sloping to steep.

The design of forebay is similar to that of that of settling basin, in general except that exit portion is replaced by a trash rack and penstock entrance area. The entrance to the penstock should fully submerge in its design. The different parts of the forebay; entrance bay or basin, spillway, flushing sluice, screens, valve chamber and conduit or penstock gate.

5.5.2 DESIGN CONSIDERATION OF FOREBAY

The forebay has been designed for storing the water required for running the turbine for 3 minutes. Stored water is utilized while starting the turbine. The transition canal is provided for lowering the velocity gradually. Forebay is constructed immediately before the inlet of the penstock pipe and started at the end of the headrace canal.

5.5.3 PROTECTION MEASURES OF FOREBAY

The forebay is located at a flat area which has been used as the cultivate area. The top of the structure is above ground level. The downhill is provided with retaining structures to ensure its stability. The uphill side of it is provided with catch drain. The excess water from the forebay is allowed to spill form the spillway structure constructed on it. This water is safely discharge to the river using an open channel constructed for the purpose.

Gates are used at its inlet and outlet for its safe operation

5.6 PENSTOCK

5.6.1 GENERAL

The potential energy of the flow at the inlet chamber is converted into the kinetic energy at the turbine of a hydropower plant via the pipe known as penstock. Water flows under pressure in the penstock. The penstock has to fulfill various serviceability requirements for safe and reliable operation of the plant. It has to bear a very high pressure caused due water hammer effect at the sudden closure of the gate by governing mechanism of the turbine. Penstock should be smooth enough so as to result minimum head loss while flowing water and it should be corrosion resistance from durability aspect. The thickness should be sufficient to resist hoop stress developed by water hammer pressure and normal pressure not exceeding the allowable stress. Penstock alignment must be straight to avoid head loss at bents and the extra cost of anchor block unless it is mandatory by site condition. The penstock may be either embedded or exposed as per topography, location of Surge Tank, Powerhouse and construction easiness etc.

5.6.2 DESIGN CRITERIA FOR PENSTOCK

For a particular head and discharge, there may be several options for the size of penstock according to continuity equation ($Q=A\times V$). Also head loss increases squarely with increase in velocity as per Darcy-Weishbach equation, $h_l = \frac{flv^2}{2gd}$. So, a smaller size penstock saves cost of construction material but the loss of energy due to loss of head takes place and vice versa. Due to this fact, we can deduce as optimum diameter which minimizes the total cost and the same is adopted for the project. Water hammer pressure in excess of normal water pressure can be expressed in equivalent water column height as, $\mathbf{h_m = V_c \times \frac{V_o}{g}}$

Where V_o = Velocity of water in penstock,

$$V_c = \text{Velocity of wave} = \sqrt{\frac{K_m}{\rho}}\ ;\ K_m = \sqrt{\frac{1}{\left(\frac{1}{K} + \frac{D}{tE}\right)}}\ ,$$

Where, K = Bulk Modulus of water

D = Diameter of penstock

t = thickness of penstock

E = Young's Modulus of elasticity of steel

ρ = density of water.

Also, thickness of pipe, $t = \frac{P \times d}{2\sigma}$; Where, P = total pressure in pipe and σ = Permissible hoop stress of steel in pipe.

If the penstock has to feed more than one turbine, various factors govern whether use independent pipes in number equal to the equal to the no. of turbine or use one pipe and bifurcate it at turbine inlet. Length from inlet chamber to powerhouse, construction feasibility, reliability, transportation and fabrication feasibility are some important factors to be considered for this.

5.6.3 OPTIMIZATION

Penstock is one of the costly and important structures in hydropower plant. The larger size incurs more cost of the structure and a smaller size saves the cost of structure but is associated with increased head loss (which is ultimately the power loss). So, there is always an optimum size of penstock for which the total cost of loss and the material is minimum. To seek this size, optimization technique is used. Increase in size tends to increase the thickness, as thickness is directly proportional to diameter but this relation is no more valid as the water hammer pressure decreases with increase in size. The optimization is carried out considering these aspects. Optimization yielded the internal diameter and thickness of the penstock pipe.

5.6.4 PROTECTION WORKS FOR PENSTOCK

Penstock is very sensitive structure and its failure is of fatal nature. Exposed penstock is susceptible to temperature stress and hence, should be provided with expansion joints. Anchor blocks are used to resist vertical and horizontal forces in the penstock. They prevent the yielding of penstock. Expansion joints are provided adjacent to them. To support at intermediate locations and prevent bending stresses, slide blocks are used. The inner surface of penstock is galvanized and the outer surface is frequently painted to prevent from corrosion. Frequent checking of the penstock should be done to ensure its safe operation and to foresee the faults before failure.

5.7 ANCHOR BLOCK AND SUPPORT PIERS

5.7.1 GENERAL

An anchor block is an encasement of penstock designed to restrain the pipe movement and to fix the pipe in place during installation and operation. Anchor blocks tend to prevent the movement of the penstocks due to steady or transient forces including expansion and contraction forces and water hammer pressures. They provide necessary reaction to the dynamic forces at the bends. To provide the necessary degree of stability to the pipe assembly, anchor blocks find their significance. Anchor blocks are provided at all horizontal and vertical bends of the pipe.

Support piers are used to support the pipes at intermediate points so as to prevent excessive bending stresses in the pipe. They resist the weight of the pipe and water and resist the lateral movement but allow the longitudinal movement of the pipe. So, these blocks are lighter in weight than anchor blocks and save the overall cost of the support action.

5.7.2 DESIGN PHILOSOPHY

Water flowing under pressure when diverted from straight path exerts pressure as the bends. To resist various forces these blocks are designed. The blocks act as the massive structures and work as the gravity dams. Sliding, Overturning, tension and crushing are to be checked for the blocks.

5.7.3 PROVISION FOR SUPPORT PIERS

The support engages less than the full perimeter of the penstock, generally between 90 and 180 degrees of arc, and typically 120°. These are simpler to construct than full perimeter ring girder supports, but generally are spaced closer together than the ring girders. It is usually spaced between 6 to 8 m between the anchor blocks. It is constructed of concrete 1:3:6. Design procedure is same as that of the anchor blocks but only the combination of load is different.

5.7.4 PROVISION OF EXPANSION JOINTS

Mechanical joints either expansion joint or bolted sleeve type coupling is used in both exposed and buried penstocks to accommodate the longitudinal movement caused by the temperature changes and to facilitate the construction. The joints shall allow for movement where differential settlement or deflections are anticipated.

Expansion joint permit only the longitudinal movements. The joints are used primarily with aboveground installations and are located between the supports at the points where the penstock deflections are of equal magnitude and direction. These joints divide the barrel shell into separate units, which are watertight, but structurally discontinuous. It should be provided just below the anchor block. Length of the expansion joints $= \alpha \Delta t L$

5.7.5 CONSTRUCTION

Anchor blocks are the support of the penstock and are constructed to meet this purpose. As the penstock is circular, the anchor blocks are made to fit the curve surface. Saddle supports are used in it and a sufficient cover is provided above the pipe for adequate fixity.

5.7.6 MODE OF FAILURE AND SAFETY AGAINST THEM

Anchor blocks are designed similar to the gravity dam. The blocks are to be designed to resist overturning, sliding, crushing and tension failure. A firm foundation is required for the blocks. The blocks should be prevented from gulley erosion due to rain water.

5.8 POWER HOUSE

5.8.1 GENERAL

Power house is one of the major components of the hydropower project. It is used to house the electro-mechanical components. The switch gear, control room, engineer's room, reception room operator's accommodation are generally provided with it. Basically, there are two types of powerhouse i.e. surface and underground powerhouse. Surface power house is cost effective and is best suited when the power house is far away from flood plane. On other hand, the underground powerhouse is located inside the rock mass which makes it more stable against flood effects and other external forces. Due to underground construction and high technological methods, the underground powerhouse is highly costlier than surface ones. Some powerhouses are located as semi-underground structures being partly on surface and partly underground.

5.8.2 COMPONENTS OF POWERHOUSE

I) MACHINE HALL

It is a room in which the generating sets are usually arranged in a single line, the orientation of which will be determined according to the arrangement of the intake or penstock and of the tailrace

II) AN UNLOADING AND ERECTION BAY

It is the bay in which the plant can subsequently be dismantled or reassembled.

III) ANNEXES OF THE EXTENSION TO THE MACHINE HALL TO THE ELECTRICAL EQUIPMENT HOUSE.

IV) PASSAGE OF DUCTS FOR CABLES AND BUS BARS AND PIPES.

V) WORKSHOP WITH BASIC MACHINE TOOLS

5.8.3 POWER HOUSE SIZE

Power house size mainly depends on the discharge, head, type of turbine and generator, number of units and the general arrangement in the power house. The size of the power house should be sufficient to house all the components. Sufficient clear space should be available for installation of various components and for maintenance purpose.

5.8.3.1 HEIGHT OF POWER HOUSE

Height of power house is fixed by the dimensions of lower turbine block and its superstructure. Height of the lower turbine block from the foundation to the floor of the machine hall is to be determined by the thickness of foundation plate, dimensions of the turbine. The height of the power house should be sufficient for the installation of turbine, generator and shaft and gear mechanism. There should be sufficient space for removal and overhaul of any of the components without disturbing other components. Sufficient clear space is also provided for crane operation etc.

5.9 TAILRACE

5.9.1 GENERAL

Tailrace is the final civil structure that conveys the design flow from powerhouse back to the river where it is disposed off. Open channels or pipes can be as tailrace structure. Often adequate attention is not given to the design and construction of the tailrace, probably because it does not affect power production seriously. However, such a practice can result inadequate depth of the tailrace of the tailrace pit or erosion of slopes which could threaten the power house structure.

5.9.2 DESIGN CRITERIA

Design of the tailrace channel is similar to that of headrace channel. Since head loss does not need to be minimized a higher velocity can be allowed in tailrace channel. Note that at higher velocities higher grade of concrete is required to resist erosion. Reinforced concrete may become economical for a steep channel. The downstream end of tailrace must be protected so that there is no danger of erosion either by the river or by the flow from the tailrace. Ideally the discharge should be disposed off over rock or large boulders. If erodible slopes exist in the vicinity if the exit, a stilling basin may be required to dissipate energy.

6.0 COST ESTIMATION

6.1 GENERAL

This section of the report describes the methodology used for derivation of the project cost. The estimate is the final and shall be considered different from the cost estimates as used in the optimization study.

The costing of the project has been carried out on the basis of feasibility study carried out by consultants and experienced gained in this field wherever possible. Current costs of equipment and material have been acquired from manufacturers and suppliers where possible. Where these have not been available, costs have been taken on the basis of past projects carried out as well as unit rate analysis appropriate for hydropower projects.

The cost estimation has been carried out in parallel with construction planning approach as discussed in construction planning section as these two activities are envisioned complementary to each other.

All prices and cost data are calculated in US$ and conversion rate is taken as NRs 75 per US$. To arrive at the total project cost, the quantity of various items is estimated for each work separately in accordance with the related drawings.

6.1.1 UNIT RATE ANALYSIS

Unit rate analysis for the various jobs has been carried out as per the norms published by Ministry of Physical Planning and Works, Government of Nepal. The rates of the locally available materials such as sand, boulders, aggregates, softwood and labors are taken from the approved district rates by District Rate Fixation Committee for the running fiscal year. Regarding electromechanical equipment costs, rates from manufactures/suppliers is sought.

ASSUMPTIONS

The following criteria and assumptions are the basis of the cost estimate:

- The cost estimate and financial analysis have been based on the US dollar.
- The exchange rate used for cost estimate is US $ 1 = NRs 75
- Price level of 2011,

  The cost estimate has been made at the price level of 2011. All costs have been first estimated on unit cost basis for each of the components. These have been added to obtain the entire project cost. Lump sum costs have been allocated for components where a detailed breakdown of costs is not available or worthwhile.
- Material price and labour cost

  Material costs reflect real costs incurred at other projects of similar size or having similar scope of works. The prices have been calculated for 2011. It is assumed that the bulk of the construction material can be obtained in the local market whereas some of the steel items and all of the electromechanical equipment need to be imported.
- Semi-skilled, unskilled and some skilled manpower can be available locally.
- Indirect cost

6.1.2 ENGINEERING AND MANAGEMENT FEES

The engineering and management fees have been allocated as 15% of the total construction cost that may be required for additional studies, all detailed design and construction phase management of the project to be carried out. The cost will cover the following activities:

- Further site investigation such as, topographical survey of access road, and transmission line.
- Preparation of tender stage design and documentation and detailed engineering design.
- Contract and tendering.
- Management of procurement and project administration.
- Reviewing and approval of contractor submittal.
- Associated cost of owner for project management.

6.1.3 CONTINGENCY SUMS

The contingencies shall cover any unforeseen cost that could incur during detailed design phase of the project as well as construction phase of the project. The more information on underground works and foundations beyond the limit of the investigations made during the feasibility study shall be accounted for. The contingency rate for the project has been allocated as 15% of the total cost.

6.1.4 VAT/TAXES AND DUTIES

The amount of VAT payable has been considered as 13% of the total project costs which exclude equipment to be imported from outside country.

Custom/duty, taxes and godown charge is lumped together and taken as 2.6% of the estimated cost of the plant and equipment.

6.1.5 PROJECT COST ESTIMATE

The detail cost estimation of the project is presented in Appendix. The total cost of the project is represented below: 1 US$ = NRs 75

S.N	Description	Total Amount (US$)
1	Civil Works	2 ,832,397.16
2	Access road (LS)	150,000.00
3	Hydro mechanical cost (pipe and gates)	566,479.43
4	Land purchase	100,000.00
5	Electromechanical cost	1,500,000.00
6	Transmission line cost	100,000.00
7	Project development cost	566,479.43
	Sub Total	**5,248,876.59**
	Engineering and Management cost(15% of Sub Total)	787,331.49
	Sub Total	**6,036,208.08**
	Contingency (15% of Sub total)	905,431.21
	Sub Total	**6,941,639.29**
	VAT and Tax (13% of Sub Total)	902,413.11
	Total Cost	**7,844,052.39**

Total cost of the project 7,877,052.39

7.0 ECONOMIC AND FINANCIAL ANALYSIS

7.1 GENERAL

Economic and financial evaluation of the project is carried out in order to determine viability of the project. The financial analysis will evaluate the acceptability of investments made in the HKHP as a source of energy supply from the view point of developers. The technical feasibility of the scheme has been established through study carried out on the technical aspect. Apart from the technical, environmental and socio-economical aspect of the project, the financial analysis provides the most important indicators for the acceptability of the HKHP for investment. The economical and financial evaluation is aimed at giving potential investors in the project an overview of the risks and benefits associated with financing the project.

Financial evaluation uses the real term monetary values of the cost and benefits and is inclusive of taxes transfers, duties and escalation. The financial evaluation concerns with the developer of the project and its impact on its accounts. Hence, from the perspective of a private developer, financial evaluation is the most important aspect of the project to determine whether to finance it or not.

The financial analysis consists of a cash flow during the project life, a financial evaluation, which suggests the payback period, benefit/cost ratio and the internal rate of return (IRR) of the project. The economical analysis of the project has been carried out on the basis of 50 years plant life.

7.2 PROJECT EVALUATION

7.2.1 ASSUMPTIONS

A financial analysis has been carried out for the base case on the basis of the following assumptions:

- Project completion period: 3 years from commencing construction,
- Economic life of the project is 50 years.
- Salvage value of the project at the end of the economic life is zero.
- Annual operation and maintenance cost is estimated 3% of the capital cost.
- Energy selling price is assumed to be NRs 8.40 and 4.80 per kWh for dry energy and wet energy respectively.
- Exchange rate of 1 US$ = NRs 75

7.2.2 PROJECT BENEFITS

For the financial analysis, the principal project benefits are revenues, which can be derived from the operation of the project. In the analysis three important economic indicators such as:

- Payback Period
- Benefit Cost (B/C) ratio and
- Internal Rate of Return (IRR).

The result of the financial analysis has been listed in table below:

7.3 ECONOMIC ANALYSIS

Total cost of the project = NRs.59,07,78,930

Annual O&M = 3% of capital cost

= NRs.1,77,23,370

Annual income = NRs.7,89,16,765

1. Calculation of PayBack Period

PayBack Period = $\dfrac{TotalCost}{AnnualInco\ me}$ = $\dfrac{59{,}07{,}78{,}930}{7{,}89{,}16{,}765} \approx 7.5$ yrs

$\therefore$PayBack Period = 7.5 yrs

2. Calculation of IRR

Using the net present worth NPW = 0

59,07,78,930= 7,89,16,765$\times\left(\dfrac{(i+1)^{50}-1}{i(i+1)^{50}}\right)$

$\therefore i = 0.13$

$\therefore \text{IRR} = 13\%$

3. Calculation of B/C ratio

Assuming MARR = 10%

Modified B/C ratio = $\dfrac{A(B) - A(O\ \&\ M)}{CR}$

A (B) = NRs. 7,89,16,765

A (O&M) = NRs. 1,77,23,370

CR = Capital Recovery

= 59,07,78,930$\times\left(\dfrac{0.1\times(0.1+1)^{50}}{(0.1+1)^{50}-1}\right)$

= NRs.59,58,545

Modified B/C ratio = $\dfrac{7{,}89{,}16{,}765 - 1{,}77{,}23{,}370}{59{,}58{,}545} = 1.03 > 1$

Hence the project is feasible.

The detailed cash flow diagram is shown in Appendix.

8.0 PROJECT PLANNING AND SCHEDULING

8.1 GENERAL

Project generates itself from ideas, which must be technically feasible, economically viable, politically suitable and socially acceptable. With the increasing complexity of larger projects, necessity for better planning and scheduling is increasing. For the success of any project it is necessary that the objectives and time schedules should be defined with reference to attainable targets, taking into account all the problems and difficulties which may be existing at the time of drawing up of the plan or during the course of construction period. The proper planning of any project is essential to achieve the real goal.

8.2 PLANNING

Planning in general is the process of establishing project goals and the ways of achieving the goals. It is a predetermined course of action to be taken in future. Project planning is a decision planning must be systematic, flexible enough to handle unique activities. Comprehensive project planning covers the following areas:

- Planning the project work
- Planning the human resources and organization
- Planning the financial resources
- Planning the information system

Planning aims at achieving the project completion, making the most effective use of time and resources. Project planning requires both the operational and strategic thinking and decision making. It is characterized by creativity, innovation and ability to think rationally and prospectively.

Project planning is a multi stage process and enumerated as:

- Establishment of objectives
- Identify the key factors of the project
- Identification of key elements of projects
- Establishing the logical sequencing of activities.
- Identification of time and resources
- Assignment of responsibilities
- Finalize project plan.

For the successful run of the project, certain development such as access road, temporary camps, facilities for drinking water, light should be provided on the project site before the actual construction starts. The construction work should be started after enough operations are lined up and definite commitments are made for arrival of material and equipments.

8.2.1 PHASE OF CONSTRUCTION

In the hydropower construction, the hydropower plant construction only is not solely a project work. Before the construction of the power plant, infrastructure required for the project such as access road, bridge, temporary camps for works etc should be developed. These all works should be scheduled and proceed on phase wise.

General phase of project construction can be summarized as:

- Access road construction
- Construction of camp
- Construction of all civil works
- Electromechanical works

8.3 PROJECT SCHEDULING

The project scheduling is done immediately after planning work is completed, approved, the budget estimate is prepared and the detailed design, tendering and the master plan is more or less finalized. The schedule of the construction works is a very important aspect of the project as it ensures not only the timely completion of the project to comply with the energy requirements of the nation but also to have a tentative idea on the cash flow patterns of the project. The management of finance as well as other resources like equipments, material, and manpower for the project implementation largely depends on the schedule of construction.

While scheduling the project, the project activities are identified and their proper technological sequences and the anticipated time duration for each of the activities are estimated.

Due to the innumerable activities interdependent on another in the project, it is necessary to make the schedule in a systematic way for easy understanding and reference. The widely used techniques are; a) Bar Chart b) Network Analysis

8.3.1 PROJECT SCHEDULE OF HEWA PROJECT

This section of the report describes the anticipated construction technology that could be applied to undertake different construction site at the possible shortest span of the construction time. As it is envisioned that the construction of the project could be completed within 2 years time but it will depend on the commencement of the construction. If the construction is schedule on season, it is possible to complete the construction in two years period, otherwise off season start delay for another six months. It is therefore envisioned that the construction of the project will be completed within or maximum of two and halves years. Construction schedule has been prepared accordingly for the major construction activities and where possible minor activities areas are also taken into account. Critical activities as well as milestone have been identified. The construction schedule of the project is shown in Appendix.

9.0 CONCLUSION AND RECOMMENDATION

The project is located at BharpaVDC of Panchathar District. All project components lie on right bank of the Hewa Khola and spread within 2.7 km stretches. The Hewa Khola is a snow fed river originates from Higher Himalayan.

The high flood at intake site for 100 years return period is estimated as 372 m^3/sec while the design discharge for 40 percentile is estimated as 7.8 m^3/sec. the total power generated is 3.8 MW and total energy of 16.08 GWh.

The project is expected to be completed at cost of NRs.59,07,78,930 and will be completed within 3 years time. Based on the financial analysis of the project, the project is found to be attractive in economic terms for a minimum selling price of 8.40 NRs/kWh and 4.80 NRs/kWh for dry and wet energy respectively. The payback period of the project is found to be 7.5 yrs and B/C ratio and IRR of the project is found as 1.03 and 13% respectively.

The project is found technically attractive, financially sound and environmental friendly.

The following recommendations are made as:

i. The analysis of various types of hydraulic structures such as weir, intake, gravel trap, settling basin, forebay, penstock should be carefully done.
ii. Detail investigation of construction material with systematic sampling and estimation of quantities of impervious core, sand and coarse aggregate are recommended.
iii. The in situ and laboratory test of rock and soil of various site are recommended to get further geo-technical properties of rock and soil in detail.
iv. It is recommended to carry out detail investigation of flood hazards around proposed project site.

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APPENDICES

HYDROLOGY DATA AND ANALYSIS

Station number : Terathum
Index No. : 1314
Estd date :
District : Terathum

Latitude 2708
Longitude 8733
Elevation 1633
Zone

Monthly and Annual Precipitation

Year	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Annual	Monsoon
1997	20.80	4.80	13.20	118.80	156.90	155.90	137.70	319.20	261.90	19.60	0.00	61.60	1270.40	874.70
1998	0.00	7.60	72.80	102.00	115.20	309.40	217.20	328.90	173.80	3.00	25.40	0.00	1355.30	1029.30
1999	0.00	0.00	2.90	14.40	170.00	354.80	307.90	195.30	48.80	97.20	0.00	10.60	1201.90	906.80
2000	0.00	45.80	4.80	90.70	0.00	128.60	88.20	196.90	61.40	3.00	2.20	0.00	621.60	475.10
2001	100.20	0.00	0.00	124.20	204.60	83.60	165.80	190.90	175.30	126.70	0.00	0.00	1171.30	615.60
2002	142.50	2.50	32.00	69.70	108.80	175.40	297.20	207.20	49.50	0.00	0.00	0.00	1084.80	729.30
2003	14.60	6.30	37.60	87.30	85.10	313.00	278.40	165.10	117.50	69.50	0.00	39.80	1214.20	874.00
2004	14.60	6.30	37.60	87.30	85.10	313.00	278.40	165.10	117.50	69.50	0.00	39.80	1214.20	874.00
2005	0.00	0.00	5.20	33.20	135.60	107.70	209.00	194.60	34.40	0.00	0.00	0.00	719.70	545.70
2006	0.00	0.00	24.80	185.00	130.80	204.00	283.80	278.70	283.30	7.40	0.00	12.20	1410.00	1049.80
Mean	**29.27**	**7.33**	**23.09**	**91.26**	**119.21**	**214.54**	**226.36**	**224.19**	**132.34**	**39.59**	**2.76**	**16.40**	**1126.34**	**797.43**
Max	142.50	45.80	72.80	185.00	204.60	354.80	307.90	328.90	283.30	126.70	25.40	61.60	1410.00	1049.80
Min	0.00	0.00	0.00	14.40	0.00	83.60	88.20	165.10	34.40	0.00	0.00	0.00	621.60	475.10

Station number : Legang
Index No. : 1326
Estd date :
District : Morang

Latitude 2644
Longitude 8730
Elevation 250
Zone

Monthly and Annual Precipitation

Year	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Annual	Monsoon
2001	0.00	0.00	7.40	80.70	207.20	356.40	655.90	442.40	519.80	367.40	1.80	0.00	2639.00	1974.50
2002	48.30	2.60	29.20	64.10	181.10	189.60	1154.00	331.30	203.50	95.70	0.00	0.10	2299.50	1878.40
2003	31.50	37.30	49.60	89.30	88.70	476.10	914.20	319.10	257.70	192.30	11.30	34.50	2501.60	1967.10
2004	30.00	0.00	19.70	207.10	81.70	316.20	958.20	246.50	476.70	276.00	0.00	0.00	2612.10	1997.60
2005	26.60	6.00	55.60	69.90	100.70	355.70	398.90	766.50	158.70	92.30	0.00	0.00	2030.90	1679.80
2006	0.00	3.00	5.50	80.30	160.80	327.50	373.20	273.30	472.60	232.50	1.00	0.00	1929.70	1446.60
Mean	**22.73**	**8.15**	**27.83**	**98.57**	**136.70**	**336.92**	**742.40**	**396.52**	**348.17**	**209.37**	**2.35**	**5.77**	**2335.47**	**1824.00**
Max	48.30	37.30	55.60	207.10	207.20	476.10	1154.00	766.50	519.80	367.40	11.30	34.50	2639.00	1997.60
Min	0.00	0.00	5.50	64.10	81.70	189.60	373.20	246.50	158.70	92.30	0.00	0.00	1929.70	1446.60

Station number : Lungthung
Index No. : 1403
Estd date :
District : Taplejung

Latitude	2733
Longitude	8747
Elevation	1780
Zone	

Monthly and Annual Precipitation

Year	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Annual	Monsoon
1997	23.70	21.00	95.50	161.40	55.30	246.00	546.10	492.20	478.60	60.80	22.10	63.20	2265.90	1762.90
1998	0.00	28.40	65.30	80.20	155.10	382.20	559.00	572.20	346.80	289.00	25.80	2.20	2506.20	1860.20
1999	9.80	3.00	38.80	122.50	365.80	402.40	654.80	527.40	457.10	198.70	9.10	2.00	2791.40	2041.70
2000	12.50	18.00	63.60	89.70	268.10	375.10	736.50	714.80	322.60	49.00	11.80	0.00	2661.70	2149.00
2001	3.70	68.30	59.50	216.70	246.30	347.90	425.90	616.60	444.60	192.80	9.10	4.00	2635.40	1835.00
2002	20.70	8.40	95.60	186.50	178.60	362.30	622.70	586.90	229.30	54.20	1.10	4.90	2351.20	1801.20
2003	27.20	90.20	124.60	125.20	159.10	265.60	806.60	660.90	556.90	151.50	32.50	0.80	3001.10	2290.00
2004	28.50	8.90	27.20	97.20	241.90	300.10	543.00	664.70	402.80	321.90	4.00	2.30	2642.50	1910.60
2005	33.90	42.80	61.40	55.70	187.70	254.10	574.90	678.70	283.80	77.30	1.00	0.00	2251.30	1791.50
2006	0.00	31.60	77.20	59.30	190.80	356.50	376.10	390.60	282.50	69.10	13.50	14.20	1861.40	1405.70
Mean	**16.00**	**32.06**	**70.87**	**119.44**	**204.87**	**329.22**	**584.56**	**590.50**	**380.50**	**146.43**	**13.00**	**9.36**	**2496.81**	**1884.78**
Max	33.90	90.20	124.60	216.70	365.80	402.40	806.60	714.80	556.90	321.90	32.50	63.20	3001.10	2290.00
Min	0.00	3.00	27.20	55.70	55.30	246.00	376.10	390.60	229.30	49.00	1.00	0.00	1861.40	1405.70

Station number : Taplethok
Index No. : 1404
Estd date :
District : Taplejung

Latitude	2721
Longitude	8740
Elevation	1732
Zone	

Monthly and Annual Precipitation

Year	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Annual	Monsoon
1997	25.50	51.20	41.00	222.70	183.10	427.90	503.70	677.10	620.20	35.90	10.30	66.80	2865.40	2228.90
1998	0	28.2	126.9	114.8	225.8	469.8	623.3	779.2	385.8	185.1	11.1	0	2950.00	2258.10
1999	8.00	0.00	29.50	138.20	425.70	413.50	808.60	686.30	450.40	209.10	10.70	15.50	3195.50	2358.80
2000	9.20	15.70	66.80	197.80	287.10	381.20	679.80	897.10	325.50	74.40	10.30	0.00	2944.90	2283.60
2001	5.80	36.80	56.30	256.30	335.40	418.10	419.10	535.90	496.30	160.70	26.10	12.40	2759.20	1869.40
2002	21.10	9.60	94.50	121.00	87.40	207.70	528.20	458.00	90.90	23.30	0.00	0.00	1641.70	1284.80
Mean	**11.60**	**23.58**	**69.17**	**175.13**	**257.42**	**386.37**	**593.78**	**672.27**	**394.85**	**114.75**	**11.42**	**15.78**	**2726.12**	**2047.27**
Max	25.50	51.20	126.90	256.30	425.70	469.80	808.60	897.10	620.20	209.10	26.10	66.80	3195.50	2358.80
Min	0.00	0.00	29.50	114.80	87.40	207.70	419.10	458.00	90.90	23.30	0.00	0.00	1641.70	1284.80

Station number : Himaligaun
Index No. : 1410
Estd date :
District : Ilam

Latitude	2653
Longitude	8802
Elevation	1654
Zone	

Monthly and Annual Precipitation

Year	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Annual	Monsoon
1997	0.00	5.00	44.00	106.60	151.50	415.70	593.20	375.80	516.50	25.30	3.00	10.50	2247.10	1901.20
1998	0.00	4.30	144.50	197.50	119.10	355.50	909.20	626.20	278.80	85.10	14.20	0.00	2734.40	2169.70
1999	4.20	0.00	2.00	51.60	222.60	506.30	701.30	751.10	329.50	143.20	0.00	8.00	2719.80	2288.20
2000	0.00	18.70	0.00	104.10	370.70	395.30	580.40	513.10	268.30	23.00	6.50	0.00	2280.10	1757.10
2001	0.00	13.00	6.50	62.20	290.00	358.20	400.00	347.40	368.20	366.10	0.00	0.00	2211.60	1473.80
2002	36.80	0.00	39.20	77.50	114.00	369.30	919.90	467.90	287.90	7.30	2.10	0.00	2321.90	2045.00
2003	18.30	39.50	65.40	75.90	106.30	717.50	963.90	429.60	236.90	118.50	8.80	39.00	2819.60	2347.90
2004	26.00	0.00	41.00	129.20	199.20	559.90	627.00	156.50	241.40	58.70	0.00	0.00	2038.90	1584.80
2005	17.30	8.50	25.40	98.30	40.00	397.60	487.00	678.70	100.90	112.40	0.00	0.00	1966.10	1664.20
2006	0.00	5.00	44.00	106.60	151.50	415.70	593.20	375.80	516.50	25.30	3.00	10.50	2247.10	1901.20
Mean	**10.26**	**9.40**	**41.20**	**100.95**	**176.49**	**449.10**	**677.51**	**472.21**	**314.49**	**96.49**	**3.76**	**6.80**	**2358.66**	**1913.31**
Max	36.80	39.50	144.50	197.50	370.70	717.50	963.90	751.10	516.50	366.10	14.20	39.00	2819.60	2347.90
Min	0.00	0.00	0.00	51.60	40.00	355.50	400.00	156.50	100.90	7.30	0.00	0.00	1966.10	1473.80

Station number : Sanischare
Index No. : 1415
Estd date :
District : Jhapa

Latitude	2641
Longitude	8758
Elevation	168
Zone	

Monthly and Annual Precipitation

Year	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Annual	Monsoon
1997	0.00	0.00	7.60	33.80	194.00	354.20	689.70	356.60	364.20	246.60	4.80	5.00	2256.50	1764.70
1998	0.00	1.20	122.40	107.20	83.20	756.60	1052.00	872.40	342.40	88.20	0.00	0.00	3425.60	3023.40
1999	0.00	0.00	7.00	66.00	140.20	439.30	661.00	1005.00	384.00	162.60	34.50	0.00	2899.60	2489.30
2000	0.00	16.60	0.00	166.70	431.20	627.60	825.40	762.90	177.00	73.60	23.20	0.00	3104.20	2392.90
2001	0.00	12.60	9.00	60.60	258.40	372.40	555.40	424.10	535.20	518.90	109.60	0.00	2856.20	1887.10
2002	49.80	0.00	32.80	83.00	184.00	415.40	1135.70	314.80	226.60	129.00	0.00	0.00	2571.10	2092.50
2003	10.00	53.50	30.20	46.40	62.60	545.20	1221.20	515.00	408.60	244.70	0.00	0.00	3137.40	2690.00
2004	22.60	0.00	13.40	130.60	112.80	492.40	986.00	392.10	330.30	252.40	0.00	6.60	2739.20	2200.80
2005	34.20	8.60	43.30	74.00	99.00	231.20	458.80	639.80	110.20	92.40	0.00	6.60	1798.10	1440.00
2006	0.00	0.00	7.60	33.80	194.00	354.20	689.70	356.60	364.20	246.60	4.80	5.00	2256.50	1764.70

Mean	**11.66**	**9.25**	**27.33**	**80.21**	**175.94**	**458.85**	**827.49**	**563.93**	**324.27**	**205.50**	**17.69**	**2.32**	**2704.44**	**2174.54**
Max	49.80	53.50	122.40	166.70	431.20	756.60	1221.20	1005.00	535.20	518.90	109.60	6.60	3425.60	3023.40
Min	0.00	0.00	0.00	33.80	62.60	231.20	458.80	314.80	110.20	73.60	0.00	0.00	1798.10	1440.00

Station number : Memeng jagat
Index No. : 1406 Representative station
Estd date :
District : Panchthar

Latitude 2712
Longitude 8756
Elevation 1830
Zone

Monthly and Annual Precipitation

Year	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Annual	Monsoon
1997	19.10	34.70	26.10	112.00	169.00	421.90	369.50	348.80	437.70	36.00	2.30	138.50	2115.60	1577.90
1998	2.50	6.80	83.70	179.30	232.50	381.80	519.90	617.20	281.60	43.70	13.20	0.40	2362.60	1800.50
1999	9.60	0.00	6.40	47.30	307.80	291.80	366.40	489.30	214.90	194.50	9.60	26.70	1964.30	1362.40
2000	34.80	0.30	16.80	101.70	368.50	290.20	377.90	591.70	207.80	79.70	14.60	0.00	2084.00	1467.60
2001	0.00	27.10	23.50	164.70	403.20	201.90	200.70	387.30	304.00	148.80	11.50	1.70	1874.40	1093.90
2002	59.70	6.80	97.60	221.40	333.30	268.80	721.40	362.10	198.00	37.10	7.30	14.50	2328.00	1550.30
2003	65.60	77.20	60.70	134.00	106.20	652.80	583.80	521.70	385.10	71.40	0.00	38.20	2696.70	2143.40
2004	41.90	16.20	33.40	310.90	191.40	312.50	422.80	276.70	356.90	130.00	10.00	0.00	2102.70	1368.90
2005	1.10	48.00	105.30	80.10	170.50	243.50	403.10	500.30	142.40	129.00	1.20	0.00	1824.50	1289.30
2006	0.00	12.50	21.70	142.80	201.80	336.00	359.50	439.80	383.20	70.50	9.00	29.50	2006.30	1518.50
Mean	**23.43**	**22.96**	**47.52**	**149.42**	**248.42**	**340.12**	**432.50**	**453.49**	**291.16**	**94.07**	**7.87**	**24.95**	**2135.91**	**1517.27**
Max	65.60	77.20	105.30	310.90	403.20	652.80	721.40	617.20	437.70	194.50	14.60	138.50	2696.70	2143.40
Min	0.00	0.00	6.40	47.30	106.20	201.90	200.70	276.70	142.40	36.00	0.00	0.00	1824.50	1093.90

Station number : Damak
Index No. : 1408
Estd date :
District : Jhapa

Latitude 2640
Longitude 8742
Elevation 163
Zone

Monthly and Annual Precipitation

Year	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Annual	Monsoon
1997	0.00	10.80	18.00	66.90	133.40	369.80	428.30	266.30	690.60	2.00	0.00	58.50	2044.60	1755.00
1998	94.20	5.40	85.50	122.10	151.30	522.80	919.40	834.20	153.00	148.10	22.90	0.00	3058.90	2429.40
1999	0.00	0.00	0.00	12.80	243.60	372.00	843.90	798.30	372.00	81.30	16.90	0.00	2740.80	2386.20
2000	21.00	17.70	0.00	122.50	349.80	740.60	600.30	888.50	228.00	61.30	30.50	0.00	3060.20	2457.40
2001	0.00	0.00	16.30	79.00	174.60	179.10	282.00	437.00	582.00	462.00	13.60	0.00	2225.60	1480.10
2002	64.20	3.10	35.20	125.30	137.30	313.00	931.70	94.60	124.60	49.20	0.00	0.00	1878.20	1463.90
2003	10.30	34.80	28.90	49.90	113.30	364.30	791.70	244.50	240.10	102.50	0.00	69.50	2049.80	1640.60

2004	32.00	0.00	12.50	112.60	198.70	329.90	766.90	220.40	437.30	199.20	1.80	0.00	2311.30	1754.50
2005	9.70	7.60	80.90	54.30	76.40	214.30	273.30	748.10	113.90	77.10	0.00	0.00	1655.60	1349.60
2006	0.00	16.30	7.40	105.80	160.90	383.10	426.10	173.30	354.60	115.50	19.50	0.00	1762.50	1337.10
Mean	**23.14**	**9.57**	**28.47**	**85.12**	**173.93**	**378.89**	**626.36**	**470.52**	**329.61**	**129.82**	**10.52**	**12.80**	**2278.75**	**1805.38**
Max	94.20	34.80	85.50	125.30	349.80	740.60	931.70	888.50	690.60	462.00	30.50	69.50	3060.20	2457.40
Min	0.00	0.00	0.00	12.80	76.40	179.10	273.30	94.60	113.90	2.00	0.00	0.00	1655.60	1337.10

Station number : Kanyam tea state
Index No. : 1416
Estd date :
District : Ilam

Latitude	2652
Longitude	8804
Elevation	1678
Zone	

Monthly and Annual Precipitation

Year	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Annual	Monsoon
1972	0.00	0.00	0.00	0.00	168.70	427.20	780.50	671.00	569.50	80.00	15.00	0.00	2711.90	2448.20
1973	18.50	25.00	10.00	13.00	304.50	731.00	504.00	543.50	373.50	504.00	45.00	2.00	3074.00	2152.00
1974	29.00	0.00	34.00	100.00	215.00	417.00	1314.00	708.00	404.00	88.00	0.00	12.50	3321.50	2843.00
1975	10.00	9.50	3.00	62.50	246.00	1006.50	1096.00	333.00	669.00	140.50	0.00	1.00	3577.00	3104.50
1976	54.50	66.00	0.00	79.50	167.00	502.00	493.50	1014.50	452.50	30.50	4.00	0.00	2864.00	2462.50
1977	7.00	9.00	53.50	153.90	243.40	279.60	720.20	615.50	373.50	238.40	66.20	57.00	2817.20	1988.80
1978	11.00	15.50	91.80	65.50	270.40	675.50	851.40	576.40	407.10	46.60	28.60	1.70	3041.50	2510.40
1979	5.10	72.40	1.50	130.80	212.00	357.70	1054.20	524.20	258.10	179.00	22.40	196.50	3013.90	2194.20
1980	5.70	25.10	43.40	14.60	240.80	489.00	896.30	686.50	396.10	178.20	0.00	0.00	2975.70	2467.90
1981	30.80	13.60	101.30	169.60	415.10	374.90	1003.30	616.60	317.00	0.50	8.60	0.00	3051.30	2311.80
1982	0.00	18.60	44.50	180.40	71.50	555.00	532.30	259.00	414.60	138.40	22.10	0.00	2236.40	1760.90
1983	24.50	36.70	12.60	51.10	266.10	584.10	1283.30	356.60	557.00	60.40	0.00	17.40	3249.80	2781.00
1984	32.90	29.00	3.60	204.30	233.20	899.80	925.60	441.50	813.00	79.20	0.00	0.70	3662.80	3079.90
1985	0.00	26.30	36.90	51.50	139.50	305.80	1151.30	430.90	411.50	586.20	7.80	41.40	3189.10	2299.50
1986	0.70	1.10	8.60	163.20	192.80	521.70	526.00	587.90	839.70	94.20	21.00	5.50	2962.40	2475.30
1987	3.90	38.60	94.00	89.60	170.10	409.50	1152.50	1356.50	622.70	325.90	0.70	19.70	4283.70	3541.20
1988	6.80	32.20	77.60	62.80	253.80	451.50	1115.20	1229.10	339.50	9.40	0.00	3.80	3581.70	3135.30
1989	56.40	52.50	47.90	17.50	644.00	852.60	779.30	495.10	909.10	98.90	38.10	18.20	4009.60	3036.10
1990	1.00	84.60	96.20	113.90	360.50	549.50	809.30	600.50	732.60	137.90	0.00	0.00	3486.00	2691.90
1991	69.00	0.00	57.40	35.30	145.50	919.10	578.20	554.30	983.60	6.60	0.00	37.60	3386.60	3035.20
1992	15.90	42.30	0.00	79.10	120.20	584.20	901.60	517.60	321.50	130.80	0.00	6.10	2719.30	2324.90
1993	24.70	5.90	44.00	52.50	317.30	504.90	663.20	678.80	481.10	78.00	36.60	0.00	2887.00	2328.00
1994	60.20	29.90	27.80	26.20	128.10	645.40	507.90	488.30	350.30	0.00	0.00	0.20	2264.30	1991.90
1995	12.10	23.80	43.20	27.60	115.20	870.30	996.60	583.20	833.70	24.90	319.90	20.20	3870.70	3283.80
1996	52.70	6.60	6.80	49.20	183.80	663.30	1214.60	780.90	251.00	227.80	0.00	0.00	3436.70	2909.80

1997	5.00	16.00	19.00	139.20	149.90	446.00	556.50	830.50	862.60	2.60	127.20	81.40	3235.90	2695.60
1998	0.00	0.60	201.70	243.10	158.60	578.20	1226.60	736.10	352.20	57.30	24.60	14.60	3593.60	2893.10
1999	0.00	0.00	0.00	112.20	444.00	660.70	521.50	0.00	0.00	0.00	0.00	0.00	1738.40	1182.20
2000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	13.40	29.20	14.60	0.00	57.20	13.40
2001	0.00	28.60	6.40	74.20	301.20	624.10	421.20	383.00	338.40	286.40	0.00	0.00	2463.50	1766.70
2002	31.60	2.00	47.20	111.00	147.00	685.60	1152.40	385.10	182.40	75.40	0.00	0.00	2819.70	2405.50
2003	12.00	8.70	49.20	67.40	167.60	0.00	0.00	602.90	332.30	212.20	0.80	27.80	1480.90	935.20
2004	28.00	0.40	30.20	129.00	207.60	622.00	713.30	282.90	296.20	119.40	0.00	0.00	2429.00	1914.40
2005	19.00	12.00	53.50	59.10	80.40	425.60	668.20	738.10	134.40	145.00	0.00	0.00	2335.30	1966.30
2006	0.00	5.20	22.60	117.30	200.80	430.60	688.00	604.40	723.00	21.00	3.80	23.80	2840.50	2446.00
Mean	**17.94**	**21.08**	**39.13**	**87.03**	**219.47**	**544.28**	**794.23**	**577.50**	**466.17**	**126.65**	**23.06**	**16.83**	**2933.37**	**2382.18**
Max	69.00	84.60	201.70	243.10	644.00	1006.50	1314.00	1356.50	983.60	586.20	319.90	196.50	4283.70	3541.20
Min	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	57.20	13.40

Station number	: Phidim(Panchthar)	Latitude	2709
Index No.	: 1419	Longitude	8745
Estd date	:	Elevation	1205
District	: Panchther	Zone	

Monthly and Annual Precipitation

Year	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Annual	Monsoon
1982	0.00	13.80	27.70	114.10	78.40	168.60	330.20	199.90	131.90	38.50	43.20	0.00	1146.30	830.60
1983	15.40	6.50	11.60	89.80	153.80	147.90	407.80	229.40	198.90	54.00	0.00	36.60	1351.70	984.00
1984	11.60	12.70	15.20	124.00	216.70	223.60	332.70	184.00	331.20	4.20	0.00	1.70	1457.60	1071.50
1985	8.40	12.50	4.00	46.50	193.70	136.10	422.00	179.90	213.80	92.30	20.80	28.30	1358.30	951.80
1986	0.00	10.20	22.50	128.10	122.90	87.70	377.60	269.90	211.90	71.70	9.80	12.40	1324.70	947.10
1987	0.00	25.90	94.40	107.20	97.70	157.30	289.60	311.70	298.50	179.80	0.00	7.00	1569.10	1057.10
1988	0.00	21.00	73.80	64.30	118.10	96.30	281.80	449.40	113.80	14.10	9.00	17.30	1258.90	941.30
1989	79.20	52.90	74.90	14.70	192.80	253.70	446.40	189.60	236.60	3.70	3.40	0.00	1547.90	1126.30
1990	0.00	74.00	84.30	81.40	265.40	257.50	339.20	299.10	225.80	18.20	0.00	0.00	1644.90	1121.60
1991	39.60	0.00	49.20	60.90	243.50	346.60	255.70	435.00	339.50	0.00	0.00	6.30	1776.30	1376.80
1992	0.00	14.70	0.00	45.80	81.80	228.70	341.50	239.30	171.70	50.40	0.00	37.10	1211.00	981.20
1993	26.60	8.70	24.50	122.40	132.80	129.00	235.40	370.00	175.10	52.10	9.00	0.00	1285.60	909.50
1994	38.00	32.50	38.00	67.30	60.50	185.50	201.00	207.50	150.70	0.00	20.50	0.00	1001.50	744.70
1995	10.80	28.90	25.70	48.20	64.90	227.20	469.20	230.20	111.50	14.50	127.50	13.50	1372.10	1038.10
1996	35.50	8.00	31.50	26.70	239.20	230.80	347.20	303.20	167.20	40.50	0.00	0.00	1429.80	1048.40
1997	21.00	4.70	17.80	94.20	148.50	115.80	248.70	397.40	235.90	25.80	3.80	52.80	1366.40	997.80
1998	5.00	66.80	127.50	127.50	178.80	182.10	407.50	352.20	180.90	18.50	15.50	0.00	1662.30	1122.70
1999	0.00	0.00	4.50	32.50	151.80	283.70	394.70	308.20	210.00	66.00	0.00	7.00	1458.40	1196.60

2000	0.00	28.20	1.50	71.20	164.60	163.00	370.90	243.90	121.70	135.00	20.00	49.50	1369.50	899.50
2001	0.00	33.40	6.60	0.00	135.70	174.80	128.30	307.40	197.20	108.10	0.00	0.00	1091.50	807.70
2002	43.10	5.40	64.50	79.20	141.40	146.50	425.50	390.70	49.00	12.10	0.00	0.00	1357.40	1011.70
2003	22.30	51.70	86.20	85.00	90.20	379.60	346.60	271.00	192.50	88.30	0.00	22.60	1636.00	1189.70
2004	16.30	9.30	32.40	66.80	159.00	219.80	292.40	124.60	82.00	32.10	2.80	0.00	1037.50	718.80
2005	20.50	6.50	40.80	53.10	142.00	125.10	287.00	438.80	77.60	43.40	0.00	0.00	1234.80	928.50
2006	0.00	5.20	15.10	104.80	118.40	144.80	284.90	320.80	241.00	61.80	4.20	9.70	1310.70	991.50
2007	0.00	95.40	29.50	155.80	112.40	178.40	406.60	191.00	209.60	56.80	2.00	0.00	1437.50	985.60
2008	4.60	0.00	40.30	53.10	132.20	243.60	305.50	378.90	69.70	47.90	0.00	0.60	1276.40	997.70
Mean	**14.74**	**23.29**	**38.67**	**76.47**	**145.82**	**193.84**	**332.44**	**289.74**	**183.16**	**49.25**	**10.80**	**11.20**	**1369.41**	**999.18**
Max	79.20	95.40	127.50	155.80	265.40	379.60	469.20	449.40	339.50	179.80	127.50	52.80	1776.30	1376.80
Min	0.00	0.00	0.00	0.00	60.50	87.70	128.30	124.60	49.00	0.00	0.00	0.00	1001.50	718.80

Station number : TAPLEJUNG
Index No. : 1405
Estd date :
District :

Latitude	2721
Longitude	8740
Elevation	1732
Zone	

Monthly and Annual Precipitation

Year	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Annual	Monsoon
1997	31.00	52.60	22.20	178.00	168.90	434.40	388.10	402.50	309.60	46.60	4.50	56.00	2094.40	1534.60
1998	8.40	13.10	118.50	189.60	204.70	296.60	357.90	543.00	244.50	102.40	22.30	0.00	2101.00	1442.00
1999	9.50	0.00	18.20	77.80	252.50	274.80	485.20	321.90	295.90	243.70	3.30	0.90	1983.70	1377.80
2000	11.80	34.20	30.00	131.40	256.00	150.30	495.30	514.60	190.40	52.20	6.40	1.50	1874.10	1350.60
2001	0.90	26.60	45.80	187.60	215.10	231.00	335.60	469.70	215.90	172.40	9.10	2.70	1912.40	1252.20
2002	50.00	6.30	90.30	159.00	232.50	231.80	606.10	523.60	246.20	27.00	0.00	0.00	2172.80	1607.70
2003	34.80	103.10	69.80	207.70	199.30	558.20	595.50	324.20	281.00	86.70	0.00	44.70	2505.00	1758.90
2004	21.00	13.20	30.50	228.40	222.40	253.80	344.70	305.70	224.40	86.20	15.00	1.00	1746.30	1128.60
2005	69.00	0.00	57.40	35.30	145.50	919.10	578.20	554.30	983.60	6.60	0.00	37.60	3386.60	3035.20
2006	0.00	7.50	35.00	114.40	205.30	380.30	255.30	693.90	351.00	75.90	12.80	15.40	2146.80	1680.50
2007	0.00	118.40	39.00	153.60	133.40	255.90	515.60	323.30	346.90	155.50	13.00	0.40	2055.00	1441.70
Mean	**21.49**	**34.09**	**50.61**	**151.16**	**203.24**	**362.38**	**450.68**	**452.43**	**335.40**	**95.93**	**7.85**	**14.56**	**2179.83**	**1600.89**
Max	69.00	118.40	118.50	228.40	256.00	919.10	606.10	693.90	983.60	243.70	22.30	56.00	3386.60	3035.20
Min	0.00	0.00	18.20	35.30	133.40	150.30	255.30	305.70	190.40	6.60	0.00	0.00	1746.30	1128.60

METHODOLOGIES FOR ESTIMATING

HYDROLOGIC CHARACTERISTICS

OF

UNGAUGED LOCATIONS IN NEPAL

by The Water and Energy Commission Secretariat and
The Department of Hydrology and Meteorology

INPUT DATA:

Note: Enter the data in the green box provided

1. Physiographic Data:

RIVER NAME :	**Hewa Khola**	
LOCATION :	**Intake**	
DRAINAGE BASIN AREA :	**221**	km^2
AREA OF BASIN BELOW 5000 m ELEVATION :	**221**	km^2
AREA OF BASIN BELOW 3000 m ELEVATION :	**204.1**	km^2

2. Climatologic Data:

MONSOON WETNESS INDEX AT BASIN CENTROID :	**1500**	mm

|:: **OUTPUT:**
Hewa Khola River at Intake

LOW FLOW STATISTICS

****

Return Period (yrs)	Duration	Low Flow Discharge (m^3/s)
2	1 - day	1.65
	7 - days	1.74
	30 - days	2.03
	Monthly	2.17
10		
	1 - day	1.03
	7 - days	1.15
	30 - days	1.44
	Monthly	1.58
20		
	1 - day	0.87
	7 - days	1.03
	30 - days	1.32
	Monthly	1.45

|::

Hewa Khola River at Intake

FLOOD FLOW STATISTICS
****

Return Period (yrs)	Flood Discharge (m^3/s)	
	Daily	**Instantaneous**
2	130	201
5	190	321
10	232	409
20	273	500
50	329	627
100	372	729
200	417	837
500	477	989
1000	525	1112
5000	644	1427
10000	698	1575

|::

Hewa Khola River at Intake

LONG TERM AVERAGE DISCHARGES

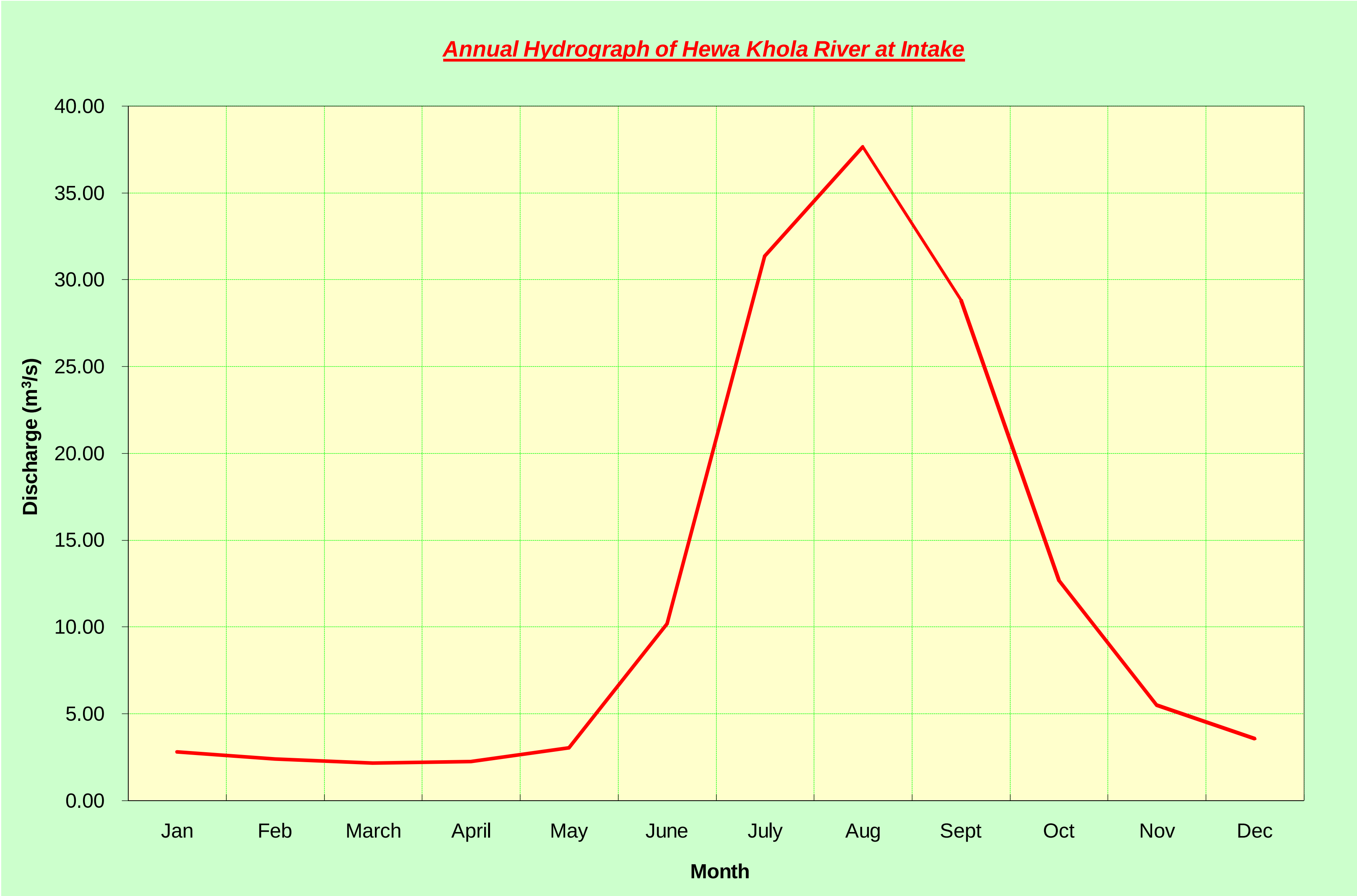
****

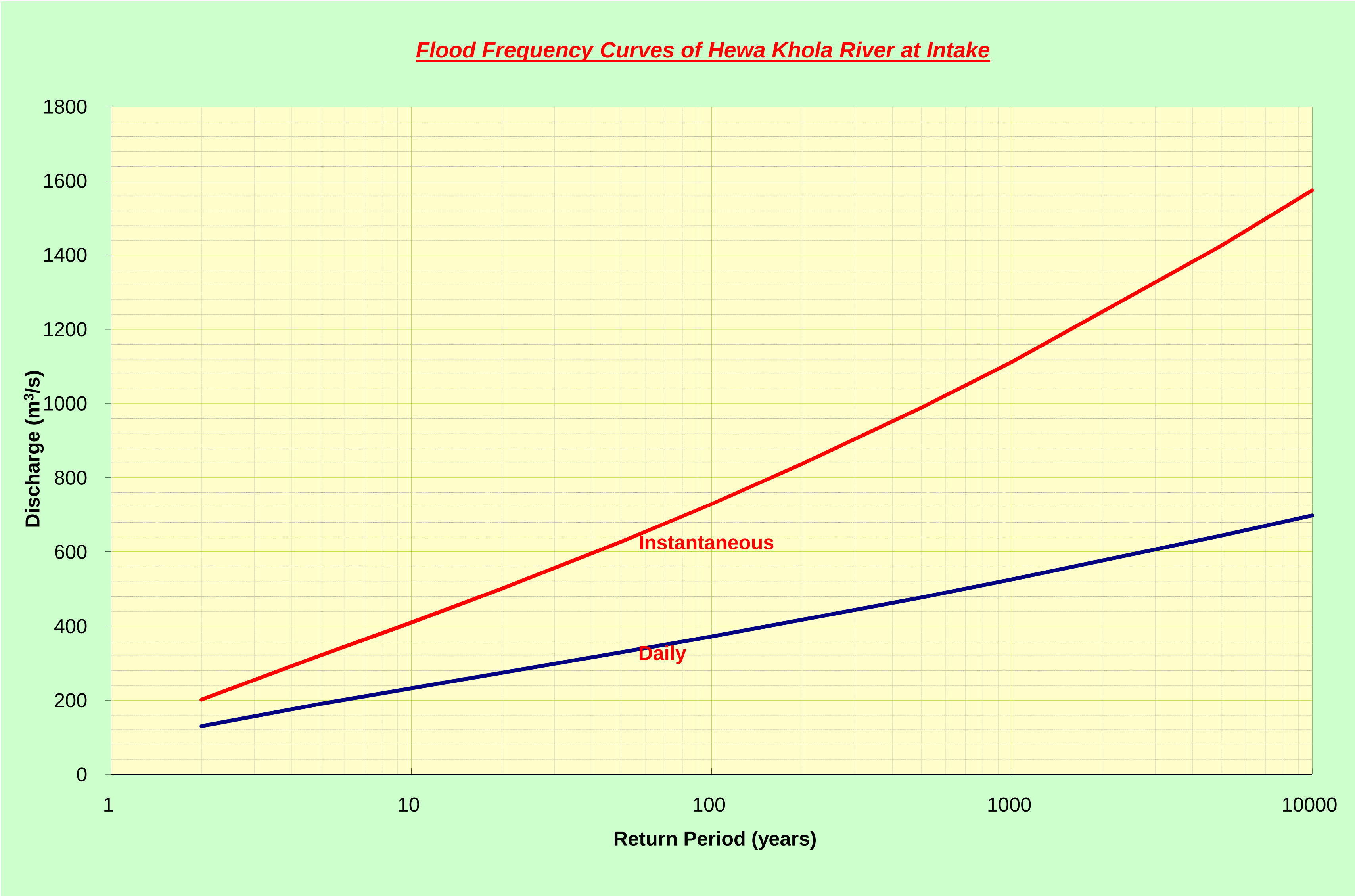
Month	Long Term Average Discharge (m^3/s)
January	2.80
February	2.38
March	2.16
April	2.24
May	3.04
June	10.17
July	31.37
August	37.67
September	28.85
October	12.67
November	5.48
December	3.57
Annual	**11.87**

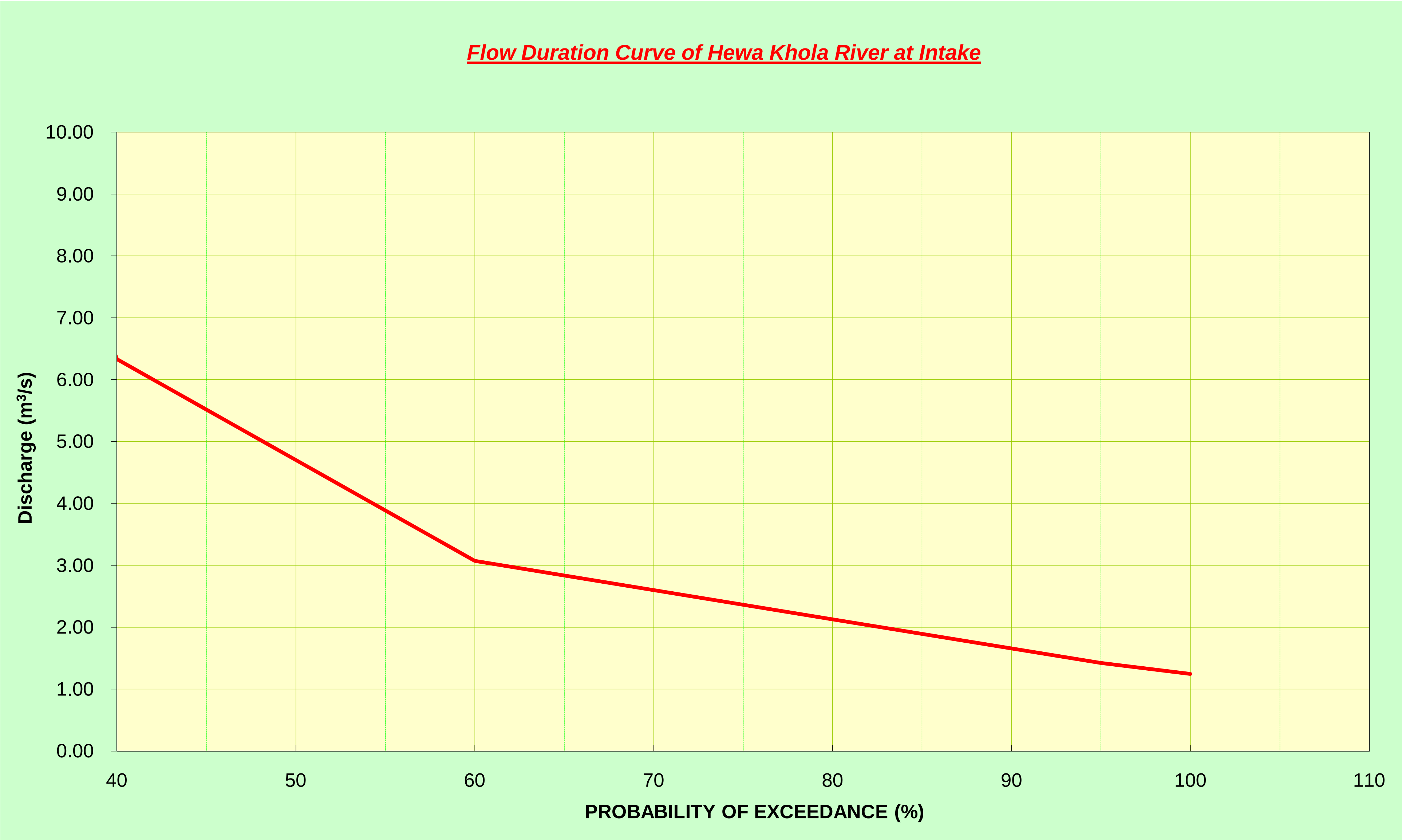
FLOW DURATION CURVE

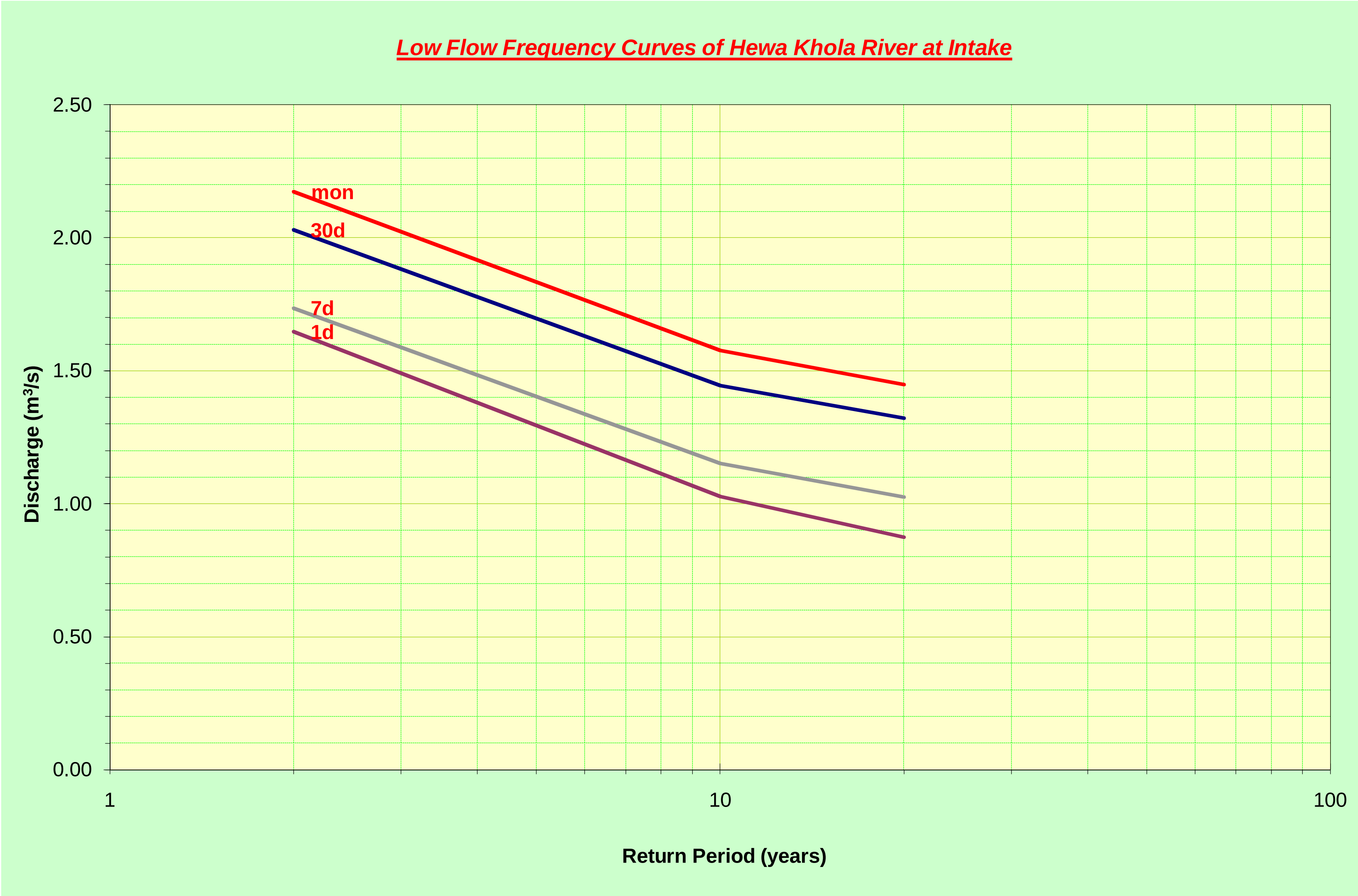
****

Probability of Exceedance (%)	Discharge (m^3/s)
0	72.04
5	42.82
20	22.00
40	6.33
60	3.07
80	2.13
95	1.42
100	1.24


Annual Hydrograph of Hewa Khola River at Intake
Discharge (m³/s)
40.00
35.00
30.00
25.00
20.00
15.00
10.00
5.00
0.00
Jan
Feb
March
April
May
June
July
Aug
Sept
Oct
Nov
Dec
Month


Flood Frequency Curves of Hewa Khola River at Intake
1800
1600
1400
1200
1000
800
600
400
200
0
Discharge (m3/s)
Instantaneous
Daily
1
10
100
1000
10000
Return Period (years)


Flow Duration Curve of Hewa Khola River at Intake
Discharge (m³/s)
10.00
9.00
8.00
7.00
6.00
5.00
4.00
3.00
2.00
1.00
0.00
40
50
60
70
80
90
100
110
PROBABILITY OF EXCEEDANCE (%)



Low Flow Frequency Curves of Hewa Khola River at Intake
2.50
2.00
1.50
1.00
0.50
0.00
Discharge (m³/s)
mon
30d
7d
1d
1
10
100
Return Period (years)

NOTE: Blue color:Input data

Study type	Prefeasibility study	
Catchment or River name	Hewa Khola	
Catchment or River location	Intake	
Basin Area	221	km^2
Basin area below 5000 m elevation level	221	km^2
Basin area below 3000 m elevation level	204.1	km^2
Average Basin Elevation	2300	masl
Annual Wetness Index	1500	mm

See monsoon index map

This calculation is based on WECS/Department of Hydrology and Meteorology (DHM) method developed by The Department of Hydrology and Meteorology, Nepal; and is applicable only for prefeasibility or Reconnaisance study.

1. Low flow Estimation

Return Period years	Duration	Low flow Discharge m^3/s
2	1 day	2.03
	7 days	2.16
	30 days	2.47
	Monthly	2.62
10	1 day	1.37
	7 days	1.45
	30 days	1.74
	Monthly	1.87
20	1 day	1.24
	7 days	1.31
	30 days	1.60
	Monthly	1.73

2. Instantaneous flood flow Estimation

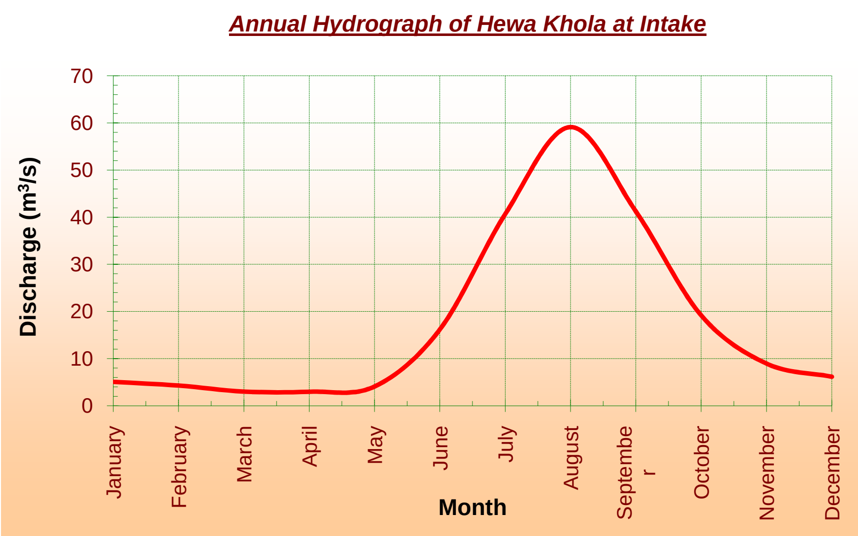
Return Period years	Daily flood discharge m^3/s	Instantaneous flood discharge m^3/s
2	130	222
5	190	376
10	232	496
20	273	622
50	329	804
100	372	953
200	417	1114
500	477	1347
1000	525	1538
5000	644	2038
10000	698	2280

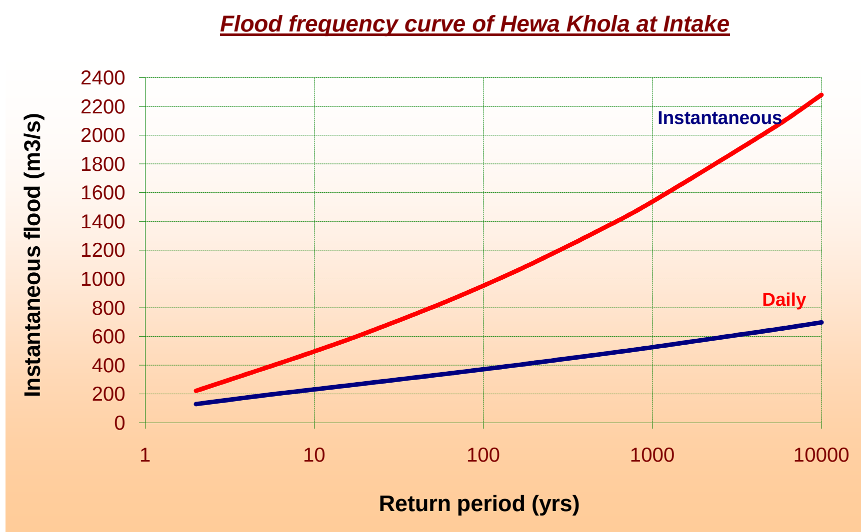
3. Mean monthly flow Estimation

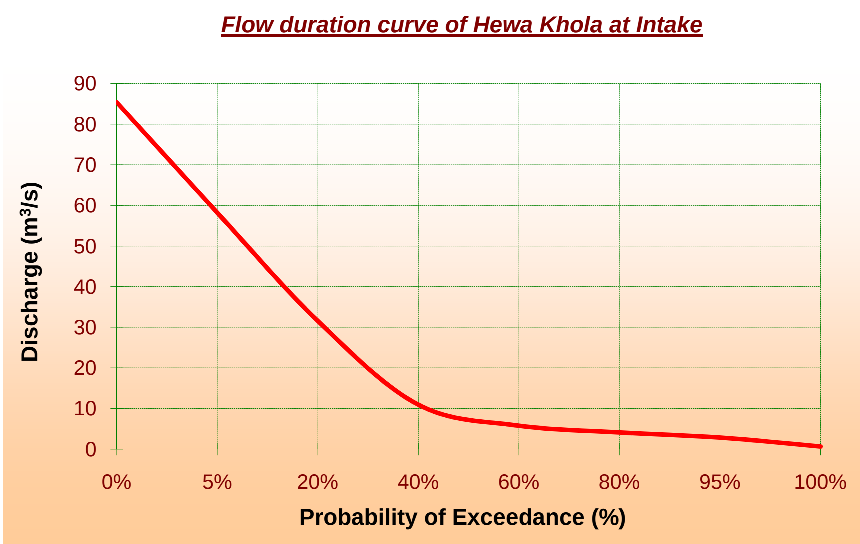
Month	Average discharge m^3/s
January	5.08
February	4.29
March	3.02
April	2.98
May	4.09
June	16.21
July	40.68
August	59.13
September	41.23
October	19.18
November	8.95
December	6.16
Annual	**17.58**

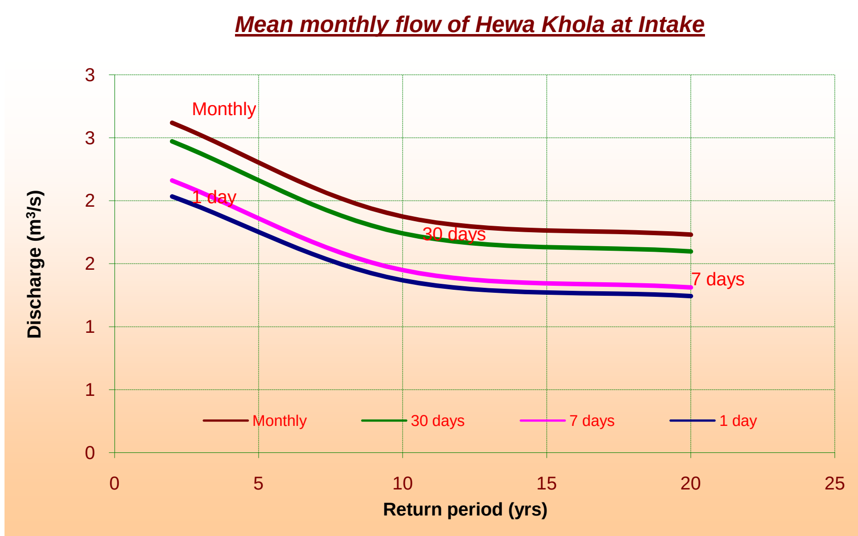
4. Flow duration Calculation

Probability of Exceedance %	Discharge m^3/s
0%	85
5%	58
20%	32
40%	11
60%	6
80%	4
95%	3
100%	1


Annual Hydrograph of Hewa Khola at Intake
70
60
50
40
30
20
10
0
Discharge (m³/s)
January
February
March
April
May
June
July
August
Septembe r
October
November
December
Month


Flood frequency curve of Hewa Khola at Intake
2400
2200
2000
1800
1600
1400
1200
1000
800
600
400
200
0
Instantaneous flood (m3/s)
Instantaneous
Daily
1
10
100
1000
10000
Return period (yrs)


Flow duration curve of Hewa Khola at Intake
90
80
70
60
50
40
30
20
10
0
Discharge (m³/s)
0%
5%
20%
40%
60%
80%
95%
100%
Probability of Exceedance (%)


Mean monthly flow of Hewa Khola at Intake
3
3
2
2
1
1
0
Discharge (m³/s)
Monthly
1 day
30 days
7 days
Monthly
30 days
7 days
1 day
0
5
10
15
20
25
Return period (yrs)

Equation used to determine Low flow;

$Q = [C_{d,T} + F_{d,T} \times sqrt(A_{<5k})]^2$

where;

$A_{<5k}$ = Basin area below 5000 m elevation level

$A_{<3k}$ = Basin area below 3000 m elevation level

$C_{d,T}$ = a constant

$F_{d,T}$ = Coefficient

Equation used for flood flow estimations;

Two year instantaneous flood; $Q_2 = 2.29(A_{<3k})^{0.86}$

Two year daily flood; $Q_2 = 0.8154(A_{<3k} + 1)^{0.9527}$

100 year instantaneous flood; $Q_{100} = 20.7(A_{<3k})^{0.72}$

100 year daily flood; $Q_{100} = 4.144(A_{<3k})^{0.8448}$

Instantaneous and daily flood at other return periods, $Q_f = \exp(\ln Q_2 + s\sigma)$

$\sigma = \ln(Q_{100}/Q_2)/2.326$

Equation used for mean monthly flow;

$Q_{JFJJASOND}$ = Exp(const + coeff. Of avg elv * $\ln A_{ELV}$ + coeff. Of Ann Ptn * Annual wetness index + coeff. of $A_{<3k}$ * Basin area below 3000m elevation)

Q_{ma} = [const + coeff. of $A_{<5k}$ * sqrt(basin area below 5000 m elevation)]^2

where;

J = January

F = February

A = August

S = September

O = October

N = November

D = December

m = March, May

a = April

A_{ELV} = Average basin Elevation

Flow duration Calculaiton

Available discharge for 0% of time; Q_0 [const + coeff. Of Ann Ptn * sqrt(Annual wetness index) + coeff. of $A_{<5k}$ * sqrt(basin area below 5000 m elevation)]^2

Available discharge for 10% to 80% of time; $Q_{10,-,80}$ = Exp(const + coeff. Of avg elv * $\ln A_{ELV}$ + coeff. Of Ann Ptn * Annual wetness index + coeff. of $A_{<3k}$ * Basin area below 3000m elevation)

Available discharge for 100% of time; Q_{100} = [const + coeff. Of Ann Ptn * sqrt(Annual wetness index) + coeff. of $A_{<5k}$ * sqrt(basin area below 5000 m elevation)]^2

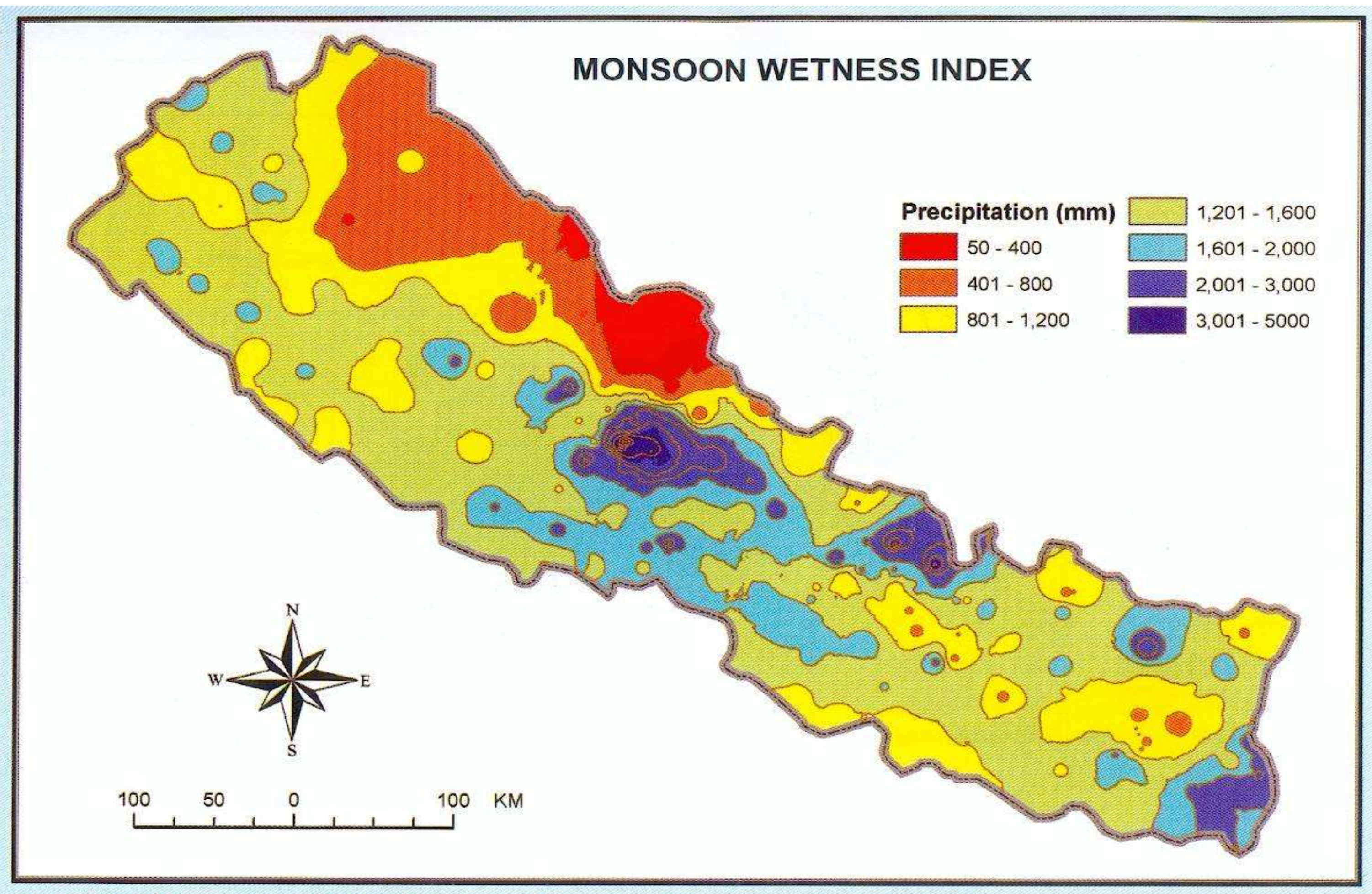


Figure 1.5 Monsoon Weightness Index in Nepal

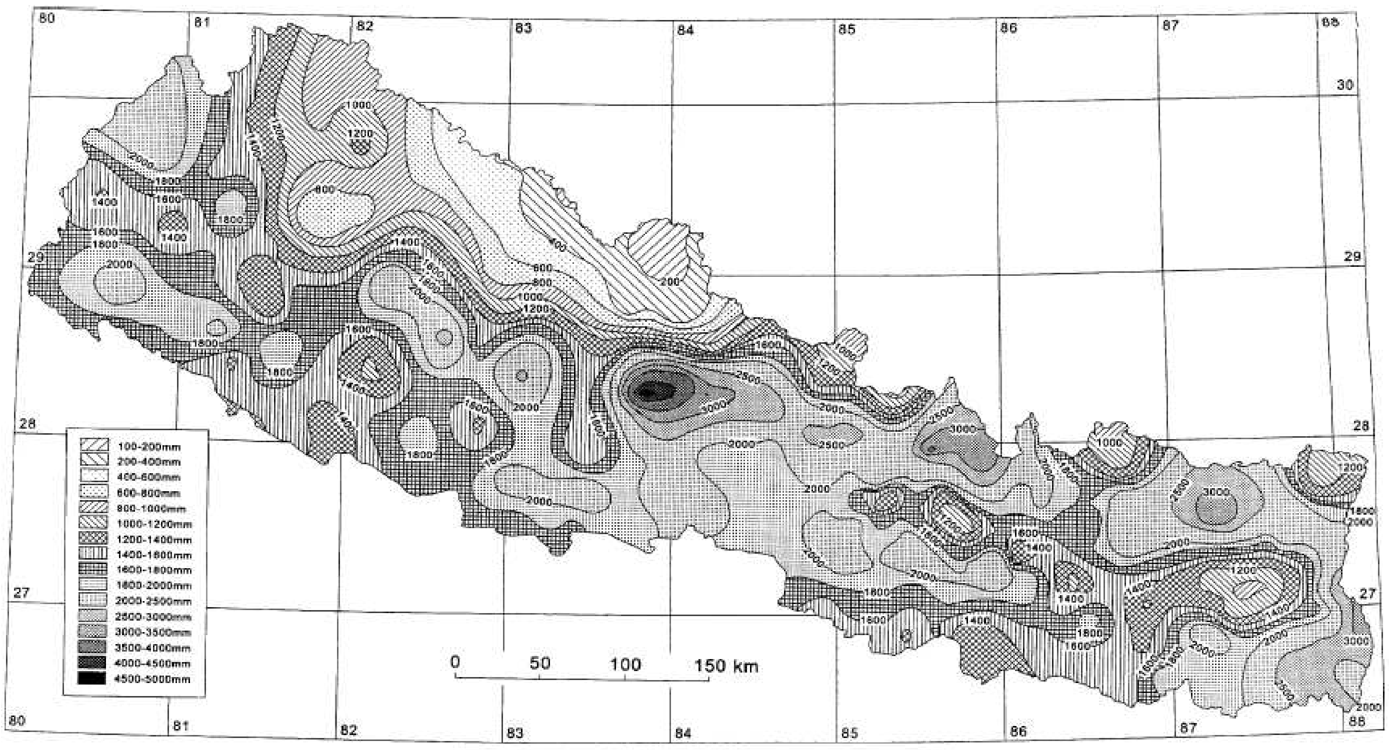


Fig. 5. Spatial distribution of long-term average annual total precipitation across Nepal (after Chalise et al. (1996); the isohyetal method was used to map areal rainfall).

Calculation of flood by using Gumbel's Distribution (Frequency Analysis)

Year	Q (m3/s)
1983	**11.81**
1984	**13.28**
1985	**11.05**
1986	**10.83**
1987	**16.47**
1989	**10.30**
1990	**15.89**
1991	**12.85**
1992	**6.41**
1993	**8.05**

$$Y_n = -\ln(\ln(\frac{T}{T-1}))$$

$$K_n = \frac{y_n - 0.507}{0.9971}$$

OUTPUT OF FREQUENCY ANALYSIS

Return Period	Discharge(Cumecs)
$\mathbf{X_2}$	**11.25196356**
$\mathbf{X_5}$	**14.81824219**
$\mathbf{X_{10}}$	**17.17942896**
$\mathbf{X_{20}}$	**19.444336**
$\mathbf{X_{50}}$	**22.37602584**
$\mathbf{X_{100}}$	**24.57291482**
$\mathbf{X_{1000}}$	**31.83212234**

Flow Data of station 728

Given discharge in cumecs

Location: Rajdwali

Latitude: 26 52 45 d-m-s

River: Mai khola

Longitude: 87 55 45 d-m-s

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year
1983	5.64	4.48	3.60	3.18	7.08	17.90	90.60	44.40	46.40	19.90	9.74	6.32	**21.61**
1984	5.35	4.17	3.43	4.54	7.20	27.70	80.70	43.20	74.80	19.80	12.20	8.19	**24.28**
1985	6.09	5.86	4.46	4.65	8.51	17.30	46.40	43.90	45.70	35.50	15.10	8.90	**20.20**
1986	5.89	4.24	3.71	6.02	9.00	16.20	48.80	37.00	65.40	23.70	10.80	6.72	**19.79**
1987	4.66	3.89	4.35	5.05	7.09	14.50	66.60	142.00	58.60	32.30	14.00	8.31	**30.12**
1988	5.91	4.92	5.63	5.07			29.60	34.00	61.30	16.90	6.83	6.55	
1989	6.05	5.21	5.10	3.88	9.65	31.50	42.50	34.50	48.50	21.90	9.85	7.28	**18.83**
1990	5.48	5.54	8.03	14.10	27.60	73.60	67.80	61.60	45.30	25.20	7.75	6.63	**29.06**
1991	8.18	6.58	6.35	6.89	8.29	31.40	70.20	61.60	55.80	13.00	8.13	5.46	**23.49**
1992	4.44	4.23	3.28	4.64	10.10	10.70	31.10	25.50	20.50	13.20	7.62	5.42	**11.73**
1993	5.13	4.00	3.42	7.12	7.96	16.00	30.70	41.80	25.80	16.80	10.60	7.25	**14.72**
1994							25.20	26.90	20.50	13.00	7.90	6.15	
1995	5.81	4.79	3.96	5.14	7.71			51.10		15.90	11.50	6.26	
Average	**5.72**	**4.83**	**4.61**	**5.86**	**10.02**	**25.68**	**52.52**	**49.81**	**47.39**	**20.55**	**10.16**	**6.88**	**21.39**

Discharge of Hewa Khola

Catchment area of station 728 = 404 Km^2

Catchment area of project = 221 Km^2

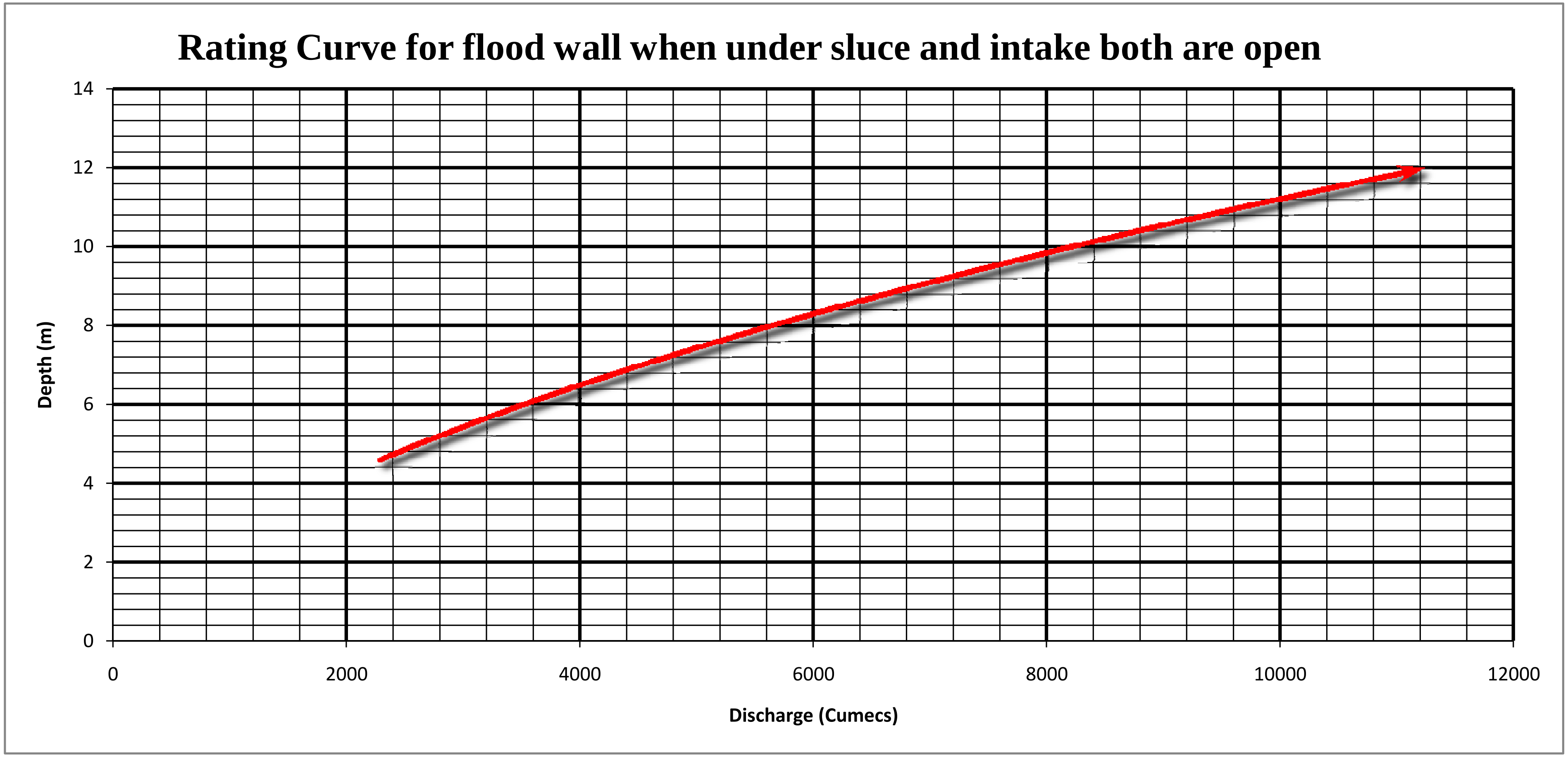
Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year
1983	3.08	2.45	1.96	1.73	3.87	9.79	49.56	24.28	25.38	10.88	5.32	3.45	**11.81**
1984	2.92	2.28	1.87	2.48	3.93	15.15	44.14	23.63	40.91	10.83	6.67	4.48	**13.28**
1985	3.33	3.20	2.43	2.54	4.65	9.46	25.38	24.01	24.99	19.41	8.26	4.86	**11.05**
1986	3.22	2.31	2.02	3.29	4.92	8.86	26.69	20.24	35.77	12.96	5.90	3.67	**10.83**
1987	2.54	2.12	2.37	2.76	3.87	7.93	36.43	77.67	32.05	17.66	7.65	4.54	**16.47**
1988	3.23	2.69	3.07	2.77	-	-	16.19	18.59	33.53	9.24	3.73	3.58	-
1989	3.30	2.85	2.78	2.12	5.27	17.23	23.24	18.87	26.53	11.97	5.38	3.98	**10.30**
1990	2.99	3.03	4.39	7.71	15.09	40.26	37.08	33.69	24.78	13.78	4.23	3.62	**15.89**
1991	4.47	3.59	3.47	3.76	4.53	17.17	38.40	33.69	30.52	7.11	4.44	2.98	**12.85**
1992	2.42	2.31	1.79	2.53	5.52	5.85	17.01	13.94	11.21	7.22	4.16	2.96	**6.41**
1993	2.80	2.18	1.87	3.89	4.35	8.75	16.79	22.86	14.11	9.19	5.79	3.96	**8.05**
1994	-	-	-	-	-	-	13.78	14.71	11.21	7.11	4.32	3.36	-
1995	3.17	2.62	2.16	2.81	4.21	-	-	27.95	-	8.69	6.29	3.42	-
Average	**3.13**	**2.64**	**2.52**	**3.20**	**5.48**	**14.05**	**28.73**	**27.25**	**25.92**	**11.24**	**5.55**	**3.76**	**11.70**

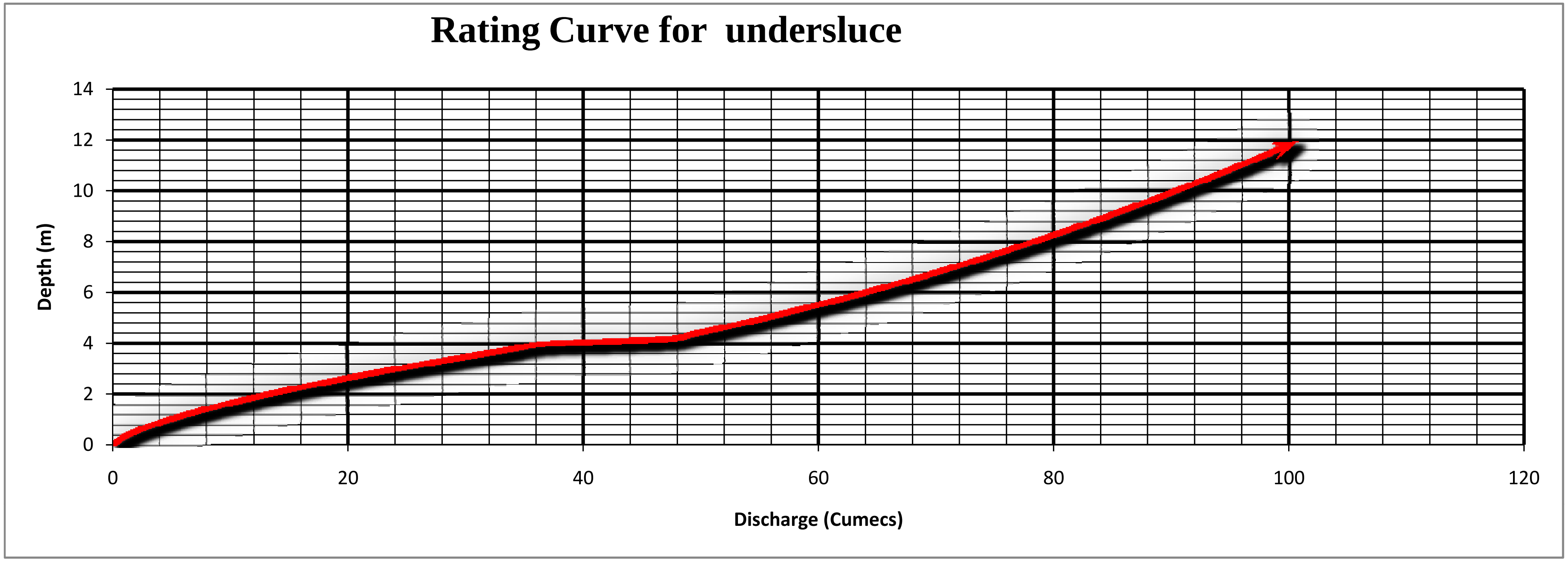
PREPARATION OF RATING CURVE

Slope
N　Manning's N　0.023

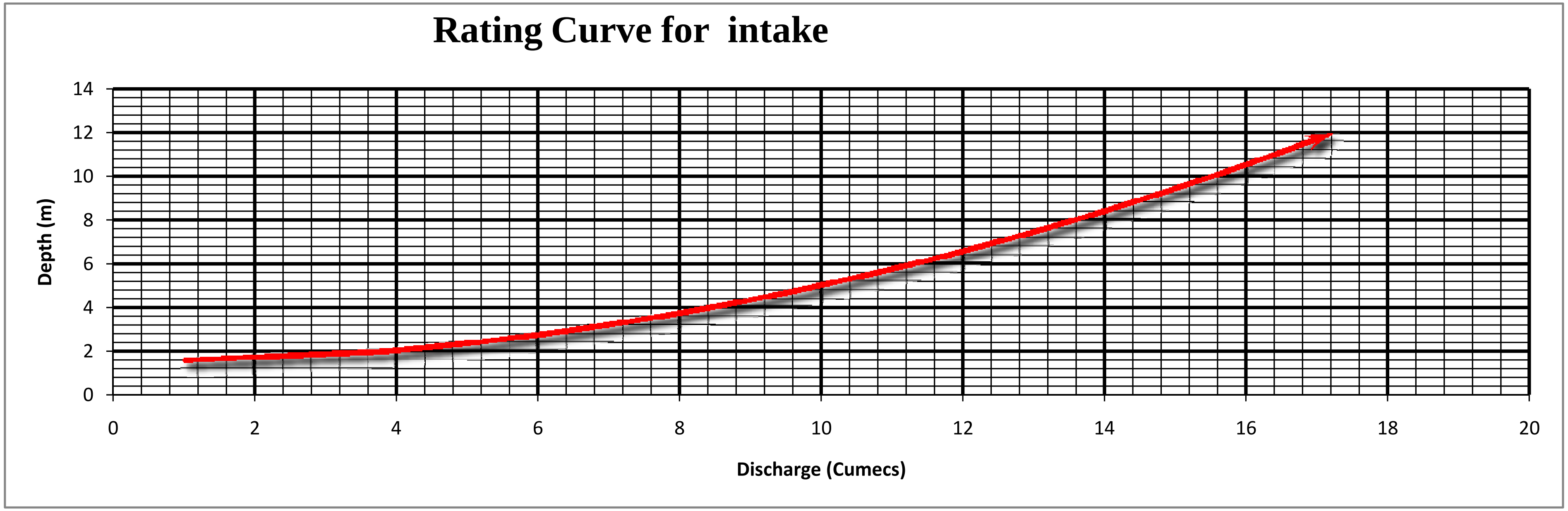
Depth (m) (Above bed level)	Area (m^2)	Wetted perimeter(m)	Hydraulic radius(m)	Velocity (m/s)	Discharge (m^3/s)				Remarks
					through undersluce (1.7*0.9*L*H^1.5)	through intake	when undersluce and intakes are closed	when undersluce and intakes are open	
0.1	2.0277	20.7	0.099	2.4	0.146				Less than 2 years flood
0.2	4.0727	21.2	0.193	3.74	0.411				Less than 2 years flood
0.3	6.135	21.7	0.283	4.83	0.755				Less than 2 years flood
0.4	8.2146	22.2	0.371	5.79	1.162				Less than 2 years flood
0.5	10.3116	22.7	0.455	6.64	1.623				Less than 2 years flood
0.6	12.4258	23.2	0.536	7.4	2.134				Less than 2 years flood
0.7	14.5574	23.7	0.615	8.11	2.689				Less than 2 years flood
0.8	16.7063	24.2	0.691	8.77	3.285				Less than 2 years flood
0.9	18.8725	24.7	0.765	9.39	3.92				Less than 2 years flood
1	21.0561	25.2	0.836	9.96	4.59				Less than 2 years flood
1.1	23.2569	25.7	0.906	10.51	5.296				Less than 2 years flood
1.2	25.4751	26.2	0.973	11.02	6.034				Less than 2 years flood
1.3	27.7106	26.7	1.039	11.51	6.804				Less than 2 years flood
1.4	29.9635	27.2	1.103	11.98	7.604				Less than 2 years flood
1.5	32.2336	27.7	1.165	12.42	8.433				Less than 2 years flood
1.6	34.5211	28.2	1.225	12.85	9.29	1.009			Less than 2 years flood
1.7	36.8259	28.7	1.284	13.26	10.174	1.664			Less than 2 years flood
1.8	39.148	29.2	1.342	13.65	11.085	2.33			Less than 2 years flood
1.9	41.4874	29.7	1.398	14.03	12.022	3.026			Less than 2 years flood
2	43.8442	30.2	1.453	14.4	12.983	3.759			Less than 2 years flood
2.1	46.2182	30.7	1.506	14.74	13.969	4.118			Less than 2 years flood
2.2	48.6096	31.2	1.559	15.09	14.978	4.448			Less than 2 years flood
2.3	51.0183	31.7	1.61	15.42	16.011	4.755			Less than 2 years flood
2.4	53.4444	32.2	1.661	15.74	17.066	5.043			Less than 2 years flood
2.5	55.8877	32.7	1.71	16.05	18.144	5.316			Less than 2 years flood
2.6	58.3484	33.2	1.759	16.35	19.243	5.575			Less than 2 years flood

2.7	60.8264	33.7	1.806	16.64	20.364	5.823			Less than 2 years flood
2.8	63.3217	34.2	1.853	16.93	21.506	6.061			Less than 2 years flood
2.9	65.8343	34.7	1.898	17.2	22.668	6.29			Less than 2 years flood
3	68.3643	35.2	1.943	17.48	23.851	6.51			Less than 2 years flood
3.2	73.4762	36.2	2.031	18	26.275	6.931			Less than 2 years flood
3.4	78.6573	37.2	2.116	18.5	28.777	7.327			Less than 2 years flood
3.6	83.9077	38.2	2.198	18.97	31.353	7.703			Less than 2 years flood
3.8	89.2275	39.2	2.277	19.42	34.001	8.062			Less than 2 years flood
4	94.6165	40.2	2.355	19.87	36.72	8.405			Less than 2 years flood
4.2	100.0747	41.2	2.43	20.29	47.304	8.734			Less than 2 years flood
4.4	105.6023	42.2	2.504	20.7	49.407	9.052			Less than 2 years flood
4.6	111.1991	43.2	2.575	21.09	51.425	9.359	2345.19	2284.406	Greater than 1000 years flood
4.8	116.8653	44.2	2.645	21.47	53.366	9.656	2509.1	2446.078	Greater than 1000 years flood
5	122.6007	45.2	2.714	21.84	55.239	9.945	2677.6	2612.416	Greater than 1000 years flood
6	152.317	50.2	3.035	23.53	63.785	11.276	3584.02	3508.959	Greater than 1000 years flood
7	183.7653	55.2	3.33	25.03	71.313	12.466	4599.65	4515.871	Greater than 1000 years flood
8	216.9457	60.2	3.605	26.39	78.12	13.552	5725.2	5633.528	Greater than 1000 years flood
9	251.8581	65.2	3.864	27.64	84.379	14.557	6961.36	6862.424	Greater than 1000 years flood
10	288.5026	70.2	4.111	28.8	90.205	15.497	8308.88	8203.178	Greater than 1000 years flood
11	326.8791	75.2	4.348	29.9	95.677	16.383	9773.69	9661.63	Greater than 1000 years flood
12	366.9877	80.2	4.577	30.94	100.852	17.224	11354.6	11236.524	Greater than 1000 years flood


Rating Curve for flood wall when under sluce and intake both are open
Depth (m)
0
2
4
6
8
10
12
14
0
2000
4000
6000
8000
10000
12000
Discharge (Cumecs)



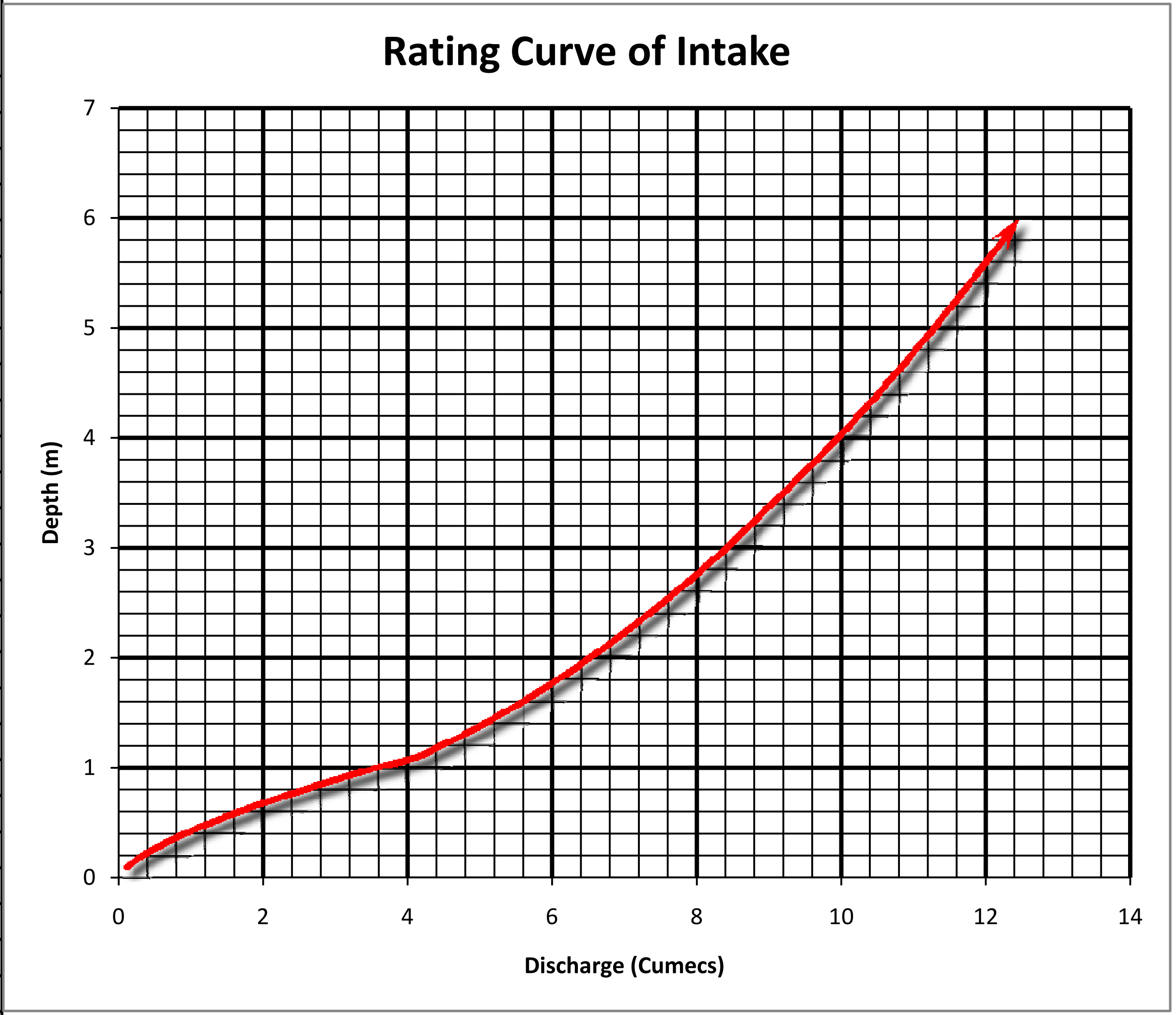
Rating Curve for undersluce
Depth (m)
0
2
4
6
8
10
12
14
0
20
40
60
80
100
120
Discharge (Cumecs)



Rating Curve for intake
Depth (m)
0
2
4
6
8
10
12
14
0
2
4
6
8
10
12
14
16
18
20
Discharge (Cumecs)

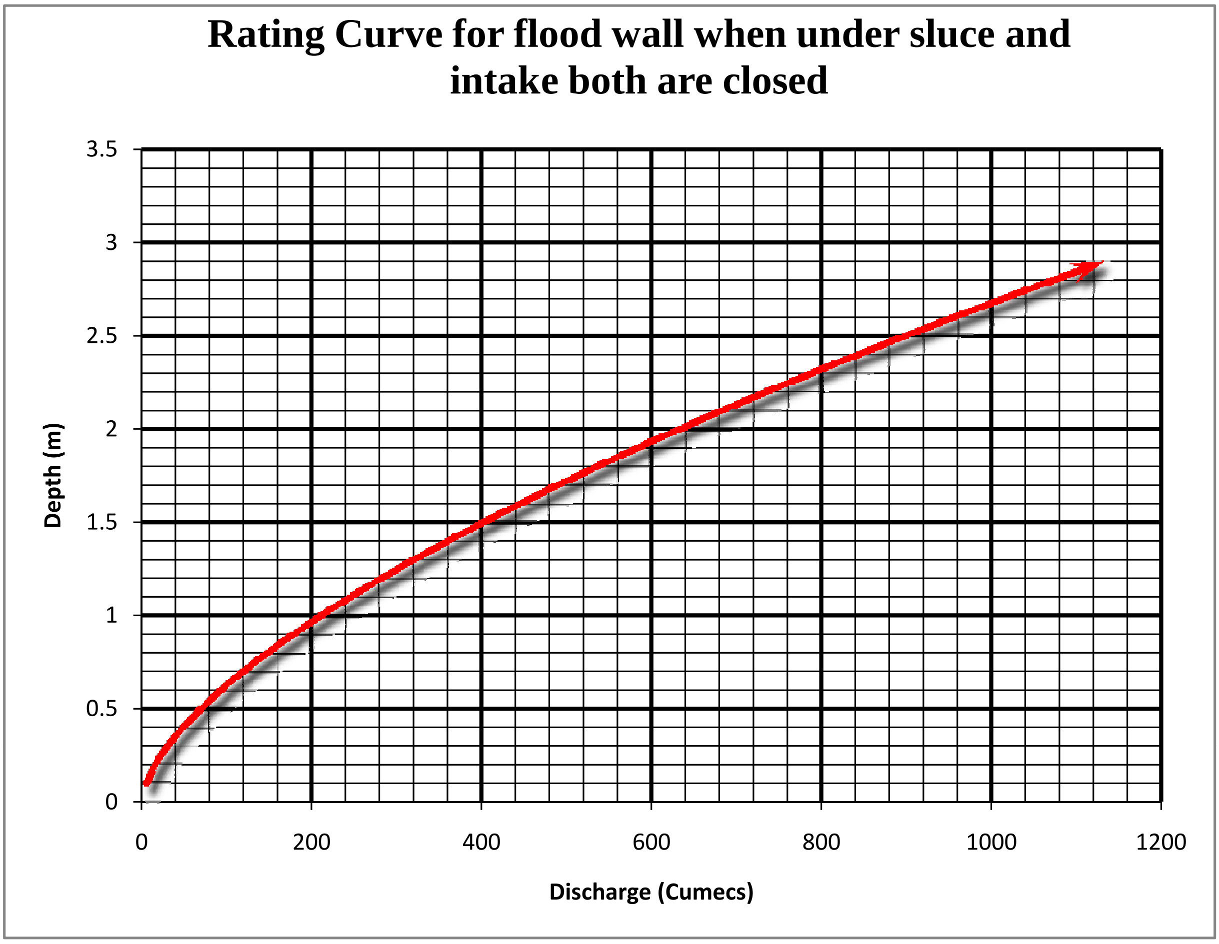
Rating Curve intake (Single Opening)

Depth (m)	h	Area (m^2)	Discharge(m^3)	Remarks
0.1		0.2	0.112	
0.2		0.4	0.316	
0.3		0.6	0.582	
0.4		0.8	0.896	
0.5		1	1.252	
0.6		1.2	1.646	
0.7		1.4	2.075	
0.8		1.6	2.535	
0.9		1.8	3.025	
1		2	3.543	
1.1	0.6	2	4.117	
1.2	0.7	2	4.447	
1.3	0.8	2	4.754	
1.4	0.9	2	5.042	
1.5	1	2	5.315	
1.6	1.1	2	5.574	
1.7	1.2	2	5.822	
1.8	1.3	2	6.06	
1.9	1.4	2	6.289	
2	1.5	2	6.509	
2.1	1.6	2	6.723	
2.2	1.7	2	6.93	
2.3	1.8	2	7.131	
2.4	1.9	2	7.326	
2.5	2	2	7.517	
2.6	2.1	2	7.702	
2.7	2.2	2	7.883	
2.8	2.3	2	8.061	
2.9	2.4	2	8.234	
3	2.5	2	8.404	
4	3.5	2	9.944	
5	4.5	2	11.275	
6	5.5	2	12.465	



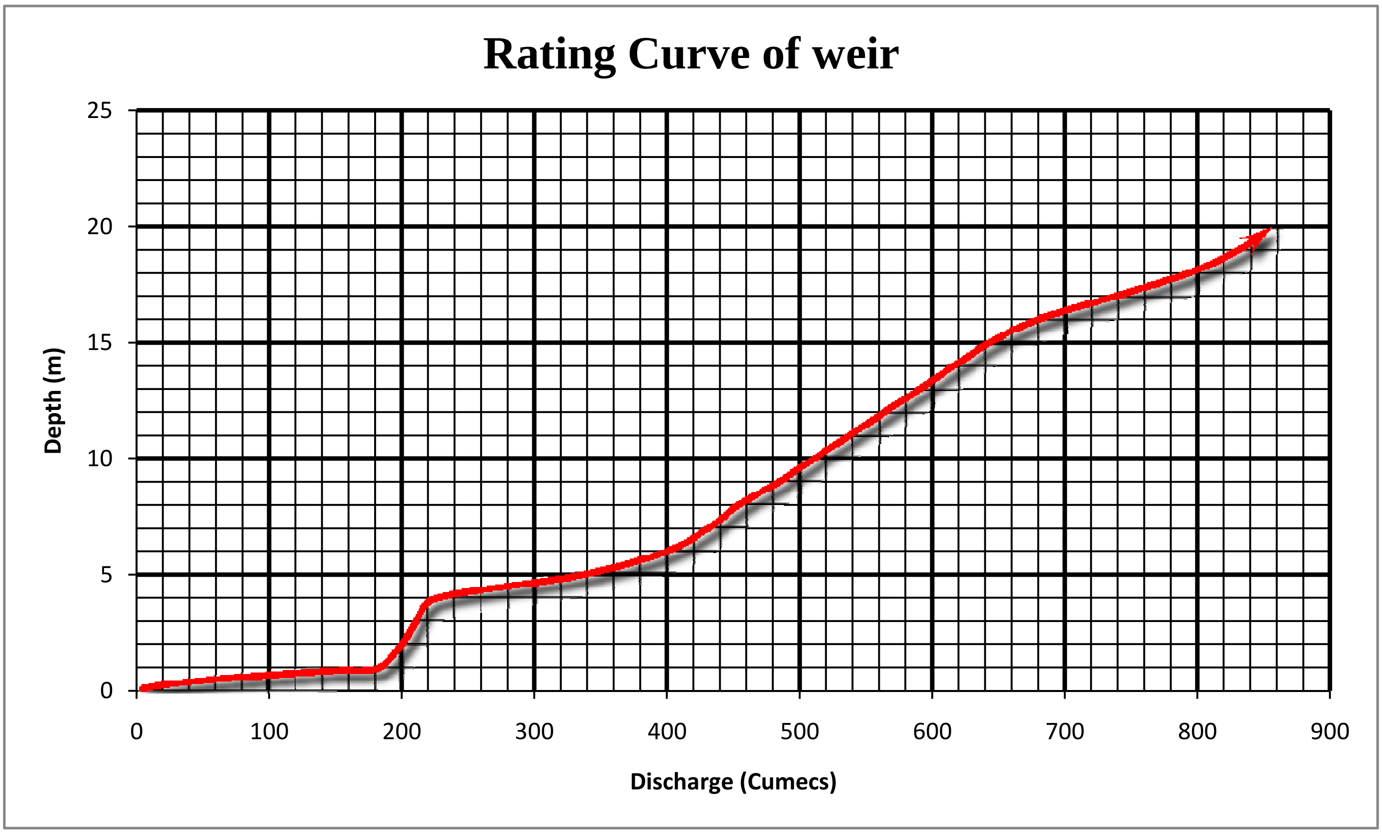
Rating Curve for Flood Wall

Depth (m) (Above weir)	Area (m^2)	Wetted perimeter (m)	Hydraulic radius(m)	Velocity (m/s)	Discharge (m^3)	Remarks
0.1	2.0277	20.7	0.099	2.4	4.87	Less than 2 years flood
0.2	4.0727	21.2	0.193	3.74	15.24	Less than 2 years flood
0.3	6.135	21.7	0.283	4.83	29.64	Less than 2 years flood
0.4	8.2146	22.2	0.371	5.79	47.57	Less than 2 years flood
0.5	10.3116	22.7	0.455	6.64	68.47	Less than 2 years flood
0.6	12.4258	23.2	0.536	7.4	91.96	Less than 2 years flood
0.7	14.5574	23.7	0.615	8.11	118.07	Less than 2 years flood
0.8	16.7063	24.2	0.691	8.77	146.52	Greater than 2 years flood
0.9	18.8725	24.7	0.765	9.39	177.22	Greater than 2 years flood
1	21.0561	25.2	0.836	9.96	209.72	Greater than 5 yeras flood
1.1	23.2569	25.7	0.906	10.51	244.44	Greater than 10 years flood
1.2	25.4751	26.2	0.973	11.02	280.74	Greater than 20 years flood
1.3	27.7106	26.7	1.039	11.51	318.95	Greater than 20 years flood
1.4	29.9635	27.2	1.103	11.98	358.97	Greater than 50 years flood
1.5	32.2336	27.7	1.165	12.42	400.35	Greater than 100 years flood
1.6	34.5211	28.2	1.225	12.85	443.6	Greater than 100 years flood
1.7	36.8259	28.7	1.284	13.26	488.32	Greater than 100 years flood
1.8	39.148	29.2	1.342	13.65	534.38	Greater than 1000 years flood
1.9	41.4874	29.7	1.398	14.03	582.07	Greater than 1000 years flood
2	43.8442	30.2	1.453	14.4	631.36	Greater than 1000 years flood
2.1	46.2182	30.7	1.506	14.74	681.26	Greater than 1000 years flood
2.2	48.6096	31.2	1.559	15.09	733.52	Greater than 1000 years flood
2.3	51.0183	31.7	1.61	15.42	786.71	Greater than 1000 years flood
2.4	53.4444	32.2	1.661	15.74	841.22	Greater than 1000 years flood
2.5	55.8877	32.7	1.71	16.05	897	Greater than 1000 years flood
2.6	58.3484	33.2	1.759	16.35	954	Greater than 1000 years flood
2.7	60.8264	33.7	1.806	16.64	1012.16	Greater than 1000 years flood
2.8	63.3217	34.2	1.853	16.93	1072.04	Greater than 1000 years flood
2.9	65.8343	34.7	1.898	17.2	1132.35	Greater than 1000 years flood


Rating Curve for flood wall when under sluce and intake both are closed
Depth (m)
3.5
3
2.5
2
1.5
1
0.5
0
0
200
400
600
800
1000
1200
Discharge (Cumecs)

Rating Curve at Weir

Depth (m)	Area (m^2)			Wetted perimeter (m)	Hydraulic radius(m)	Velocity (m/s)	Discharge(m^3)	Remarks
0.1	1.64	33.06	16.49	16.57	0.099	2.4	3.94	Less than 2 years flood
0.2	3.3	33.6	16.72	16.88	0.196	3.78	12.48	Less than 2 years flood
0.3	4.9	34.15	16.96	17.19	0.286	4.87	23.87	Less than 2 years flood
0.4	6.69	34.7	17.19	17.51	0.383	5.92	39.61	Less than 2 years flood
0.5	8.42	35.24	17.42	17.82	0.473	6.81	57.35	Less than 2 years flood
0.6	10.17	35.79	17.66	18.13	0.561	7.63	77.6	Less than 2 years flood
0.7	11.95	36.34	17.89	18.45	0.648	8.4	100.38	Less than 2 years flood
0.8	13.75	36.88	18.12	18.76	0.733	9.12	125.4	Less than 2 years flood
0.9	15.58	37.43	18.35	19.08	0.817	9.81	152.84	Greater than 2 years flood
1	17.42	37.98	18.59	19.39	0.899	10.45	182.04	Greater than 2 years flood
2	19.33	41.98	20.08	21.9	0.883	10.33	199.68	Greater than 5 yeras flood
3	20.91	46.35	21.75	24.6	0.85	10.07	210.57	Greater than 5 yeras flood
4	22.78	51.3	23.82	27.48	0.829	9.9	225.53	Greater than 5 yeras flood
5	34.13	79.46	37.68	41.78	0.817	9.81	334.82	Greater than 50 years flood
6	39.16	85.99	40.63	45.36	0.864	10.18	398.65	Greater than 100 years flood
7	42.26	92.2	43.34	48.86	0.865	10.19	430.63	Greater than 100 years flood
8	44.89	99.01	46.44	52.57	0.854	10.1	453.39	Greater than 100 years flood
9	47.86	105.3	49.27	56.03	0.855	10.11	483.87	Greater than 100 years flood
10	50.62	111.41	51.98	59.43	0.852	10.08	510.25	Greater than 100 years flood
11	53.35	117.56	54.72	62.84	0.849	10.06	536.71	Greater than 1000 years flood
12	56.07	123.65	57.42	66.23	0.847	10.04	562.95	Greater than 1000 years flood
13	58.79	129.8	60.15	69.65	0.845	10.03	589.67	Greater than 1000 years flood
14	61.51	135.9	62.87	73.03	0.843	10.01	615.72	Greater than 1000 years flood
15	64.32	142.34	65.76	76.58	0.84	9.99	642.56	Greater than 1000 years flood
16	68.37	153.13	70.98	82.15	0.833	9.93	678.92	Greater than 1000 years flood
17	73.87	165.06	76.77	88.29	0.837	9.97	736.49	Greater than 1000 years flood
18	78.79	173.63	80.81	92.82	0.849	10.06	792.63	Greater than 1000 years flood
19	82.24	180	83.68	96.32	0.854	10.1	830.63	Greater than 1000 years flood
20	84.83	185.45	85.99	99.46	0.853	10.09	855.94	Greater than 1000 years flood


Rating Curve of weir
25
20
15
10
5
0
Depth (m)
0
100
200
300
400
500
600
700
800
900
Discharge (Cumecs)

HYDRAULIC DESIGN

DESIGN OF WEIR

Description	**Output**	**Reference**
Design discharge(Q_{100}) = 372.00 m^3/sec Return periods = 100 years R.L of river bed level = 665.00 m R.L of crest level = 665.00 + 3.00 = 668.00 m $Q = C_d * L * H^{3/2}$ or, $372 = 1.6 * 16.25 * H^{3/2} \rightarrow$ H = 5.90 m (head over the crest) R.L of HFL = 665.00 + 5.90+3.00 = 673.90 m Length =16.25 m Discharge intensity, q = $\frac{Q}{L} = \frac{372}{16.25}$ = 22.89 m^3/sec Normal scour depth, R = $1.35\left(\frac{q^2}{f}\right)^{\frac{1}{3}} = 1.35\left(\frac{22.89^2}{1.25}\right)^{\frac{1}{3}}$ = 10.1 m $\approx$ 10m Regime velocity of flow, $V = \frac{q}{R} = \frac{22.89}{10}$ = 2.29 m/sec Velocity head $h_a = \frac{V^2}{2g} = \frac{22.89^2}{2\times 9.81}$ = 0.27 m Total energy level, U/S HFL = 673.90 m U/S TEL = U/S HFL + h_a = 673.90 + 0.27 = 674.17 m HFL at D/S of weir = crest level - head over the crest = 668.00 - 5.90 = 662.10m Width of weir = 4m (assume)		Irrigation Engineering and Hydraulic Structures By: S.K. Garg (Page No. 525,934)

CHECKING FOR THE STABILITY OF THE WEIR

S.N	Symbol	Description	Forces(KN/m)		Lever arm(m)	Moments(KN)	
			V	H		MO	MR
1.	**W**	**Wt. of weir**					
	W1	$\frac{1}{2}\times 15\times 3\times 24$	540		39		21060
	W2	$4\times 3.5\times 24$	288		32		9216
	W3	$\frac{1}{2}\times 30\times 3\times 24$	1080		20		21600
2.	**U**	**Uplift pressure**					
		$\frac{1}{2}\times 49\times 3\times 9.81$	-721.04		35.333	23556.38	
3.	**P**	**U/S water pressure**					
	P1	$5..9\times 15\times 9.81$	868.185		41.5		35946.89
	P2	$\frac{1}{2}\times 15\times 3\times 9.81$	220.73		44		9712.12
	P3	$\frac{1}{2}\times 9.81\times 8.9^2$		388.53	2.96	1150.05	
4.	**S**	**U/S Silt pressure**					
		$\frac{1}{2}\times 18 * 0.33 * 3^2$		26.76	1.167	28.73	
SUM			**2273.875**	**415.28**		**24733.16**	**97535.01**

1. **Safety against sliding**

   FOS = $\mu\times\frac{\sum V}{\sum H} = 0.65\times\frac{2273.875}{415.26}$ = 3.56> 1 **OK**

2. **Safety against overturning**

   FOS = $\frac{\sum MR}{\sum MO} = \frac{97535.01}{24733.16} = 3.94 > 1.5$ **OK**

3. **Check for tension**

   Distance of resultant from Toe

   X' = $\frac{\sum M}{\sum V} = \frac{72801.85}{2273.875} = 32.02$ m

   Eccentricity, e = $\frac{B}{2} -$ X'

   $= \frac{49}{2} - 32.01 = 7.51m$

   $\frac{B}{6} = \frac{49}{6} = 8.17 > e$ **OK**

4. Safety against Principle Stress

$$\sigma_{\max} = \frac{\sum V}{B}\left(1+\frac{6e}{B}\right)$$

$$= \frac{2273.875}{49}\times\left(1+\frac{6\times 7.51}{49}\right)$$

$= 89.08 KN/m^2 < 2500 KN/m^2$ **Safe OK**

$$\sigma_{\min} = \frac{\sum V}{B}\left(1-\frac{6e}{B}\right)$$

$$= \frac{2273.875}{49}\times\left(1-\frac{6\times 7.51}{49}\right)$$

$=3.73 KN/m^2$ Safe. OK

DESIGN OF INTAKE

Description	Output	Reference
$Q_d = 7.8m^3/s$ Increasing discharge by 30% $Q_{intake} = 1.3*7.8 = 10.14$ m^3/s Adopt velocity through orifice intake regulator, V= 2 m/s A= Q/V =10.14/2 = 5.07 m^2 Assume two openings; Area of each opening =5.07/2 =2.54 m^2 Assume; H= 1m, B= 2m Q= $C_d A\sqrt{2gH}$ =1*2* $\sqrt{2*9.81*1.5}$ = 6.51m^3/s Total discharge = 2*6.51= 13 m^3/s > 7.8m^3/s ok Losses in intakes : Trashrack losses : HL= $K_f*(t/b)^{4/3}*\sin\beta*(v^2/2*g)$ where: K_f = head loss factor (=2.42 assuming rectangular bars) t= thickness of bars (mm) (=50mm, assumed) b= clear bar spacing (mm) (=150mm, assumed) β=angle of inclination to horizontal (degree) (= 75°, assumed) v =approach velocity (m/s)= $0.5*Q_d^{0.2}$ m/s =.05*7.80.2 =0.75 m/s (for trash rack that are mannualy cleaned, v should not exceed 1m/s) Therefore; $H_L=2.42*(50/150)^{4/3}*\sin(75)*(0.75^2/2*9.81)$ = 0.0113m Form losses =$0.3*(v_2^2/2*g) = 0.3*(2^2/2*9.81)$= 0.061mTotal losses= 0.0113+0.0621= 0.073m ~ 0.1m Now crest level of diversion weir = sill height + height intake orifice + total losses =1+1+0.1 =2.1m (at normal operation level) Assuming FB as 0.9m ,total height of diversion weir = 2.1+ 0.9= 3m Now discharge passing through each orifice with considering total loss, H=1.5-0.1= 1.4m $Q_1= Q_2$= Cd*A* $\sqrt{2gH}$ = 0.6*2*1* $\sqrt{2*9.81*(1.4/2)}$ m^3/s = 4.45 m^3/s Therefore; Total discharge Q= 2*4.45= 8.9 m3/s > 7.8 m^3/s ok	B=5.25m H=1m Height of flood wall=5.5m	

DESIGN OF GRAVEL TRAP

Description	**Output**	**Reference**
1.Given Discharge (Q) = $7.8m^3/s$ Design discharge(Q_d) $= 1.3 \times 7.8 = 10.14m^3/s$ Partical size = 2mm 2.Settling velocity v = $\sqrt{\frac{4}{3} \times \frac{g \times d \times (S-1)}{C_d}}$ For Reynold's no.R = 1000 to 10000 $C_d = 0.65$ $S = 2.65$ $V_s = \sqrt{\frac{4}{3} \times \frac{9.81 \times 0.002 \times (2.65-1)}{0.65}}$ =0.257694 m/s Now, $R = \frac{V_s \times d}{\mu} = \frac{0.26 \times 0.002}{1.32 \times 10^{-6}} =$ 393.93≈400 where μ is the kinetic viscosity of water for R=400,that is between 0.1 and 1000 we have, $C_d = \frac{24}{R} + \frac{3}{\sqrt{R}} + 0.34$ =0.4075 After iteration we get $V_s = 0.287m/s$	 Settling velocity = 0.287m/s	
3. Transit velocity = a (d)$^{0.5}$ a=36 for 2mm partical $V_f = 36 \times \sqrt{2} = 50.91cm/s = 0.509m/s$ 4. X-section area (A) = $\frac{Q_d}{v_f}$ =19.92m2 assume height of gravel trap(H) = 3m weadth(B) = A/d = 6.64m≈6.7m 5. **Length of gravel trap** Assume settling time(T) = $\frac{H}{V_s}$ =10.45sec		

Length of gravel trap = $V_f \times T$ = 5.32m ≈ 5.5m Height = 3m, width = 6.7m, length = 5.5m 6. **Transition design** Outlet length = $\frac{6.7-2.5}{2\times tan15}$ =7.84≈8m Overall length = 8+5.5=13.5m 7. Flushing discharge = 10% of Q $Q_f = 0.1\times 7.8 = 0.78 m^3/s$ Assume 0.5m*0.5 m sized flushed canal and bed slope is 1 in 40 Velocity $= V = \frac{1}{n}\times R^{\frac{2}{3}}\times\sqrt{S}$ [$R = \frac{B\times H}{B+2H}$ = 0.167] V = 3.2m/s 8.Tractive shear stress $\tau_0 = \gamma_w \times R\times S$ = 40.95N/m^2 $\tau_c = 0.056\times\gamma_w\times d\times(S-1)$ =9.06N/m^2 =(critical shear stress) d=10cm 10. Opening for flushing canal Head over the orifice = 3+0.2=3.2m Q=0.65*a*(2gh)$^{0.5}$ a=b*h b=0.5m, h=0.38m **Design of Approach canal from Gravel trap to settling basin**: Manning's coefficient = 0.015 Q intake = 7.8*1.1= 8.58 m3/s (Q_d) Assume bed slope =1 in 750 assume width of canal (B) = 2.5m $Q_d = \frac{1}{n}\times A\times R^{\frac{2}{3}}\times S^{\frac{1}{2}} = \frac{1}{n}\times B\times H\times(\frac{B\times H}{B+2H})^{\frac{2}{3}}\times\sqrt{S}$ Hence by trial and error value of height (H) = 1.74m≈1.8m $V = \frac{1}{n}\times R^{\frac{2}{3}}\times\sqrt{S}$ ≈2m/s Hence, width(B) = 2.5m, Height(H) = 1.8m	Height = 3m Width = 6.7m Length = 5.5m	

DESIGN OF SETTLING BASIN

S.N	Description	Output	Reference
1.	Known parameters Discharge(D) = 7.8 m^3/sec Increasing 10% discharge for flushing Design discharge(Q_D) = $1.1\times7.8 = 8.58 m^3/s$ Limiting diameter of the silt D_{limit} = 0.2 mm Viscosity, $\nu = 1.31\times10^{-6}$ m^2/sec Specific gravity of particles = 2.65		
2.	Flow velocity, $V = a\times\sqrt{d_{mm}}$ $= 44\times\sqrt{0.2}$ = 19.68 cm/sec ≈ 0.2 m/sec	Flow velocity = 0.2 m/sec CWGL page 70	Camp formula
3.	Fall velocity $w = \frac{1}{18}\times\frac{g}{\nu}(\rho_s-\rho)\times d^2$ $= \frac{1}{18}\times\frac{9.81}{1.31\times10^{-6}}(2.65-1)\times(0.2\times10^{-3})^2$ = 0.0275 m/sec		Stoke's law
4.	Reynold's number (Re) = $\frac{w\times d}{\nu}$ $= \frac{0.0275\times10^{-3}\times0.2}{1.31\times10^{-6}}$ = 4.19 > 0.1 Using graph of dia. Vs velocity for tem.10º.C w = 0.0179 m/s	Not Ok	
5.	For 90% removal of particle $\eta = 1-e^{-wAs/Q}$ or, As = 1103.7 m^2 Taking, $\frac{L}{B} = 8$ $B = 11.75$ m $L = 93.97$ m		Vetter's Equation
6.	Provide two chambers As = 551.85 m^2 Width = 8.31 ≈ 8.50 m Length = 66.50 m Provide baffle wall of 0.2 m thickness		

<table>
<tr>
<td>7.

8.

9.</td>
<td>Total width of settling basin = $8.5 \times 2 + 0.2 = 17.2$ m

Now,
Settling basin depth:
$Q = B \times H \times V$
$H = \dfrac{8.58}{2 \times 8.5 \times 0.2}$ = 2.52 ≈ 2.6 m
Inlet profile:
Provide slope of 1 in 5
Length of the inlet transition
$= \dfrac{(17.2 - 2.5)}{2} \times 5$
$= 36.5m$
Outlet profile:
Provided slope of 1 in 2
Length of the outlet transition
$= \frac{(17.2 - 2.5}{2} \times 2$
=14.7m≈15m
Total length = 66.5+36.5+15 = 118 m

Sedimentation storage volume:
Let, C =2.3 kg/m^3
Sediment density, $S_{density}$ = 2600 kg/m^3
Flushing frequency = 24 hrs
Provide factor of safety, P_{factor}= 2
Sediment load = Q × t ×C
S_{load} =8.85×24×60×60×2.3
= 1758672 kg
$Volume\ of\ storage(Vs) = \dfrac{S_{load}}{S_{density}} \times P_{factor}$
$= \dfrac{1758672}{2600} \times 2$
= 1352.82 ≈ 1353 m^3
Storage depth:
$Y_{storage} = \dfrac{Vs}{A} = \dfrac{1353}{1103.7}$
= 1.225 m ≈1.3 m
Total depth of basin = Free board + H + $Y_{storage}$
= 0.5 + 2.6 + 1.3 =4.4 ≈ 5m
D = 5 m
L = 118 m</td>
<td>Total width = 17.2 m
Length = 66.5 m
Depth = 2.6 m</td>
<td>CWGL for micro hydropower page 72</td>
</tr>
</table>

10.	B = 8.5m Flushing canal: Assuming 15% extra flow for flushing Q_{scour} = 1.15×7.8 = 8.97 m^3/sec From design guidelines, for particle size of 0.2 mm Scour velocity V_{scour} = 2.4 m/sec Scour depth $Y_{scour} = \dfrac{8.97}{2.4\times8.5\times2}$ $= 0.2$ m $v = \dfrac{1}{n}\times R^{\frac{2}{3}}\times s^{\frac{1}{2}}$ $2.4 = \dfrac{1}{0.016}\times0.19^{\frac{2}{3}}\times(S)^{\frac{1}{2}}$ S = 1 in 80		Hill Irrigation Engineering

DESIGN OF FOREBAY

S.N	Description	Output	Reference
1.	Discharge Q = 7.8 m^3/sec Velocity V = 2.56 m/sec Submergence head, $h_s = \frac{1.5\times V^2}{2g} = \frac{1.5\times 2.56^2}{2\times 9.81}$ = 0.50 m According to design guidelines, the value should not be less than one times hence we assumed to increase by 2.75 times $h_s = 2.75 \times 0.50 = 1.38$ m total depth = submergence head + diameter of penstock + storage depth below penstock + Free board = 1.38 + 1.97 + 0.3 + 0.5 = 4.14 m ≈ 4.20 m		S.K. Garg
2.	Storage period T = 30 sec (minimum of 15 sec) Size of fore bay Volume V = Q × T V = 7.8 × 30 = 234 m^3 Area A = $\frac{V}{hs} = \frac{234}{1.38} = 169.57\ m^2$ Width = 15 m	Length =21m Width = 15 m Depth = 4.15 m	
3.	Length L = $\frac{169.57}{15} = 11.30 \approx 11.50$ m Design of transition length: Assume transition angle = 35° Transition length = $\frac{(15-2.04)}{2\times \tan 35}$ = 9.25 ≈ 9.5 m Total length of forebay = 11.50+9.5=21m		

SPILLWAY IN A FOREBAY

$Q_{spillway} = C_w \times L_{spillway} \times H_{overtop}^{\frac{3}{2}}$

$Q_{spillway}$ = 7.8 m^3/s

C_w = 1.6

$H_{overtop}$ = 1 m

Now,

$$7.8 = 1.6 \times L_{spillway} \times 1^{\frac{3}{2}}$$

$\therefore L_{spillway} = 4.88 \approx 5$ m hence adopted length is 5m

SELECTION OF TURBINE

S.N	Description	Output	Reference
1.	We have, Net head = 55.63 m Design discharge = 7.8m^3/sec No. of turbine = 2 Hence, Q= 7.8/2= 3.9m^3/sec So, Power = $9.81 \times Q \times H \times \eta$ = 9.81×3.9×55.63×0.9 = 1915.51 KW For the following criteria, Head Select Francis Turbine.	**Francis Turbine selection**	

DESIGN OF FRANCIS TURBINE:

S.N	Description	Output	Reference
	Design discharge Q = 3.9 m^3/sec Effective head H = 55.63 m		
1.	Power $P = 1915.51$ KW = 2554.01 HP		
2.	Specific speed $N_S = \frac{2400}{\sqrt{H}} = \frac{2400}{\sqrt{55.63}} = 321.78$ RPM		
3.	Synchronous speed $N = \frac{N_S \times H^{\frac{5}{4}}}{\sqrt{P}}$ $= \frac{321.78 \times 55.63^{\frac{5}{4}}}{\sqrt{2554.01}}$ $= 967.35$ RPM		
4.	Number of poles P = $\frac{120 \times f}{N} = \frac{120 \times 50}{967.35}$ = 6.2, adopt P=8		
5.	Corrected synchronous speed $N' = \frac{120 \times 50}{8}$ = 750 RPM		
6.	Corrected specific speed $N_S' = \frac{750 \times \sqrt{2554.01}}{55.63^{\frac{5}{4}}}$ = 249.48 RPM Calculation of diameter of Francis Turbine: Specific speed $N_S = \frac{750 \times \sqrt{2554.01}}{55.63^{\frac{5}{4}}} =$ 249.48RPM		

7.	$\phi = 0.0197 N_S^{\frac{2}{3}} + 0.0275$ $= 0.0197 \times 249.48^{\frac{2}{3}} + 0.0275$ $= 0.8$ Diameter D = $\dfrac{84.6 \times \phi \times \sqrt{H}}{N}$ $= \dfrac{84.6 \times 0.808 \times \sqrt{55.63}}{750}$ = 0.68 m Setting of turbine: $H_S = H_A - H_V - \sigma \times H$ H_S = Turbine Setting in m H_A = Atmospheric Pressure in m (=10.3m assumed) H_V = Vapour Pressure in m (=0.2m assumed) σ = Cavitation Coefficient H = Net Effective Head in m $\sigma = 0.0318 \times \left(\dfrac{N_S}{100}\right)^2 = 0.0318 \times \left(\dfrac{249.48}{100}\right)^2$ = 0.198 H_s = 10.3-0.2-0.198X55.63 = 0.91 Hence, The turbine should be set 0.91 m below the water level at tailrace.	Diameter =0.68 m	

DIMENSIONING OF POWERHOUSE

S.N	Description	Output	Reference
1.	Machine hall c/c spacing between two turbine = $5D + 2.5$ = 5 × 0.68 + 2.5 = 5.90 ≈ 6.00 m Total length = no. unit*(5D+2.5) + (6/2) + (6/2) = 2*6.00 +3.00 +3.00 = 18.00m Width of hall B = $5D + 2.5$ = 5X0.68+2.5= 6.00 m Height of superstructure = 4.5 (D + 1) = 4.5 (0.68 + 1) = 7.56 ≈ 7.6 m Height of intermediate structure = (2 to 2.5) m= 2.5m (approximately) Height of substructure = 1.5 m Total height = 7.5 + 2.5 + 1.5 = 11.5 m	Length=18.00 m Width =6.00 m Height = 11.5 m	M.M. Dandekar and K.N. Sharma
2.	Size of loading bay = 9m ×6 m= 54.00m		

DESIGN OF TAILRACE CANAL:

S.N	Description	Output	Reference
	Design discharge Q = 7.8m^3/sec Assume; slope 1 in 500 n = 0.015 widh (B) = 2.5m $Q_d = \frac{1}{n} \times A \times R^{\frac{2}{3}} \times S^{\frac{1}{2}} = \frac{1}{n} \times B \times H \times (\frac{B \times H}{B + 2H})^{\frac{2}{3}} \times \sqrt{S}$ Depth(H) = 1.385 m= 1.5 m Calculation of Canal Slope $V = \frac{1}{n} \times R^{\frac{2}{3}} \times S^{\frac{1}{2}}$ $= \frac{1}{0.016} \times 0.68^{\frac{2}{3}} \times (1/500)^{\frac{1}{2}}$ = 2.31 m/s	Depth = 1.5 m Width = 2.5 m Slope = 1 in 500	Civil works for guidelines in micro-hydropower in Nepal

DESIGN OF ANCHOR BLOCK

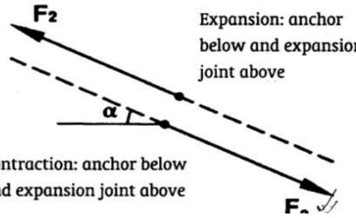
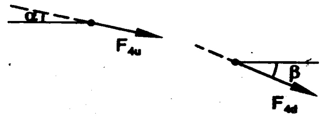
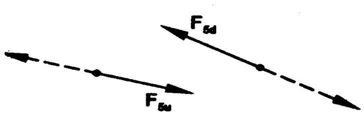
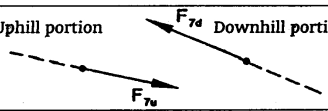
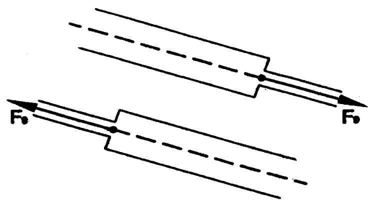
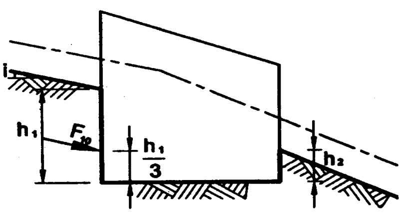
SN	Calculations	Output	Reference
	Design parameters: Total head (h_{total}) = h_{surge} + h_{gross} =(97.35 + 65.45) m Specific weight of soil (γ_{soil}) = 20 KN/m^2 Specific weight of concrete (γ_{conc}) = 22KN/m^2 Specific weight of steel (γ_{steel}) = 77 KN/m^2 Consider the block shown in figure (?) Weight of block(W_b) =89.51 × 22 = 1969.22 KN Weight of pipe (W_p): $= 3.14 \times (d+t) \times t \times \gamma_{steel}$ = 3.14 ×(1.97+ 0.008) × 0.008 × 77 = 3.83 KN/m Weight of water (W_w): $= \Pi \times \frac{d^2}{4} \times \gamma_{water}$ $= 3.14 \times \frac{1.97^2}{4} \times 9.81$ = 29.9 KN/m $W_p + W_w$ =33.73 KN/m Calculation of the relevant forces $F_{1u} = (W_p + W_w) \times l_{1u} \times \cos\alpha$ $= 33.73 \times 6 \times \cos 18$ $= 160.4 KN$ $F_{1d} = (W_p + W_w) \times l_{1d} \times \cos\beta$ $= 25 \times 6 \times \cos 25$ $= 183.42$ KN Frictional force per support pier: $= f \times (Wp + Ww) \times l2u \times \cos\alpha$ $= 0.25 \times 33.73 \times 7 \times \cos 25$ $= 56.14$ KN Since there are five support piers between two anchor Blocks so,	F_1 is the component of weight of pipe and water perpendicular to the pipe. Applies to both support piers and anchor block.	Civil Works Guidelines

	F_{2u} = 118.19 × 5 = 280.7 KN F_{2d} = 0 , since expansion joint is located immediately d/s of the anchor block. $F_3 = 15.4 \times h_{total} \times d^2 \times \sin\frac{(\beta-\alpha)}{2}$ $= 15.4 \times 162.8 \times 1.97 \times \sin\frac{(25\text{-}18)}{2}$ $= 593.99\ \text{KN}$ F_{4u} = W_p ×5 l_{4u} × sinα = 3.83 × 5×30× sin 19 = 177.53 KN F_{4d} = negligible F_6 = 100 × d = 100 × 1.97 = 197 KN F_{7u} = 31 × h_{total} × (d+t) × t = 31 × 162.8 × (1.97+0.008) × 0.008 = 79.86 KN F_{7d} = 79.89 KN $F_8 = 2.5 \times \frac{Q^2}{d^2} \times \sin\frac{(\beta-\alpha)}{2}$ =2.39≈ negligible F_9 = 0 , since pipe Φ does not change. $F_{10} = \gamma_{soil} \times h^2 \cos i \times K_a \times \frac{(W_p + W_W)}{2}$ $= \frac{20 \times 1.8^2}{2} \times \cos 25 \times 0.387 \times 2$ = 22.73 KN	F_2 is the frictional force due to the pipe sliding on the supports piers. F_3 is the hydrostatic force on bends that acts along the bisector of the bend. F_4 is the component of pipe weight acting parallel to pipe. F_6 is the frictional force in the expansion joint. The F_6 is felt because joint will resist sliding. F_7 is the hydrostatic force on exposed ends of pipe within expansion joint. F_8 is the dynamic force at a bend due to change in direction of moving water. Velocities are usually low in penstocks so this force is small. F_9 is the force due to reduction in pipe diamenter from d_{big} to d_{small}. F_{10} is the force due to soil pressure u/s of block.	

Resolution of Forces:

Forces (KN)	$X - Component\,(KN) \rightarrow +$	$y - component(KN) \downarrow +$
F_{1u}	$= -F_{1u}\sin\alpha$ $= -49.56$	$= +F_{1u}\times\cos\alpha$ $= +152.55$
F_{1d}	$= -F_{1d}\times\sin\beta$ $= -77.52$	$= F_{1d}\times\cos\beta$ $= 166.24$
F_{2u}	$= \pm F_{2u}\times\cos\alpha$ $= \pm 266.96$	$= \pm F_{2u}\times\sin\alpha$ $= \pm 86.74$
F_3	$= F_3\sin\frac{\beta+\alpha}{2}$ $= 217.7$	$= -F_3\cos\frac{\beta+\alpha}{2}$ =552.66 $= -552.66$
F_{4u}	$= F_{4u}\cos\alpha$ $= 168.84$	$= F_{4u}\sin\alpha$ $= 54.86$
F_6	$= \pm F_6(\cos\alpha - \cos\beta)$ $= \pm 8.82$	$= \pm F_6(\sin\beta - \sin\alpha)$ $= \pm 22.38$
F_{7u}	$= F_{7u}\cos\alpha$ $= 75.97$	$= F_{7u}\sin\alpha$ $= 24.68$
F_{7d}	$= -F_{7d}\cos\beta$ $= -72.4$	$= -F_{7d}\sin\beta$ $= -33.76$
F_{10} W_b	$= F_{10}\cos i$ $= 20.6$ $= 0.0$	$= F_{10}\sin i$ $= 9.6$ $= 1969.22$

Table 7.2 Forces on anchor and slide blocks

FORCE (kN)	DIRECTION OF POTENTIAL MOVEMENT OF ANCHOR BLOCK OR SUPPORT PIER	COMMENTS SYMBOLS ARE DEFINED AT THE END OF THIS TABLE
F_1 F_1 = combination of F_{1u} and F_{1d} $F_{1u} = (W_p + W_w)\, L_{1u} \cos \alpha$ $F_{1d} = (W_p + W_w) L_{1d} \cos \beta$ If pipe is straight, $F_1 = (W_p + W_w)(L_{1u} + L_{1d}) \cos \alpha$	Uphill portion Downhill portion 	F_1 is the component of weight of pipe and water perpendicular to the pipe. Applies to both support piers and anchor blocks
F_2 $F_{2u} = f\,(W_p + W_w) L_{2u} \cos \alpha$ $F_{2d} = f\,(W_p + W_w) L_{2d} \cos \alpha$	Expansion: anchor below and expansion joint above Contraction: anchor below and expansion joint above	F_2 is the frictional force due to the pipe sliding on the support piers. Applies to support piers and anchor blocks. The force acting at an anchor block is the sum of forces acting on the support blocks between the anchor block and expansion joints, but
F_4 F_4 = combination of F_{4u} and F_{4d} $F_{4u} = W_p\, L_{4u} \sin \alpha$ $F_{4d} = W_p\, L_{4d} \sin \beta$	Uphill portion Downhill portion 	F_4 is the component of pipe weight acting parallel to pipe. Applies to anchor blocks only. Calculate only if the angles (α or β) are larger than 20°.
F_5 $F_5 = 1000\, E a \;\; T \Pi\, (d + t)\, t$ See Table 6.2 in Chapter 6 for values of E and a	Uphill portion Downhill portion 	F_5 is the thermally induced force restrained by the anchor block in the absence of an expansion joint. Applies to anchor blocks only. Calculate only if expansion joints are not installed between anchor blocks.
F_6 $F_6 = 100\, d$	F_6 directions as F_5	F_6 is the frictional force in the expansion joint. The F_6 force is felt because the joint will resist sliding. Applies to anchor blocks only.
F_7 $F_7 = \gamma_{water}\, h_{total}\, \Pi\, (d + t)\, t$ $= 31\, h_{total}\, (d + t)\, t$ Usually insignificant	Uphill portion Downhill portion 	F_7 is the hydrostatic force on exposed ends of pipe within expansion joint Applies to anchor blocks only.
F_8 $F_8 = \left(\frac{2Q^2}{\frac{1}{4}\Pi d^2}\right) \sin\left(\frac{\beta - \alpha}{2}\right)$ $= 2.5 \left(\frac{Q^2}{d^2}\right) \sin\left(\frac{\beta - \alpha}{2}\right)$ Usually insignificant	F_8 directions as F_3	F_8 is the dynamic force at a bend due to change in direction of moving water. Velocities are usually low in penstocks so this force is small. Applies to anchor blocks only.
F_9 $F_9 = \gamma_{water} h_{total} \times \frac{\Pi}{4} (d_{big}{}^2 - d_{small}{}^2)$ $= 7.7\, h_{total}\, (d_{big}{}^2 - d_{small}{}^2)$		F_9 is the force due to reduction in pipe diameter from d_{big} to d_{small}. Applies to anchor blocks only.
F_{10} $F_{10} = \frac{\gamma_{soil} h_1{}^2}{2} \cos i \times K_a \times w$		F_{10} is the force due to soil pressure upstream of the block. Applies to both anchor blocks and support piers Calculate F_{10} if $(h_1 - h_2)$ is more than 1 m. The force acts at 1/3 of the height (h_1) from the base of the block.

COST ESTIMATION

COST ESTIMATION

Quantity Estimation of Civil Works

S.N.	Item Description	Unit	Rate	Quantity	Amount(US$)	
			(US$)		US$	NRs
1	General items					
	Contractual requirements					
	Insurances	LS			10,000	750,000
	Mobilization	LS			15,000	1,125,000
	Demobilization	LS			8,000	600,000
	Sub total				**33,000**	**2,475,000**
2	River diversion structure (1.5% of total cost				**41,858**	**3,139,356**
3	Headworks					
	Weir					
	Earth excavation	m^3	3.875	1,248.00	4,836	362,700
	Earth excavation on boulder mixed soil	m^3	4.425	1,170.00	5,177	388,294
	Back Filling	m^3	2.675	212.2	568	42,573
	Clay Blanket	m^3	20.85	825	17,201	1,290,094
	Filter	m^3	64.275	1,200.00	77,130	5,784,750
	Concrete work					
	C25	m^3	206.15	756	155,849	11,688,705
	C35	m^3	245.975	100.8	24,794	1,859,571
	Reinforcement bars	tons	2350.55	26	61,114	4,583,573
	1.5 m Boulder lining	m^2	202.5	1,540.00	311,850	23,388,750
	1 m dia Boulder (U/S of weir)	m^2	168.75	907.5	153,141	11,485,547
	Form work (upto 4 m high)	m^2	7.375	364	2,685	201,338
	Expansion/Construction Joints					
	Sealant	m	16.5	165.6	2,732	204,930
	Water bars	m	41.5	165.6	6,872	515,430
	Bank protection	LS			7,000	525,000
	Grouting	m^2	54.55	120	6,546	490,950
	Sub total				**837,496**	**62,812,203**
	Intake					
	Earth excavation	m^3	3.875	50.4	3,780	283,500
	Earth excavation on boulder mixed soil	m^3	4.425	75.6	5,670	425,250
	Backfilling	m^3	2.675	47	3,525	264,375
	Concrete work					
	C25	m^3	206.15	42.3	3,173	237,938
	Reinforcement bars	tons	2350.55	11.3	26,561	1,992,091
	Form work (upto 4 m high)	m^2	7.375	139.4	1,028	77,106
	Sub total				**43,737**	**3,280,259**
	Gravel trap					
	Earth excavation	m^3	3.875	124	481	36,038
	Earth excavation on boulder mixed soil	m^3	4.425	96	425	31,860
	Back Filling	m^3	2.675	52	139	10,433
	Concrete work					
	C25	m^3	206.15	18.5	3,814	286,033

COST ESTIMATION

Quantity Estimation of Civil Works

S.N.	Item Description	Unit	Rate	Quantity	Amount(US$)	
			(US$)		US$	NRs
	C35	m^3	245.975	13.6	3,345	250,895
	Reinforcement bars	tons	2350.55	1	2,351	176,291
	Form work (upto 4 m high)	m^2	7.375	102	752	56,419
	Hard Stone Lining	m^3	253.125	19.1	4,835	362,602
	Expansion/Construction Joints					
	Sealant	m	16.5	12.8	211	15,840
	Water bars	m	41.5	12.8	212	15,900
	Sub total				**16,564**	**1,242,309**
	Approach Cannel to Settling basin					
	Earth excavation	m^3	3.875	187.2	725.4	54,405
	Earth excavation on boulder mixed soil	m^2	4.425	124.8	552.24	41,418
	Back Filling	m^2	2.675	60	160.5	12,038
	Concrete work					
	C25	m^3	206.15	19.8	4081.77	306,133
	C35	m^3	245.975	15	3689.625	276,722
	Reinforcement bars	tons	2350.55	0.87	2044.9785	153,373
	Form work (upto 4 m high)	m^2	7.375	132	973.5	73,013
	Expansion/Construction Joints					
	Sealant	m	16.5	20.6	339.9	25,493
	Water bars	m	41.5	20.6	854.9	64,118
	Sub total				**13,423**	**1,006,711**
	Settling Basin					
	Earth excavation	m^3	3.875	3,648.00	14136	1,060,200
	Earth excavation boulder mixed soil	m^3	4.425	2,432.00	10761.6	807,120
	Back Filling	m^3	2.675	1,641.60	4391.28	329,346
	Concrete work					
	C35	m^3	245.975	602.4	148175.34	11,113,151
	Reinforcement bars	tons	2350.55	23.6	55472.98	4,160,474
	Form work (upto 4 m high)	m^3	7.375	2,509.80	18509.775	1,388,233
	Stone masonry	m^3	136.025	170.7	23,219	1,741,460
	12.5 mm thick 1:4 cement sand plastering	m^2	5.45	283.3	1,544	115,799
	1:1 Cement sand punning	m^2	8.725	283.3	2,472	185,384
	Expansion/Construction Joints					
	Sealant	m	16.5	299	4,934	370,013
	Water bars	m	41.5	299	12,409	930,638
	Side wall batten support					
	C35 (Structural concrete)	m^3	245.975	128.2	31,534	2,365,050
	Form work (4 to 5 m high)	m^2	7.375	328.4	2,422	181,646
	Steel reinforcement work	tons	2350.55	3.5	8,227	617,019
	Screeding (100 mm)	m^2	15.05	500	7,525	564,375
	500 mm dia. & 6mm thick perforated pipe	rm	52.85	250	13,213	990,938
	25 mm dia. pebbles drainage filter	m^3	64.25	300	19,275	1,445,625
	Side drain	LS			8,000	600,000

COST ESTIMATION

Quantity Estimation of Civil Works

S.N.	Item Description	Unit	Rate	Quantity	Amount(US$)	
			(US$)		US$	NRs
	Sub total				**386,220**	**28,966,469**
	River protection					
	Gabion wall protection	m^3	140.325	600	84,195	6,314,625
	Sub total				**84,195**	**6,314,625**
4	Waterways					
	Pressure pipe					
	Penstock length	m	5,500			
	Earth excavation	m^3	3.875	10,136.30	39,278	2,945,862
	Earth excavationin boulder mixed soil	m^3	4.425	8,784.80	38,873	2,915,456
	Back Filling with selected fill	m^3	2.675	5,432.30	14,531	1,089,855
	Back Filling with regular fill	m^3	5.175	1,696.80	8,781	658,571
	Dry stone packing	m^3	168.75	2,100.80	354,510	26,588,250
	Side drain	LS			13,000	975,000
	Sub total				**468,973**	**35,172,993**
	Anchor blocks					
	Earth excavation	m^3	3.875	44.8	174	13,020
	C15 plum concrete	m^3	156.625	194.3	30,432	2,282,418
	C25 concrete	m^3	206.15	87.7	18,079	1,355,952
	Reinforcement bars	tons	2350.55	1.7	3,996	299,695
	Form work	m^2	7.375	76.8	566	42,480
	Sub total				**53,248**	**3,993,565**
	Support piers					
	Earth excavation	m^3	3.875	972	3,767	282,488
	C15 concrete	m^3	4.425	48.6	215	16,129
	C25 concrete	m^3	156.625	97.2	15,224	1,141,796
	Reinforcement bars	tons	2350.55	1.4	3,291	246,808
	Form work	m^2	7.375	259.2	1,912	143,370
	Stone masonry work in 1c/s mortar		123.55	972	120,091	9,006,795
	Sub total				**144,498**	**10,837,386**
	Earth excavation	m^3	3.875	2,609.30	10,111	758,328
	Earthwork excavation in boulder mixed soil	m^3	4.425	1,739.60	7,698	577,330
	Back Filling	m^3	2.675	111.4	298	22,350
	C25 Concrete work	m^3	206.15	371.4	76,564	5,742,308
	Form work upto 4 m high	m^2	7.375	282	2,080	155,981
	Reinforcement bars	tons	2350.55	3.8	8,932	669,907
	Gabion wall protection downstream face	m^3	140.325	720	101,034	7,577,550
	Stone riprap upstream face	m^3	168.75	540	91,125	6,834,375
	Sub total				**297,842**	**22,338,128**
5	Powerhouse					
	Earthwork Excavation					
	Earth excavation	m^3	3.875	2,340.00	9,068	680,063
	Excavation in boulder mixed soil	m^3	4.425	1,560.00	6,903	517,725
	Brick work in 1:4 Cement mortar	m^3	145.9	184.2	26,875	2,015,609

COST ESTIMATION

Quantity Estimation of Civil Works

S.N.	Item Description	Unit	Rate	Quantity	Amount(US$)	
			(US$)		US$	NRs
	Concrete work					
	C35	m^3	245.975	407	100,112	7,508,387
	Reinforcement bars	tons	2350.55	5	11,753	881,456
	Form work (4 to 5 m high)	m^2	7.375	3,257.30	24,023	1,801,694
	12.5 mm thick 1:3 Cement sand Plastering					
	Walls, floor	m^2	6.075	1,040.90	6,323	474,260
	1:1 Cement sand punning	m^2	8.5	360.6	3,065	229,883
	Wood works for frame	m^3	64.5	134.9	8,701	652,579
	Door shutters	m^2	64.5	12	774	58,050
	Window shutters	m^2	64.5	72	4,644	348,300
	Rolling Shutters	m^2	63.725	12	765	57,353
	Roof truss	kg	4.075	12,133.00	49,442	3,708,148
	C.G.I. Sheets in roof	m^2	39.975	441.6	17,653	1,323,972
	G.I. Ridging	rm	23.95	38.5	922	69,156
	White washing 2 coats on wall	m^2	0.35	1,102.00	386	28,928
	Protection works					
	Stone Masonry (1:3 cement sand mortar)	m^2	123.55	346.2	42,773	3,207,976
	Filter clothes	m^3	77.125	360.6	27,811	2,085,846
	Gravel	rm	30	216.4	6,492	486,900
	75 mm dia PVC pipe for weep holes @ 2m c/c	LS	10.85	200	2,170	162,750
	Drainage facilities				10,000	750,000
	Sub total				360,654	27,049,032
	Sanitary fittings @7.5%				27,049	2,028,677
	Total				**387,703**	**29,077,709**
6	Tailrace					
	Earth excavation	m^3	3.875	1,200.00	4,650	348,750
	Back Filling	m^3	2.675	100	268	20,063
	Concrete work					
	C25	m^3	206.15	81	16,698	1,252,361
	Filter material	m^3	77.125	28.5	2,198	164,855
	Reinforcement bars	tons	2350.55	1.1	2,586	193,920
	Form work	m^2	7.375	390	2,876	215,719
	Boulder riprap	m^3	168.75	100	16,875	1,265,625
	Gabion Works	m^3	140.325	32	4,490	336,780
	Bank protection work	LS			6000	450,000
	Sub total				**56,641**	**4,248,073**
	Total civil cost without river diversion				**2,790,539**	**209,290,430**
	Grand total civil cost				**2,832,397**	**212,429,787**

PENSTOCK OPTIMIZATION

HEWA KHOLA SMALL HYDROPOWER PROJECT

Revenue loss in Energy generation due to head loss:

Head (m):	65.450	
Design flow at (40%) Q_d (m3/sec):	7.8m^3/s	
Overall effency :	0.9	
Dry season outage % :	5	
Wet season outage % :	10	
Dry season energy (NRs/kwh) :	8.400	
Wet season energy (NRs/kwh) :	4.800	
Length (m):	133	
k/d =	0.000030	
Fricition factor 'f' =	0.0105	*(From Moody Chart)*
Diameter 'D' in m =	1.500	

Energy calcutaton

Month	Days	River flow m^3/sec	Design flow m3/sec	Discharge for energy generation (m^3/s)	Velocity (m/s)	Head loss (m)	Inlet+Bent loss (m)	Total head loss (m)	Power loss (KW)	Energy loss KW-hr		Actual energy loss KW-hr		Revenue loss in NRs (,000)	
										wet	dry	wet	dry	wet	dry
Jan	31	3.130	7.8	3.13	1.77	0.15	0.12	0.27	7.47	0	5559.27	0	5281.31	0	44363.00
Feb	28	2.640	7.8	2.64	1.49	0.11	0.09	0.19	4.48	0	3012.96	0	2862.31	0	24043.41
Mar	31	2.520	7.8	2.52	1.43	0.10	0.08	0.18	3.90	0	2901.26	0	2756.20	0	23152.05
Apr	15	3.200	7.8	3.2	1.81	0.16	0.13	0.28	7.98	0	2874.51	0	2730.79	0	22938.63
Apr	15	3.200	7.8	3.2	1.81	0.16	0.13	0.28	7.98	2874.51	0	2587.06	0.00	12417.90	0.00
May	31	5.480	7.8	5.48	3.10	0.46	0.37	0.83	40.10	29835.04	0	26851.54	0.00	128887.39	0.00
Jun	30	14.050	7.8	7.8	4.41	0.92	0.75	1.68	115.64	83258.46	0	74932.61	0.00	359676.54	0.00
Jul	31	28.730	7.8	7.8	4.41	0.92	0.75	1.68	115.64	86033.74	0	77430.37	0.00	371665.75	0.00
Aug	31	27.250	7.8	7.8	4.41	0.92	0.75	1.68	115.64	86033.74	0	77430.37	0.00	371665.75	0.00
Sep	30	25.920	7.8	7.8	4.41	0.92	0.75	1.68	115.64	83258.46	0	74932.61	0.00	359676.54	0.00
Oct	31	11.240	7.8	7.8	4.41	0.92	0.75	1.68	115.64	86033.74	0	77430.37	0.00	371665.75	0.00
Nov	30	5.550	7.8	5.55	3.14	0.47	0.38	0.85	41.66	29993.25	0	26993.92	0.00	129570.83	0.00
Dec (1-15)	15	3.760	7.8	3.76	2.13	0.21	0.18	0.39	12.95	4663.14	0	4196.82	0.00	20144.75	0.00
Dec (15-31)	16	3.760	7.8	3.76	2.13	0.21	0.18	0.39	12.95	0.00	4974.01	0	4725.31	0	39692.61
								Total wet season energy =				**442785.67**		2125371.204	
								Total dry seson energy =					**18355.92**		154189.70
												Total revenue loss		**2279560.90**	

Diameter 'D' = 1.600 m.
k/d = 0.000028
Fricition factor 'f' = 0.0106 *(From Moody Chart)*
Energy calcutaton

Month	**Days**	**River flow m^3/sec**	**Design flow m3/sec**	**Discharge for energy generation (m^3/s)**	**Velocity (m/s)**	**Head loss (m)**	**Inlet+Bent loss (m)**	**Total head loss (m)**	**Power loss (KW)**	**Energy loss KW-hr**		**Actual energy loss KW-hr**		**Revenue loss in NRs (,000)**	
										wet	**dry**	**wet**	**dry**	**wet**	**dry**
Jan	31	3.130	7.8	3.13	1.56	0.11	0.09	0.20	5.60	0	4167.75	0	3959.36	0	33258.61
Feb	28	2.640	7.8	2.64	1.31	0.08	0.07	0.14	3.36	0	2258.79	0	2145.85	0	18025.16
Mar	31	2.520	7.8	2.52	1.25	0.07	0.06	0.13	2.92	0	2175.05	0	2066.30	0	17356.92
Apr	15	3.200	7.8	3.2	1.59	0.11	0.10	0.21	5.99	0	2155.00	0	2047.25	0	17196.91
Apr	15	3.200	7.8	3.2	1.59	0.11	0.10	0.21	5.99	2155.00	0	1939.50	0.00	9309.61	0.00
May	31	5.480	7.8	5.48	2.73	0.33	0.29	0.62	30.06	22367.11	0	20130.40	0.00	96625.90	0.00
Jun	30	14.050	7.8	7.8	3.88	0.68	0.58	1.26	86.69	62418.24	0	56176.42	0.00	269646.80	0.00
Jul	31	28.730	7.8	7.8	3.88	0.68	0.58	1.26	86.69	64498.85	0	58048.96	0.00	278635.02	0.00
Aug	31	27.250	7.8	7.8	3.88	0.68	0.58	1.26	86.69	64498.85	0	58048.96	0.00	278635.02	0.00
Sep	30	25.920	7.8	7.8	3.88	0.68	0.58	1.26	86.69	62418.24	0	56176.42	0.00	269646.80	0.00
Oct	31	11.240	7.8	7.8	3.88	0.68	0.58	1.26	86.69	64498.85	0	58048.96	0.00	278635.02	0.00
Nov	30	5.550	7.8	5.55	2.76	0.34	0.30	0.64	31.23	22485.71	0	20237.14	0.00	97138.28	0.00
Dec (1-15)	15	3.760	7.8	3.76	1.87	0.16	0.14	0.29	9.71	3495.92	0	3146.33	0.00	15102.37	0.00
Dec (15-31)	16	3.760	7.8	3.76	1.87	0.16	0.14	0.29	9.71	0.00	3728.98	0	3542.53	0	29757.25
									Total wet seson energy =			**331953.09**		1593374.826	
										Total dry seson energy =			**13761.29**		115594.86
												Total revenue loss		**1708969.68**	

Diameter 'D' = 1.700 m.

k/d = 0.000026

Fricition factor 'f' = 0.0106 *(From Moody Chart)*

Energy calcutaton

Month	Days	River flow m^3/sec	Design flow m3/sec	Discharge for energy generation (m^3/s)	Velocity (m/s)	Head loss (m)	Inlet+Bent loss (m)	Total head loss (m)	Power loss (KW)	Energy loss KW-hr		Actual energy loss KW-hr		Revenue loss in NRs (,000)	
										wet	dry	wet	dry	wet	dry
Jan	31	3.130	7.8	3.13	1.38	0.08	0.07	0.15	4.26	0	3167.00	0	3008.65	0	25272.66
Feb	28	2.640	7.8	2.64	1.16	0.06	0.05	0.11	2.55	0	1716.42	0	1630.60	0	13697.02
Mar	31	2.520	7.8	2.52	1.11	0.05	0.05	0.10	2.22	0	1652.79	0	1570.15	0	13189.23
Apr	15	3.200	7.8	3.2	1.41	0.08	0.08	0.16	4.55	0	1637.55	0	1555.67	0	13067.65
Apr	15	3.200	7.8	3.2	1.41	0.08	0.08	0.16	4.55	1637.55	0	1473.79	0.00	7074.22	0.00
May	31	5.480	7.8	5.48	2.41	0.25	0.23	0.47	22.84	16996.39	0	15296.75	0.00	73424.40	0.00
Jun	30	14.050	7.8	7.8	3.44	0.50	0.46	0.96	65.88	47430.57	0	42687.52	0.00	204900.08	0.00
Jul	31	28.730	7.8	7.8	3.44	0.50	0.46	0.96	65.88	49011.59	0	44110.43	0.00	211730.08	0.00
Aug	31	27.250	7.8	7.8	3.44	0.50	0.46	0.96	65.88	49011.59	0	44110.43	0.00	211730.08	0.00
Sep	30	25.920	7.8	7.8	3.44	0.50	0.46	0.96	65.88	47430.57	0	42687.52	0.00	204900.08	0.00
Oct	31	11.240	7.8	7.8	3.44	0.50	0.46	0.96	65.88	49011.59	0	44110.43	0.00	211730.08	0.00
Nov	30	5.550	7.8	5.55	2.45	0.25	0.23	0.48	23.73	17086.52	0	15377.86	0.00	73813.75	0.00
Dec (1-15)	15	3.760	7.8	3.76	1.66	0.12	0.11	0.22	7.38	2656.49	0	2390.84	0.00	11476.03	0.00
Dec (15-31)	16	3.760	7.8	3.76	1.66	0.12	0.11	0.22	7.38	0.00	2833.59	0	2691.91	0	22612.04
									Total wet seson energy =			**252245.59**		1210778.809	
										Total dry seson energy =			**10456.98**		87838.59
												Total revenue loss		**1298617.40**	

Diameter 'D' = 1.800 m.
k/d = 0.000025
Fricition factor 'f' = 0.0106 *(From Moody Chart)*
Energy calcutaton

Month	Days	River flow m^3/sec	Design flow m3/sec	Discharge for energy generation (m^3/s)	Velocity (m/s)	Head loss (m)	Inlet+Bent loss (m)	Total head loss (m)	Power loss (KW)	Energy loss KW-hr		Actual energy loss KW-hr		Revenue loss in NRs (,000)	
										wet	dry	wet	dry	wet	dry
Jan	31	3.130	7.8	3.13	1.23	0.06	0.06	0.12	3.29	0	2446.68	0	2324.35	0	19524.54
Feb	28	2.640	7.8	2.64	1.04	0.04	0.04	0.08	1.97	0	1326.03	0	1259.73	0	10581.71
Mar	31	2.520	7.8	2.52	0.99	0.04	0.04	0.08	1.72	0	1276.87	0	1213.03	0	10189.42
Apr	15	3.200	7.8	3.2	1.26	0.06	0.06	0.12	3.51	0	1265.10	0	1201.84	0	10095.49
Apr	15	3.200	7.8	3.2	1.26	0.06	0.06	0.12	3.51	1265.10	0	1138.59	0.00	5465.23	0.00
May	31	5.480	7.8	5.48	2.15	0.19	0.18	0.36	17.65	13130.66	0	11817.59	0.00	56724.45	0.00
Jun	30	14.050	7.8	7.8	3.07	0.38	0.36	0.74	50.89	36642.76	0	32978.49	0.00	158296.74	0.00
Jul	31	28.730	7.8	7.8	3.07	0.38	0.36	0.74	50.89	37864.19	0	34077.77	0.00	163573.30	0.00
Aug	31	27.250	7.8	7.8	3.07	0.38	0.36	0.74	50.89	37864.19	0	34077.77	0.00	163573.30	0.00
Sep	30	25.920	7.8	7.8	3.07	0.38	0.36	0.74	50.89	36642.76	0	32978.49	0.00	158296.74	0.00
Oct	31	11.240	7.8	7.8	3.07	0.38	0.36	0.74	50.89	37864.19	0	34077.77	0.00	163573.30	0.00
Nov	30	5.550	7.8	5.55	2.18	0.19	0.18	0.37	18.33	13200.29	0	11880.26	0.00	57025.24	0.00
Dec (1-15)	15	3.760	7.8	3.76	1.48	0.09	0.08	0.17	5.70	2052.29	0	1847.06	0.00	8865.88	0.00
Dec (15-31)	16	3.760	7.8	3.76	1.48	0.09	0.08	0.17	5.70	0.00	2189.11	0	2079.65	0	17469.06
									Total wet seson energy =			**194873.79**		935394.1916	
										Total dry seson energy =			**8078.60**		67860.22
												Total revenue loss		**1003254.41**	

Diameter 'D' = 1.900 m.

k/d = 0.000024

Fricition factor 'f' = 0.0106 *(From Moody Chart)*

Energy calcutaton

Month	Days	River flow m^3/sec	Design flow m3/sec	Discharge for energy generation	Velocity (m/s)	Head loss (m)	Inlet+Bent loss (m)	Total head loss (m)	Power loss (KW)	Energy loss KW-hr		Actual energy loss KW-hr		Revenue loss in NRs (,000)	
										wet	dry	wet	dry	wet	dry
Jan	31	3.130	7.8	3.13	1.10	0.05	0.05	0.09	2.58	0	1918.20	0	1822.29	0	15307.27
Feb	28	2.640	7.8	2.64	0.93	0.03	0.03	0.07	1.55	0	1039.61	0	987.63	0	8296.08
Mar	31	2.520	7.8	2.52	0.89	0.03	0.03	0.06	1.35	0	1001.07	0	951.01	0	7988.52
Apr	15	3.200	7.8	3.2	1.13	0.05	0.05	0.10	2.76	0	991.84	0	942.25	0	7914.88
Apr	15	3.200	7.8	3.2	1.13	0.05	0.05	0.10	2.76	991.84	0	892.66	0.00	4284.75	0.00
May	31	5.480	7.8	5.48	1.93	0.14	0.14	0.29	13.84	10294.46	0	9265.02	0.00	44472.07	0.00
Jun	30	14.050	7.8	7.8	2.75	0.29	0.29	0.58	39.90	28728.00	0	25855.20	0.00	124104.94	0.00
Jul	31	28.730	7.8	7.8	2.75	0.29	0.29	0.58	39.90	29685.59	0	26717.04	0.00	128241.77	0.00
Aug	31	27.250	7.8	7.8	2.75	0.29	0.29	0.58	39.90	29685.59	0	26717.04	0.00	128241.77	0.00
Sep	30	25.920	7.8	7.8	2.75	0.29	0.29	0.58	39.90	28728.00	0	25855.20	0.00	124104.94	0.00
Oct	31	11.240	7.8	7.8	2.75	0.29	0.29	0.58	39.90	29685.59	0	26717.04	0.00	128241.77	0.00
Nov	30	5.550	7.8	5.55	1.96	0.14	0.15	0.29	14.37	10349.05	0	9314.14	0.00	44707.89	0.00
Dec (1-15)	15	3.760	7.8	3.76	1.33	0.07	0.07	0.13	4.47	1609.00	0	1448.10	0.00	6950.86	0.00
Dec (15-31)	16	3.760	7.8	3.76	1.33	0.07	0.07	0.13	4.47	0.00	1716.26	0	1630.45	0	13695.78
										Total wet seson energy =		**152781.41**		733350.7654	
											Total dry seson energy =		**6333.63**		53202.53
												Total revenue loss		**786553.30**	

Diameter 'D' = 2.000 m.

k/d = 0.000023

Fricition factor 'f' = 0.0106 *(From Moody Chart)*

Energy calcutaton

Month	Days	River flow m^3/sec	Design flow m3/sec	Discharge for energy generation	Velocity (m/s)	Head loss (m)	Inlet+Bent loss (m)	Total head loss (m)	Power loss (KW)	Energy loss KW-hr		Actual energy loss KW-hr		Revenue loss in NRs (,000)	
										wet	dry	wet	dry	wet	dry
Jan	31	3.130	7.8	3.13	1.00	0.04	0.04	0.07	2.05	0	1523.80	0	1447.61	0	12159.91
Feb	28	2.640	7.8	2.64	0.84	0.03	0.03	0.05	1.23	0	825.85	0	784.56	0	6590.30
Mar	31	2.520	7.8	2.52	0.80	0.02	0.02	0.05	1.07	0	795.24	0	755.47	0	6345.98
Apr	15	3.200	7.8	3.2	1.02	0.04	0.04	0.08	2.19	0	787.91	0	748.51	0	6287.48
Apr	15	3.200	7.8	3.2	1.02	0.04	0.04	0.08	2.19	787.91	0	709.11	0.00	3403.75	0.00
May	31	5.480	7.8	5.48	1.74	0.11	0.12	0.23	10.99	8177.79	0	7360.01	0.00	35328.06	0.00
Jun	30	14.050	7.8	7.8	2.48	0.22	0.24	0.46	31.70	22821.16	0	20539.05	0.00	98587.43	0.00
Jul	31	28.730	7.8	7.8	2.48	0.22	0.24	0.46	31.70	23581.87	0	21223.68	0.00	101873.67	0.00
Aug	31	27.250	7.8	7.8	2.48	0.22	0.24	0.46	31.70	23581.87	0	21223.68	0.00	101873.67	0.00
Sep	30	25.920	7.8	7.8	2.48	0.22	0.24	0.46	31.70	22821.16	0	20539.05	0.00	98587.43	0.00
Oct	31	11.240	7.8	7.8	2.48	0.22	0.24	0.46	31.70	23581.87	0	21223.68	0.00	101873.67	0.00
Nov	30	5.550	7.8	5.55	1.77	0.11	0.12	0.23	11.42	8221.16	0	7399.04	0.00	35515.40	0.00
Dec (1-15)	15	3.760	7.8	3.76	1.20	0.05	0.06	0.11	3.55	1278.17	0	1150.35	0.00	5521.68	0.00
Dec (15-31)	16	3.760	7.8	3.76	1.20	0.05	0.06	0.11	3.55	0.00	1363.38	0	1295.21	0	10879.76
										Total wet seson energy =		**121367.66**		582564.7693	
											Total dry seson energy =		**5031.36**		42263.43
												Total revenue loss		**624828.20**	

Diameter 'D' = 2.100 m.

k/d = 0.000021

Fricition factor 'f' = 0.0106 *(From Moody Chart)*

Energy calcutaton

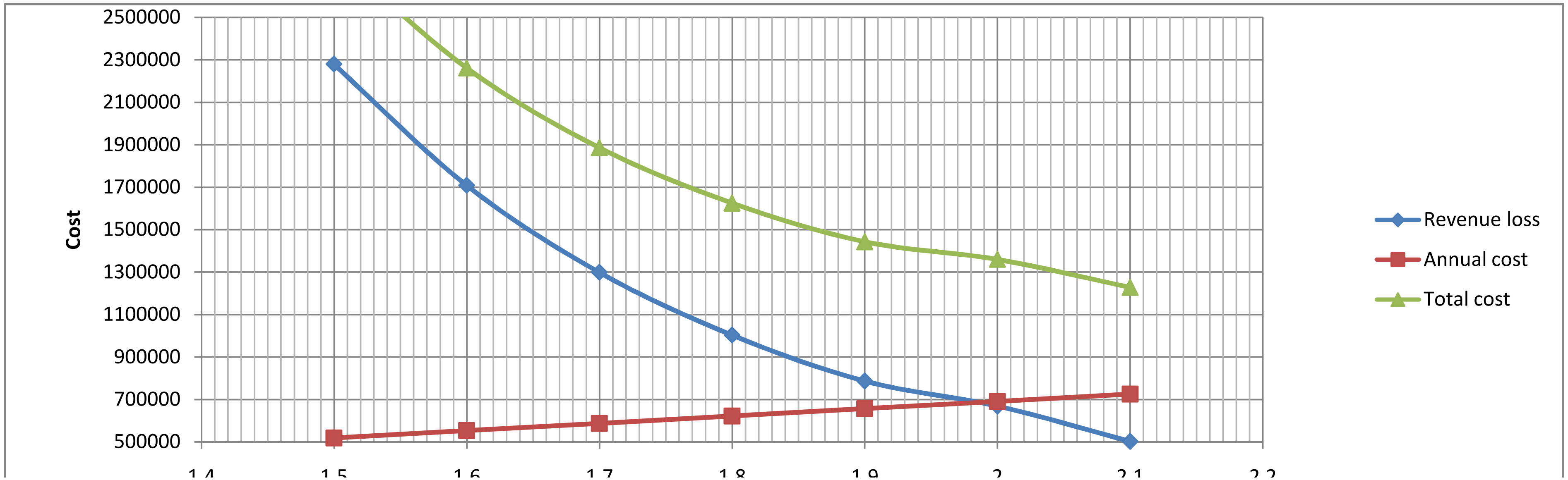
Month	**Days**	**River flow m^3/sec**	**Design flow m3/sec**	**Discharge for energy generation**	**Velocity (m/s)**	**Head loss (m)**	**Inlet+Bent loss (m)**	**Total head loss (m)**	**Power loss (KW)**	**Energy loss KW-hr**		**Actual energy loss KW-hr**		**Revenue loss in NRs**	
										wet	**dry**	**wet**	**dry**	**wet**	**dry**
Jan	31	3.130	7.8	3.13	0.90	0.03	0.03	0.06	1.65	0	1224.91	0	1163.66	0	9774.76
Feb	28	2.640	7.8	2.64	0.76	0.02	0.02	0.04	0.99	0	663.86	0	630.67	0	5297.62
Mar	31	2.520	7.8	2.52	0.73	0.02	0.02	0.04	0.86	0	639.25	0	607.29	0	5101.23
Apr	15	3.200	7.8	3.2	0.92	0.03	0.03	0.06	1.76	0	633.36	0	601.69	0	5054.20
Apr	15	3.200	7.8	3.2	0.92	0.03	0.03	0.06	1.76	633.36	0	570.02	0.00	2736.11	0.00
May	31	5.480	7.8	5.48	1.58	0.09	0.10	0.18	8.84	6573.73	0	5916.35	0.00	28398.50	0.00
Jun	30	14.050	7.8	7.8	2.25	0.17	0.20	0.37	25.48	18344.82	0	16510.34	0.00	79249.61	0.00
Jul	31	28.730	7.8	7.8	2.25	0.17	0.20	0.37	25.48	18956.31	0	17060.68	0.00	81891.27	0.00
Aug	31	27.250	7.8	7.8	2.25	0.17	0.20	0.37	25.48	18956.31	0	17060.68	0.00	81891.27	0.00
Sep	30	25.920	7.8	7.8	2.25	0.17	0.20	0.37	25.48	18344.82	0	16510.34	0.00	79249.61	0.00
Oct	31	11.240	7.8	7.8	2.25	0.17	0.20	0.37	25.48	18956.31	0	17060.68	0.00	81891.27	0.00
Nov	30	5.550	7.8	5.55	1.60	0.09	0.10	0.19	9.18	6608.59	0	5947.73	0.00	28549.09	0.00
Dec (1-15)	15	3.760	7.8	3.76	1.09	0.04	0.05	0.09	2.85	1027.46	0	924.71	0.00	4438.61	0.00
Dec (15-31)	16	3.760	7.8	3.76	1.09	0.04	0.05	0.09	2.85	0.00	1095.95	0	1041.16	0	8745.70
									Total wet seson energy =			**97561.53**		468295.3303	
										Total dry seson energy =			**4044.47**		33973.51
												Total revenue loss		**502268.84**	

HEWA KHOLA SMALL HYFROPOWER PROJECT
COST OPTIMIZATION

Present worth factor = 11%
Return peroid (yrs) = **50**

S.No	Dameter (m)	Cost NRs.	Revenue loss NRs.	Annual cost Nrs	Total cost NRs.
1	1.5	4,710,998.43	-	518,209.83	518,209.83
2	1.6	5,025,064.99	-	552,757.15	552,757.15
3	1.7	5,339,131.55	-	587,304.47	587,304.47
4	1.8	5,653,198.11	-	621,851.79	621,851.79
5	1.9	5,967,264.67	-	656,399.11	656,399.11
6	2.0	6,281,331.24	-	690,946.44	690,946.44
7	2.1	6,595,397.80	-	725,493.76	725,493.76

Optimum diameter = 1970 mm
Thickness = 8 8 mm



ENERGY CALCULATION SHEET

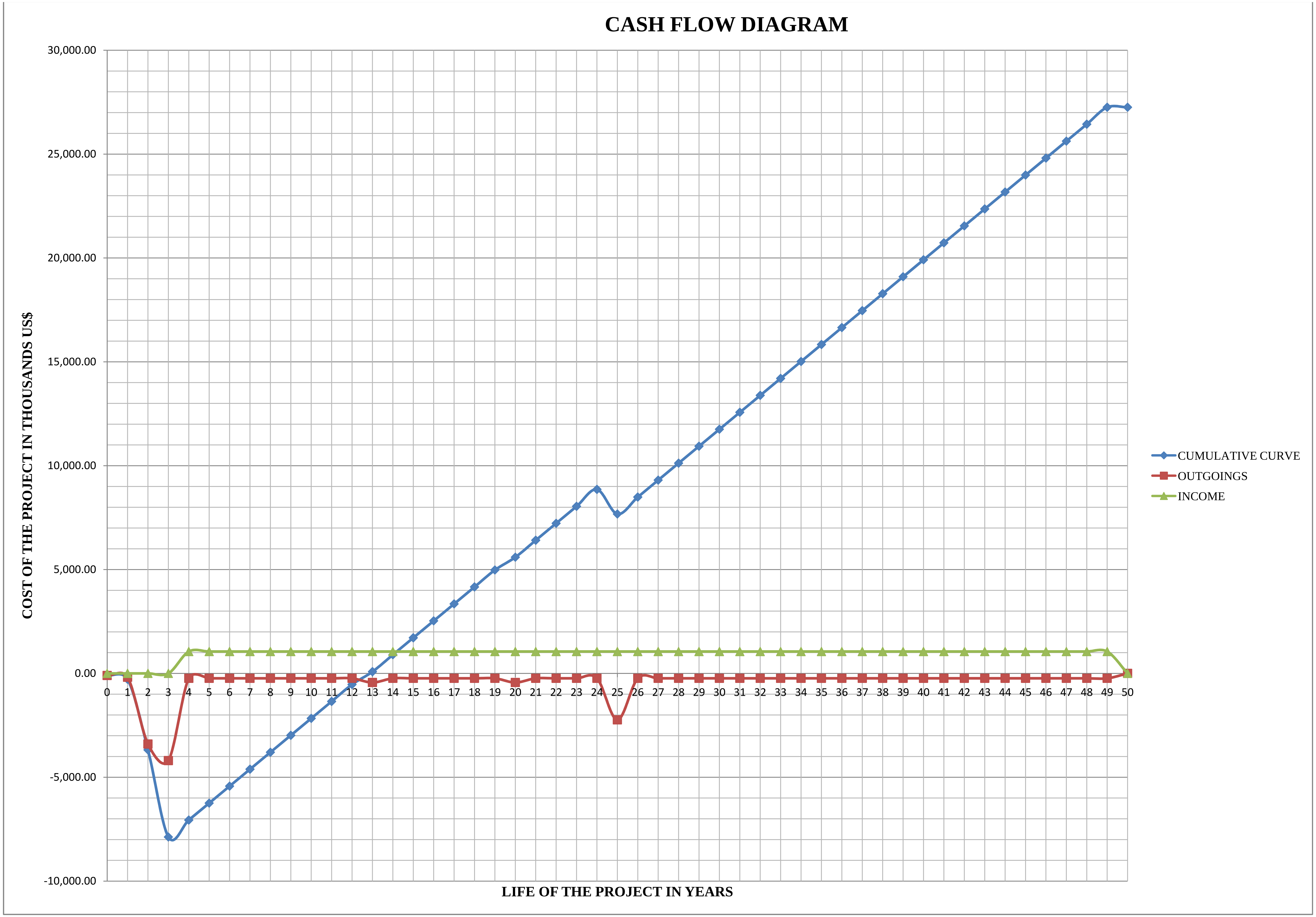
Net Head (m):	55.630	Dry energy rate: NRs. 8.40/KWh
Design flow at (40%) Q_d (m3/sec):	7.8	Wet energy rate: NRs. 4.80/KWh
Overall effency :	0.9	
Dry season outage % :	5	
Wet season outage % :	10	

Month	**Days**	**River flow m^3/sec**	**Design flow m3/sec**	**Discharge for energy generation (m^3/s)**	**Discharge after riparian release (m3/s)**	**Power available (MW)**		**Power after outage(MW)**		**Energy available KW-h**		**Annual Income (NRs)**		Remarks
						Wet season	**Dry season**	**Wet season**	**Dry season**	**Wet season**	**Dry season**	**Wet season**	**Dry season**	
Jan	31	3.130	7.8	3.13	2.817	-	1383.59003	-	1314.410528	-	98738.51887	-	829403.5585	
Feb	28	2.640	7.8	2.64	2.376	-	1166.989674	-	1108.64019	-	70243.44243	-	590044.9164	
Mar	31	2.520	7.8	2.52	2.268	-	1113.944688	-	1058.247454	-	64002.80601	-	537623.5705	
Apr	15	3.200	7.8	3.2	2.88	-	1414.532938	-	1343.806291	-	103204.3231	-	866916.3143	
Apr	15	3.200	7.8	3.2	2.88	1414.532938	-	1273.079644	-	458308.6718	-	2199881.625	-	
May	31	5.480	7.8	5.48	4.932	2422.387656	-	2180.14889	-	1622030.774	-	7785747.716	-	
Jun	30	14.050	7.8	7.8	7.02	3447.924035	-	3103.131632	-	2234254.775	-	10724422.92	-	
Jul	31	28.730	7.8	7.8	7.02	3447.924035	-	3103.131632	-	2308729.934	-	11081903.68	-	
Aug	31	27.250	7.8	7.8	7.02	3447.924035	-	3103.131632	-	2308729.934	-	11081903.68	-	
Sep	30	25.920	7.8	7.8	7.02	3447.924035	-	3103.131632	-	2234254.775	-	10724422.92	-	
Oct	31	11.240	7.8	7.8	7.02	3447.924035	-	3103.131632	-	2308729.934	-	11081903.68	-	
Nov	30	5.550	7.8	5.55	4.995	2453.330564	-	2207.997507	-	1589758.205	-	7630839.385	-	
Dec (1-15)	15	3.760	7.8	3.76	3.384	1662.076202	-	1495.868582	-	538512.6893	-	2584860.909	-	
Dec (15-31)	16	3.760	7.8	3.76	3.384	-	1662.076202	-	1578.972392	-	142486.4686		1196886.336	
									Total	**15603309.69**	**478675.56**	**74895886.53**	**4020874.696**	
									Grand total	**16081985.25**		**78,916,761.22**		

Annual Income in US $ **1,052,223.48**

CASH FLOW OF THE PROJECT

Descriptions of the works	Years	Outgoing		Income		Cumulative	
			(,000)		(,000)		(,000)
Land purchase, contract award, mobilization etc.	0	-100,000.00	-100.00		0.00	-100,000.00	-100.00
Access road, Establishment of offices, insurance etc	1	-183,000.00	-183.00		0.00	-283,000.00	-283.00
Civil works, hydromechanical cost etc.	2	-3,398,876.59	-3,398.88		0.00	-3,681,876.59	-3,681.88
Electromechanical works (Water to wire), transmission line, project development cost, engineering and management cost, contengency, VAT	3	-4,195,175.81	-4,195.18		0.00	-7,877,052.39	-7,877.05
Operation and maintenance co	4	-236,311.57	-236.31	1,052,223.48	1,052.22	-7,061,140.49	-7,061.14
	5	-236,311.57	-236.31	1,052,223.48	1,052.22	-6,245,228.58	-6,245.23
	6	-236,311.57	-236.31	1,052,223.48	1,052.22	-5,429,316.67	-5,429.32
	7	-236,311.57	-236.31	1,052,223.48	1,052.22	-4,613,404.76	-4,613.40
	8	-236,311.57	-236.31	1,052,223.48	1,052.22	-3,797,492.85	-3,797.49
	9	-236,311.57	-236.31	1,052,223.48	1,052.22	-2,981,580.95	-2,981.58
	10	-236,311.57	-236.31	1,052,223.48	1,052.22	-2,165,669.04	-2,165.67
	11	-236,311.57	-236.31	1,052,223.48	1,052.22	-1,349,757.13	-1,349.76
	12	-236,311.57	-236.31	1,052,223.48	1,052.22	-533,845.22	-533.85
Turbine maintenance	13	-436,311.57	-436.31	1,052,223.48	1,052.22	82,066.69	82.07
	14	-236,311.57	-236.31	1,052,223.48	1,052.22	897,978.60	897.98
	15	-236,311.57	-236.31	1,052,223.48	1,052.22	1,713,890.50	1,713.89
	16	-236,311.57	-236.31	1,052,223.48	1,052.22	2,529,802.41	2,529.80
	17	-236,311.57	-236.31	1,052,223.48	1,052.22	3,345,714.32	3,345.71
	18	-236,311.57	-236.31	1,052,223.48	1,052.22	4,161,626.23	4,161.63
	19	-236,311.57	-236.31	1,052,223.48	1,052.22	4,977,538.14	4,977.54
	20	-436,311.57	-436.31	1,052,223.48	1,052.22	5,593,450.04	5,593.45
	21	-236,311.57	-236.31	1,052,223.48	1,052.22	6,409,361.95	6,409.36
	22	-236,311.57	-236.31	1,052,223.48	1,052.22	7,225,273.86	7,225.27
	23	-236,311.57	-236.31	1,052,223.48	1,052.22	8,041,185.77	8,041.19
	24	-236,311.57	-236.31	1,052,223.48	1,052.22	8,857,097.68	8,857.10
Turbine exchange	25	-2,236,311.57	-2,236.31	1,052,223.48	1,052.22	7,673,009.59	7,673.01
	26	-236,311.57	-236.31	1,052,223.48	1,052.22	8,488,921.49	8,488.92
	27	-236,311.57	-236.31	1,052,223.48	1,052.22	9,304,833.40	9,304.83
	28	-236,311.57	-236.31	1,052,223.48	1,052.22	10,120,745.31	10,120.75
	29	-236,311.57	-236.31	1,052,223.48	1,052.22	10,936,657.22	10,936.66
	30	-236,311.57	-236.31	1,052,223.48	1,052.22	11,752,569.13	11,752.57
	31	-236,311.57	-236.31	1,052,223.48	1,052.22	12,568,481.03	12,568.48
	32	-236,311.57	-236.31	1,052,223.48	1,052.22	13,384,392.94	13,384.39
	33	-236,311.57	-236.31	1,052,223.48	1,052.22	14,200,304.85	14,200.30
	34	-236,311.57	-236.31	1,052,223.48	1,052.22	15,016,216.76	15,016.22
	35	-236,311.57	-236.31	1,052,223.48	1,052.22	15,832,128.67	15,832.13
	36	-236,311.57	-236.31	1,052,223.48	1,052.22	16,648,040.58	16,648.04
	37	-236,311.57	-236.31	1,052,223.48	1,052.22	17,463,952.48	17,463.95
	38	-236,311.57	-236.31	1,052,223.48	1,052.22	18,279,864.39	18,279.86
	39	-236,311.57	-236.31	1,052,223.48	1,052.22	19,095,776.30	19,095.78
	40	-236,311.57	-236.31	1,052,223.48	1,052.22	19,911,688.21	19,911.69
	41	-236,311.57	-236.31	1,052,223.48	1,052.22	20,727,600.12	20,727.60
	42	-236,311.57	-236.31	1,052,223.48	1,052.22	21,543,512.02	21,543.51
	43	-236,311.57	-236.31	1,052,223.48	1,052.22	22,359,423.93	22,359.42
	44	-236,311.57	-236.31	1,052,223.48	1,052.22	23,175,335.84	23,175.34
	45	-236,311.57	-236.31	1,052,223.48	1,052.22	23,991,247.75	23,991.25
	46	-236,311.57	-236.31	1,052,223.48	1,052.22	24,807,159.66	24,807.16
	47	-236,311.57	-236.31	1,052,223.48	1,052.22	25,623,071.57	25,623.07
	48	-236,311.57	-236.31	1,052,223.48	1,052.22	26,438,983.47	26,438.98
	49	-236,311.57	-236.31	1,052,223.48	1,052.22	27,254,895.38	27,254.90
	50	0.00	0.00		0.00	27,254,895.38	27,254.90



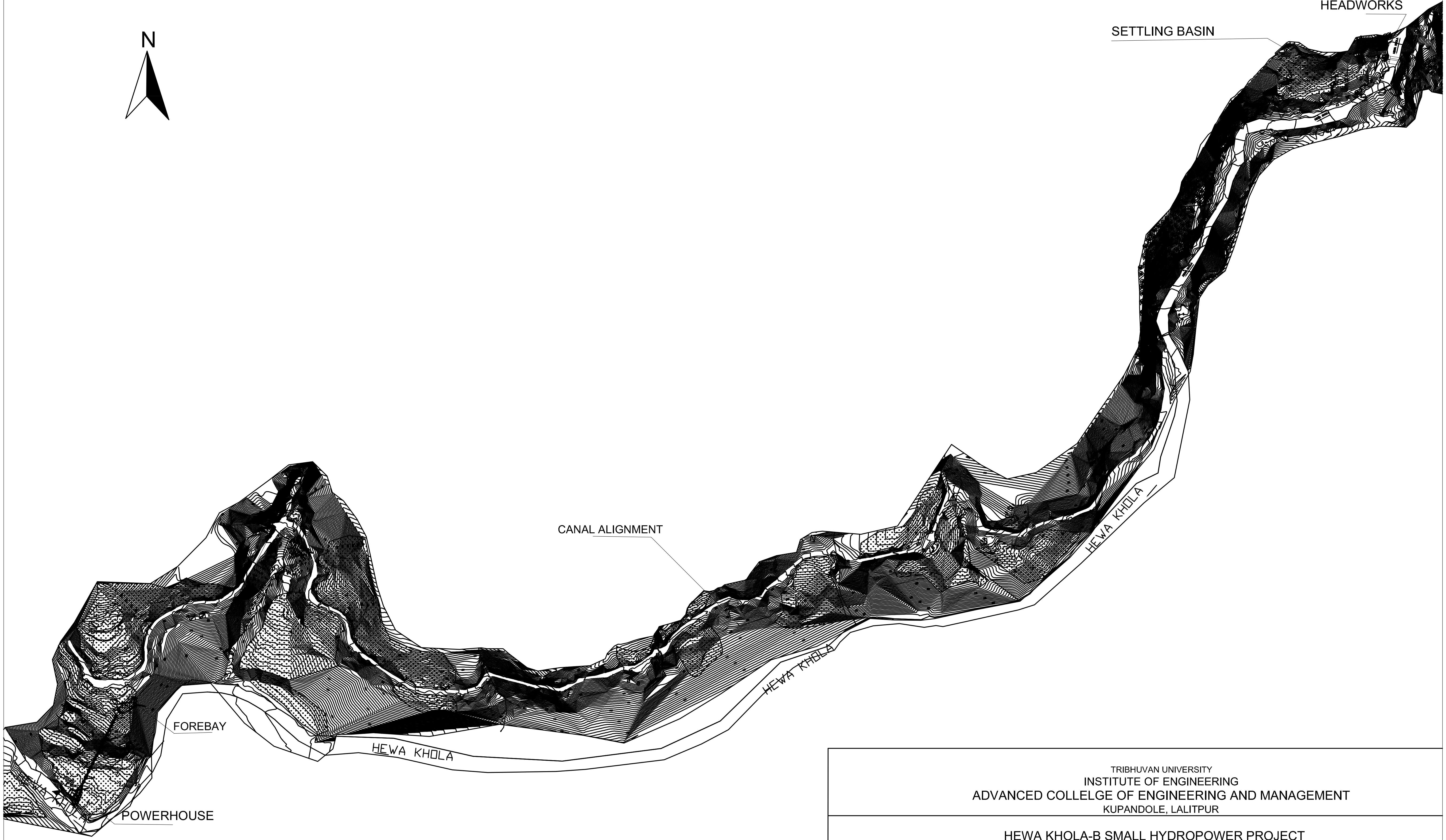
CASH FLOW DIAGRAM
30,000.00
25,000.00
20,000.00
15,000.00
10,000.00
5,000.00
0.00
-5,000.00
-10,000.00
COST OF THE PROJECT IN THOUSANDS US$
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 31 32 33 34 35 36 37 38 39 40 41 42 43 44 45 46 47 48 49 50
LIFE OF THE PROJECT IN YEARS
CUMULATIVE CURVE
OUTGOINGS
INCOME

CONSTRUCTION SCHEDULE (BAR CHART)

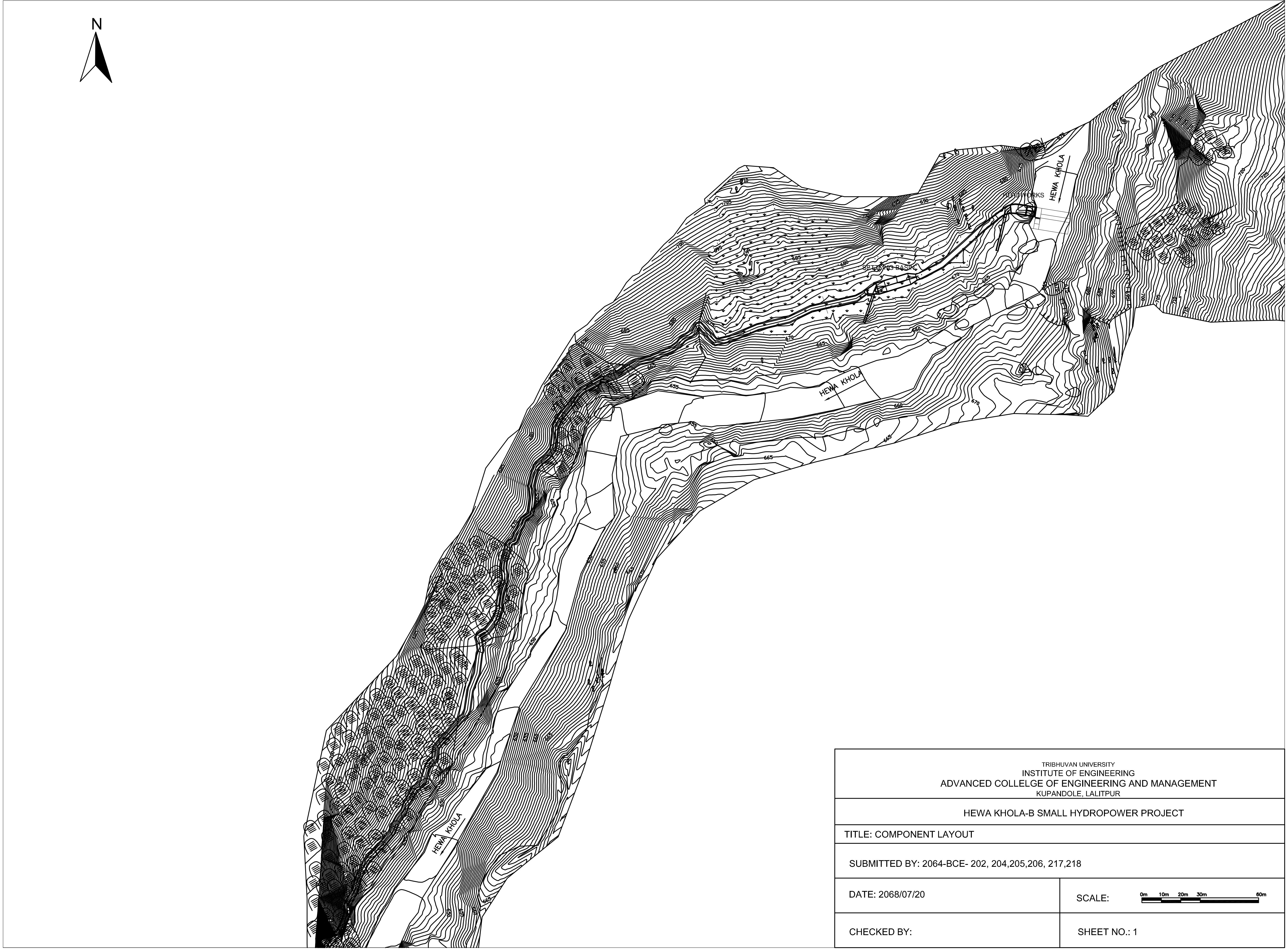
HEWA KHOLA-B SMALL HYDROPOWER PROJECT

SN	Activities / Time in months	FIRST YEAR												SECOND YEAR												THIRD YEAR											
		Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1	Invitation of bids																																				
2	Evaluation of bids and agreement																																				
3	Construction of access road																																				
4	Transportation of construction materials																																				
5	Earthwork in excavation																																				
6	Construction of diversion structures																																				
7	Construction of intake, settling basin and forebay																																				
8	Canal construction																																				
9	Construction of cross drainage structures																																				
10	Construction of penstock pipe																																				
11	Construction of powerhouse																																				
12	Construction of transmission lines																																				
13	Hydromechanical works																																				
14	Mechanical works																																				
15	Electrification																																				
16	Commercial date of operation																																				

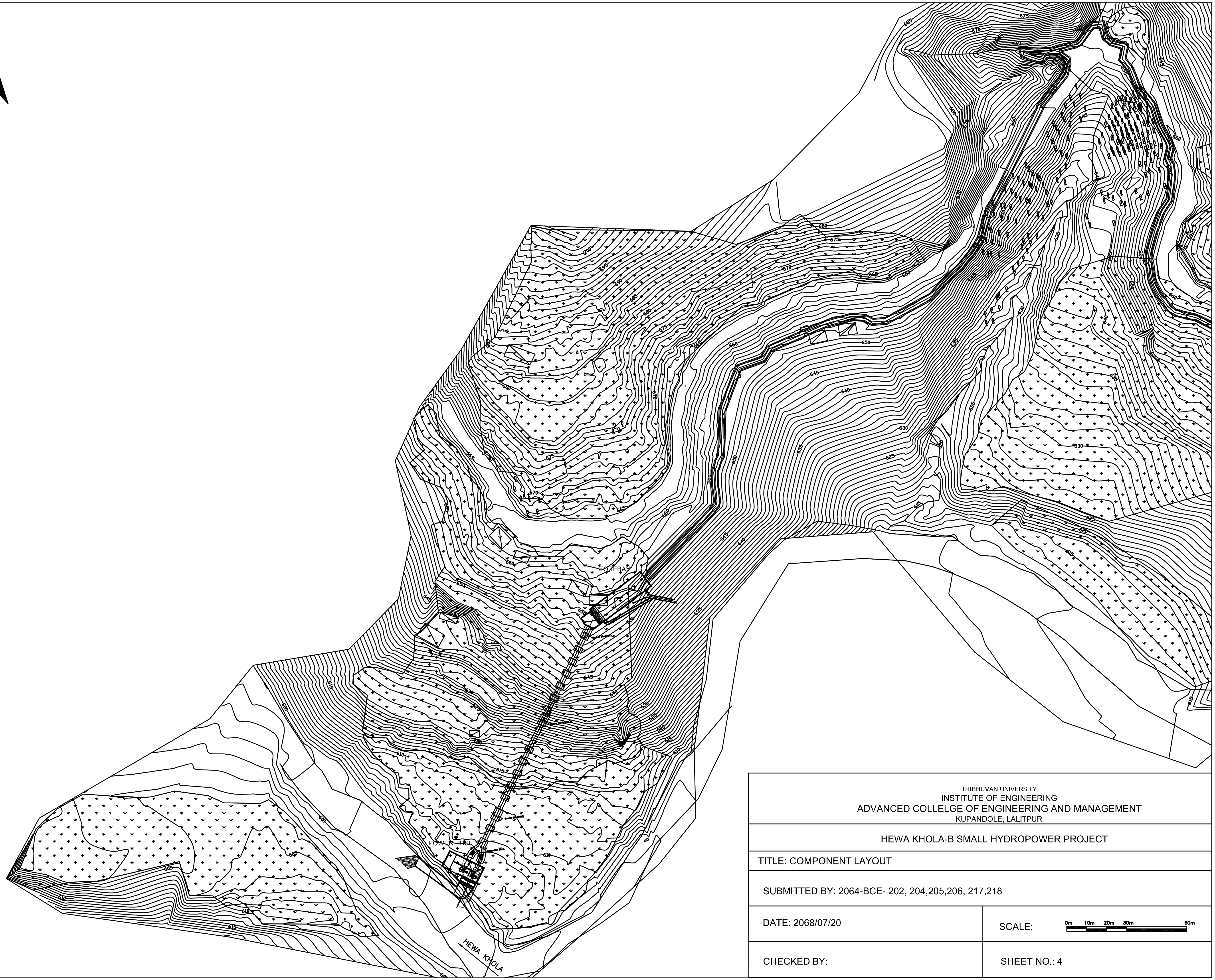
DRAWINGS



TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING ADVANCED COLLELGE OF ENGINEERING AND MANAGEMENT KUPANDOLE, LALITPUR	
HEWA KHOLA-B SMALL HYDROPOWER PROJECT	
TITLE: LOCATION MAP OF HEWA KHOLA -B SMALL HYDROPOWER PROJECT	
SUBMITTED BY: 2064-BCE- 202, 204,205,206, 217,218	
DATE: 2068/07/20	SCALE: 0 40 80 120 240m
CHECKED BY:	SHEET NO.:

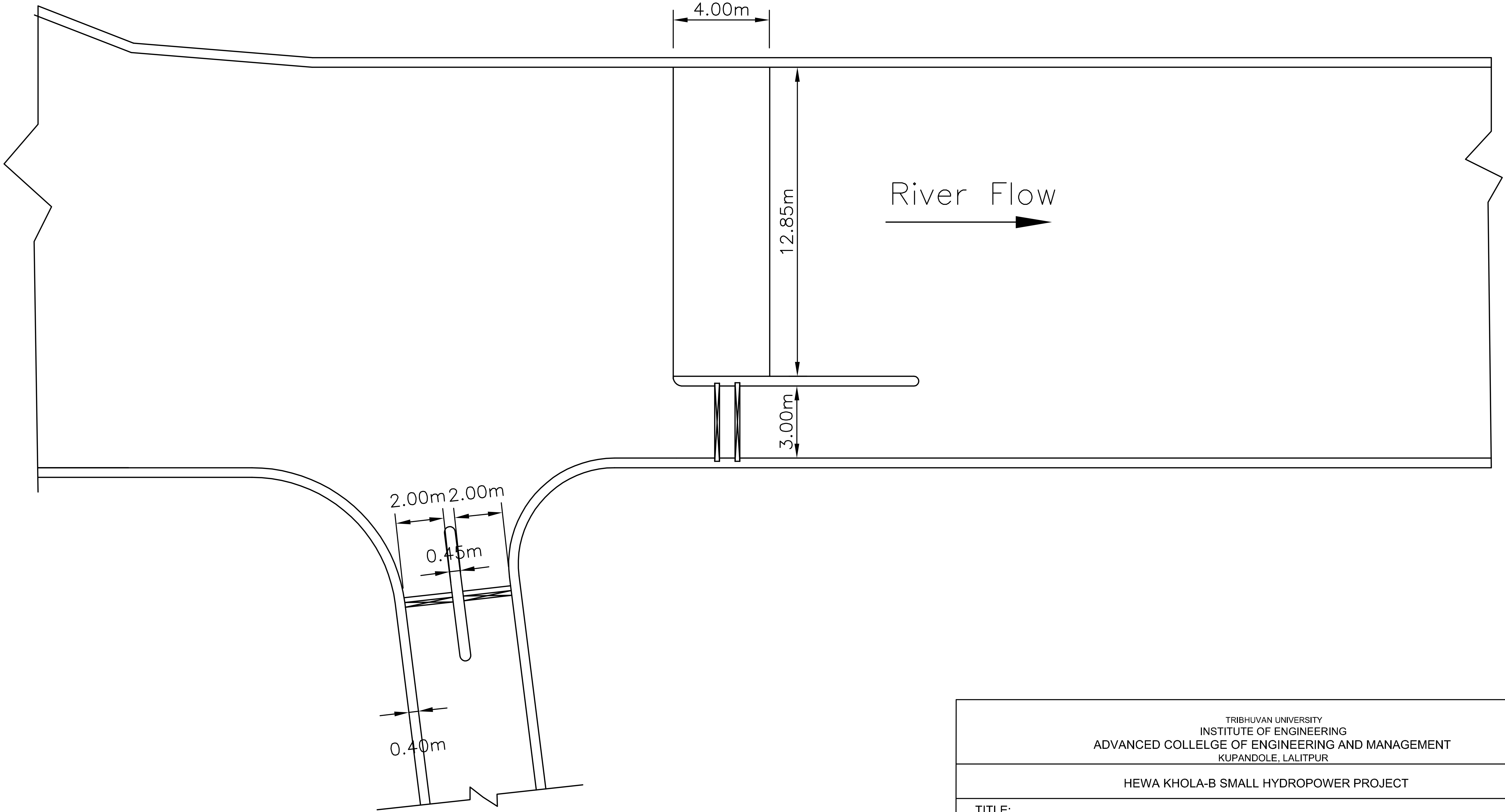


TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING ADVANCED COLLELGE OF ENGINEERING AND MANAGEMENT KUPANDOLE, LALITPUR	
HEWA KHOLA-B SMALL HYDROPOWER PROJECT	
TITLE: COMPONENT LAYOUT	
SUBMITTED BY: 2064-BCE- 202, 204,205,206, 217,218	
DATE: 2068/07/20	SCALE: 0m 10m 20m 30m 60m
CHECKED BY:	SHEET NO.: 1



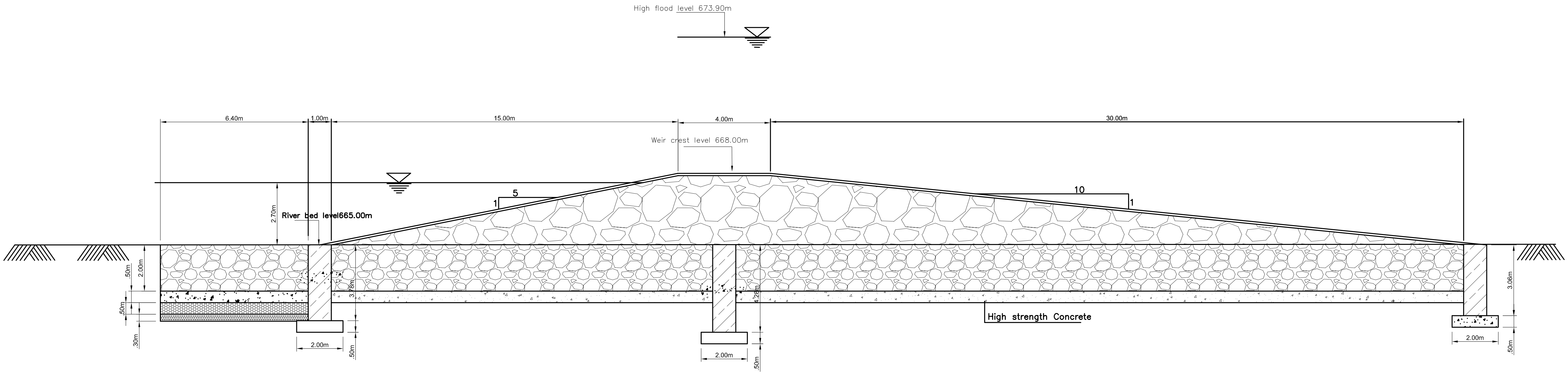
TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING ADVANCED COLLELGE OF ENGINEERING AND MANAGEMENT KUPANDOLE, LALITPUR
HEWA KHOLA-B SMALL HYDROPOWER PROJECT
TITLE: COMPONENT LAYOUT
SUBMITTED BY: 2064-BCE- 202, 204,205,206, 217,218

DATE: 2068/07/20	SCALE: 0m 10m 20m 30m 60m
CHECKED BY:	SHEET NO.: 4



PLAN OF HEADWORKS

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING ADVANCED COLLELGE OF ENGINEERING AND MANAGEMENT KUPANDOLE, LALITPUR	
HEWA KHOLA-B SMALL HYDROPOWER PROJECT	
TITLE:	
SUBMITTED BY: 2064-BCE- 202, 204,205,206, 217,218	
DATE: 2068/07/20	SCALE: 0m 1m 2m 3m 7m
CHECKED BY:	SHEET NO.: 3



SECTION OF DIVERSION WEIR

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INSTITUTE OF ENGINEERING
ADVANCED COLLELGE OF ENGINEERING AND MANAGEMENT
KUPANDOLE, LALITPUR

HEWA KHOLA-B SMALL HYDROPOWER PROJECT

TITLE: SECTION OF DIVERSION WEIR

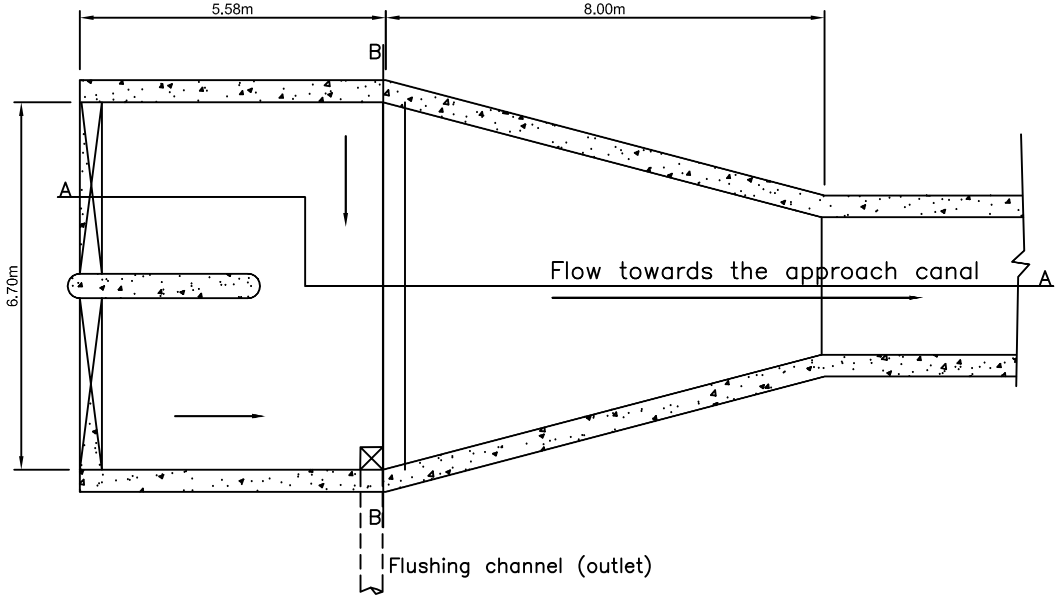
SUBMITTED BY: 2064-BCE- 202, 204,205,206, 217,218

DATE: 2068/07/20

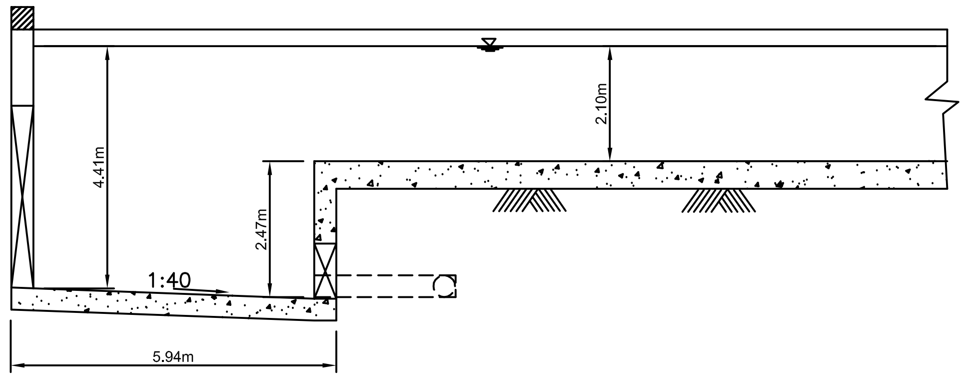
SCALE: 0m 1m 2m 3m 7m

CHECKED BY:

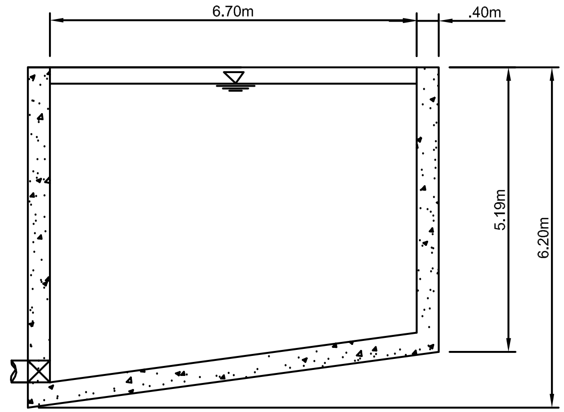
SHEET NO.: 4



PLAN OF GRAVEL TRAP

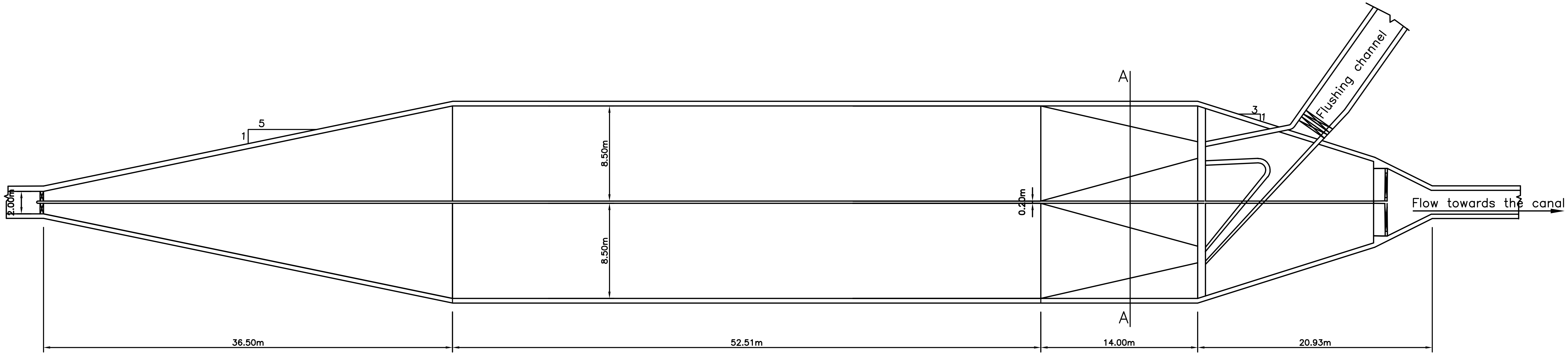


SECTION AT A-A

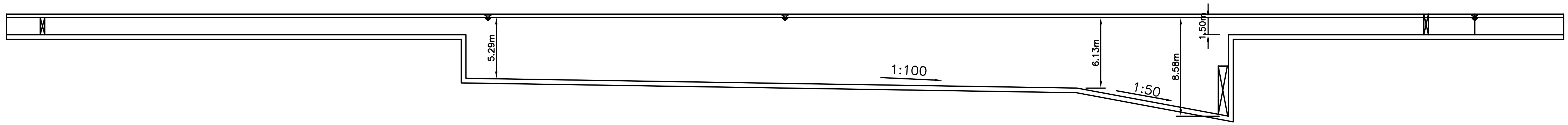


SECTION AT B-B

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING ADVANCED COLLELGE OF ENGINEERING AND MANAGEMENT KUPANDOLE, LALITPUR	
HEWA KHOLA-B SMALL HYDROPOWER PROJECT	
TITLE:	
SUBMITTED BY: 2064-BCE- 202, 204,205,206, 217,218	
DATE: 2068/07/20	SCALE: 0m 0.5m 1m 3m
CHECKED BY:	SHEET NO.: 5

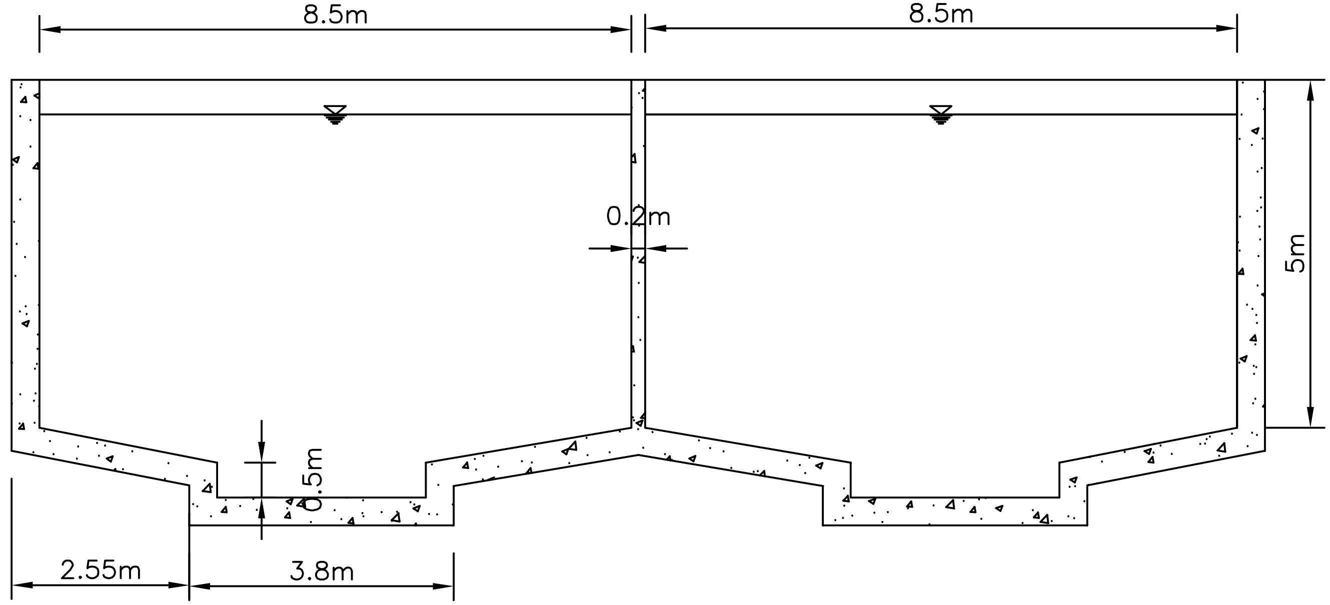


PLAN OF SETTLING BASIN

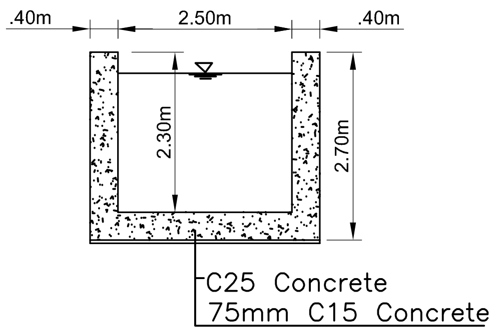


L–SECTION OF SETTLING BASIN

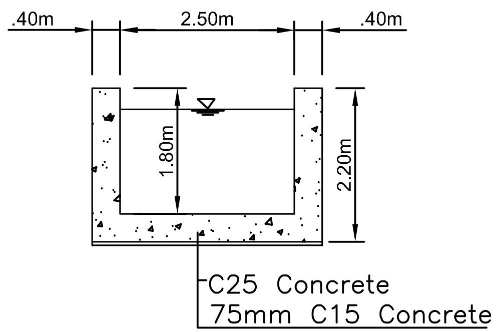
TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING ADVANCED COLLELGE OF ENGINEERING AND MANAGEMENT KUPANDOLE, LALITPUR	
HEWA KHOLA-B SMALL HYDROPOWER PROJECT	
TITLE: PLAN OF SETTLING BASIN	
SUBMITTED BY: 2064-BCE- 202, 204,205,206, 217,218	
DATE: 2068/07/20	SCALE: 0 2 4 15m
CHECKED BY:	SHEET NO.: 7



SECTION AT A–A (SETTLING BASIN)



TYPICAL CROSS SECTION OF CANAL



CROSS SECTION OF TAILRACE CANAL

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KUPANDOLE, LALITPUR

HEWA KHOLA-B SMALL HYDROPOWER PROJECT

TITLE: SECTION OF SETTLING BASIN

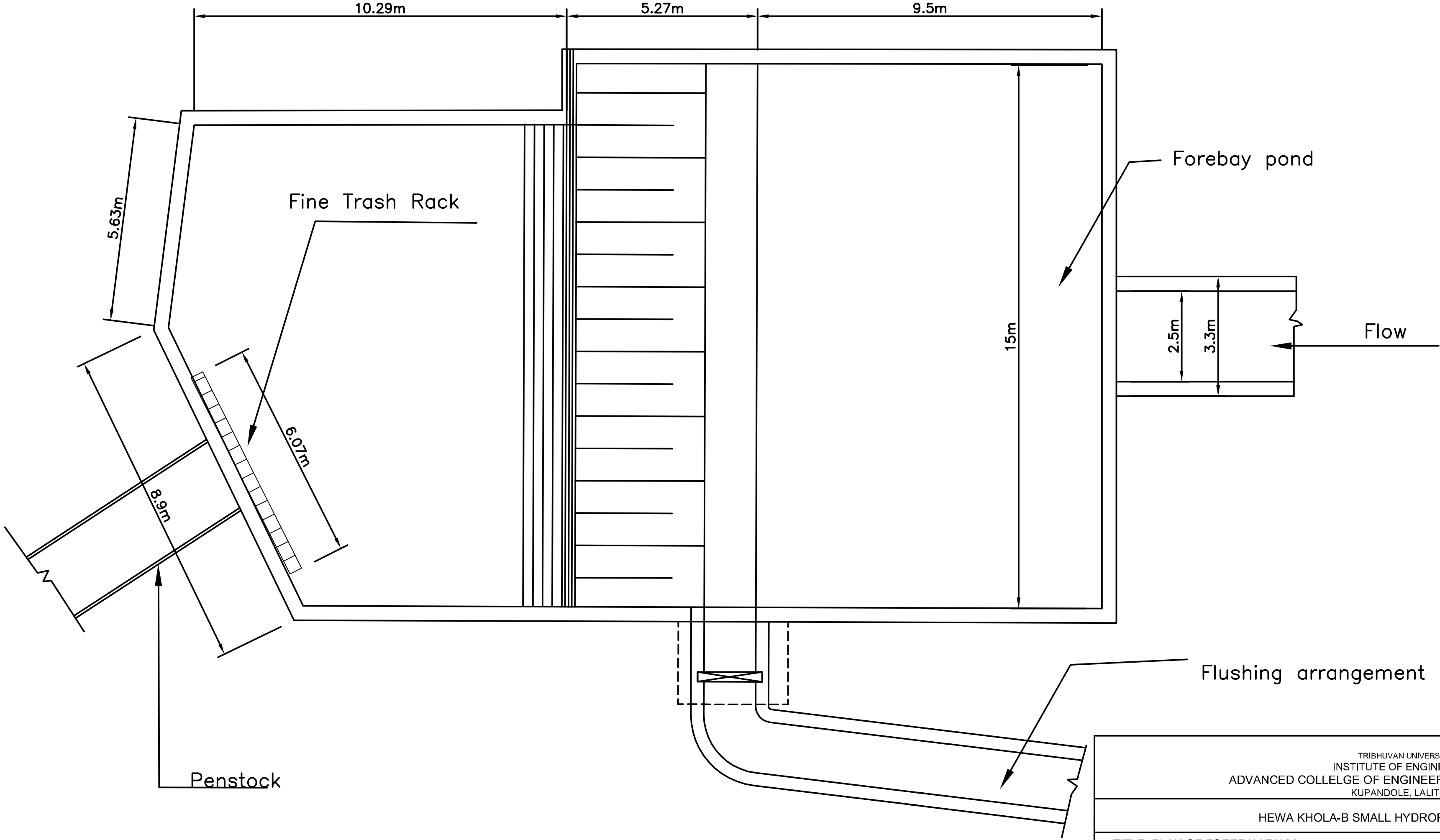
SUBMITTED BY: 2064-BCE- 202, 204,205,206, 217,218

DATE: 2068/07/20

SCALE: 0m 0.5m 1m 3m

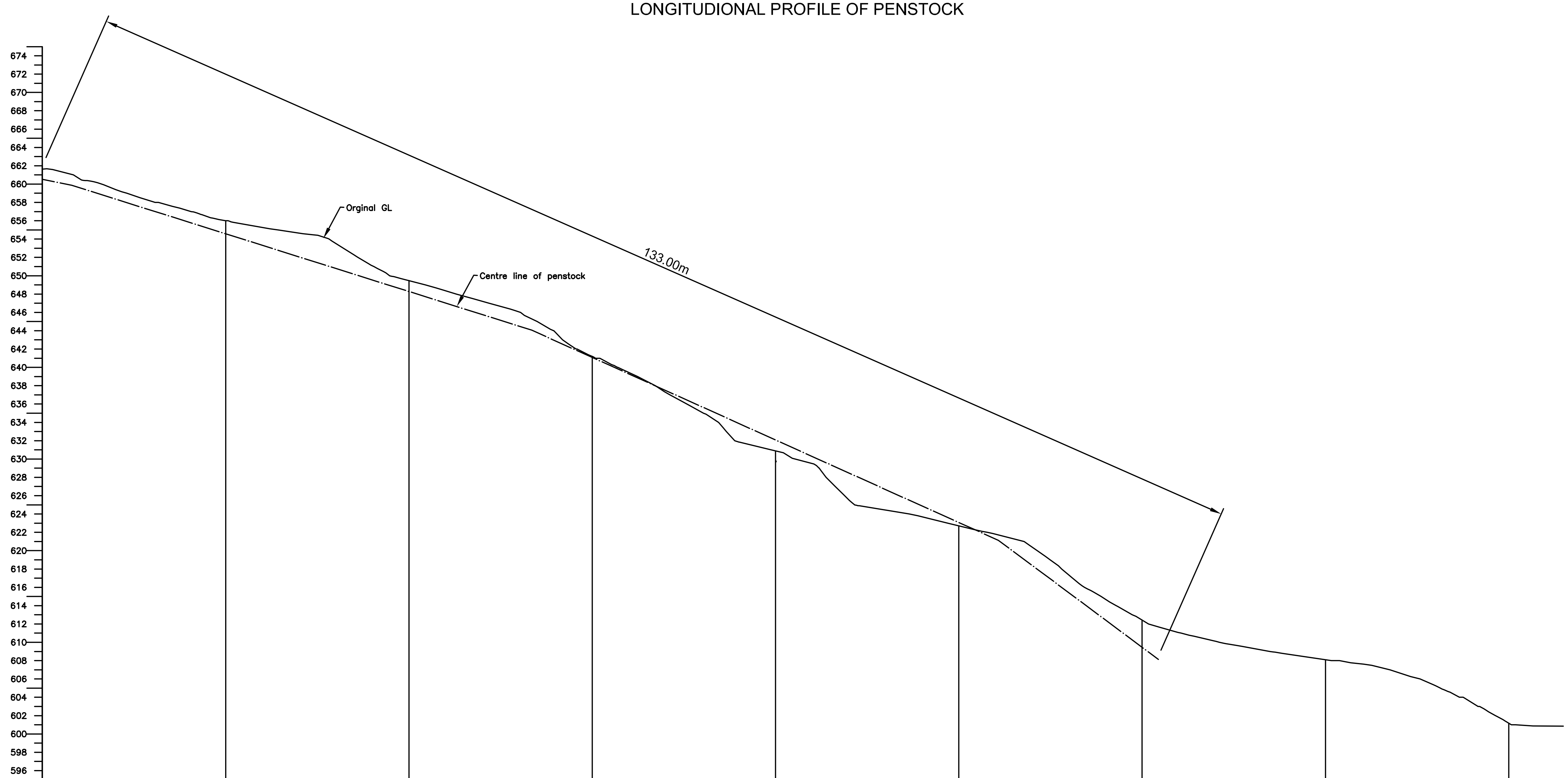
CHECKED BY:

SHEET NO.: 6



TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING ADVANCED COLLELGE OF ENGINEERING AND MANAGEMENT KUPANDOLE, LALITPUR	
HEWA KHOLA-B SMALL HYDROPOWER PROJECT	
TITLE: PLAN OF FOREBAY TANK	
SUBMITTED BY: 2064-BCE- 202, 204,205,206, 217,218	
DATE: 2068/07/20	SCALE:
CHECKED BY:	SHEET NO.: 8

LONGITUDIONAL PROFILE OF PENSTOCK



DATUM ELEV
595.000

Ground elevation (m above msl)	661.62	656.00	649.47	641.20	630.88	622.71	612.42	608.10	601.17
Penstock Center Line (m above msl)	660.6	654.728	648.338	641.085	632.080	623.074	609.457		
Chainage (m)	0+000	0+020	0+040	0+060	0+080	0+100	0+120	0+140	0+160
Penstock size	INTERNAL DIAMETER OF PENSTOCK IS 1970MM Thickness : 8mm								

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ADVANCED COLLELGE OF ENGINEERING AND MANAGEMENT
KUPANDOLE, LALITPUR

HEWA KHOLA-B SMALL HYDROPOWER PROJECT

TITLE:

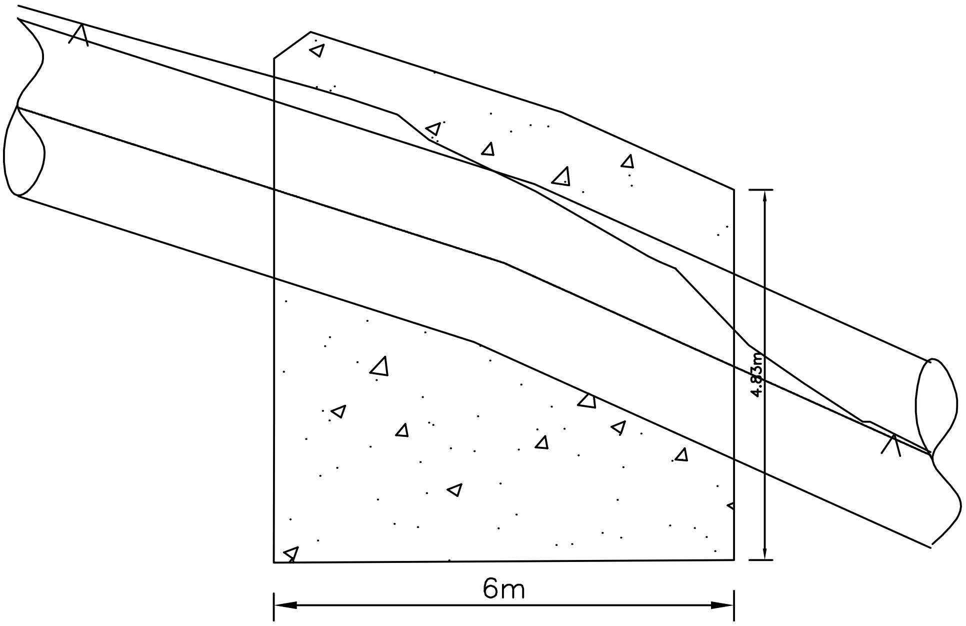
SUBMITTED BY: 2064-BCE- 202, 204,205,206, 217,218

DATE: 2068/07/20

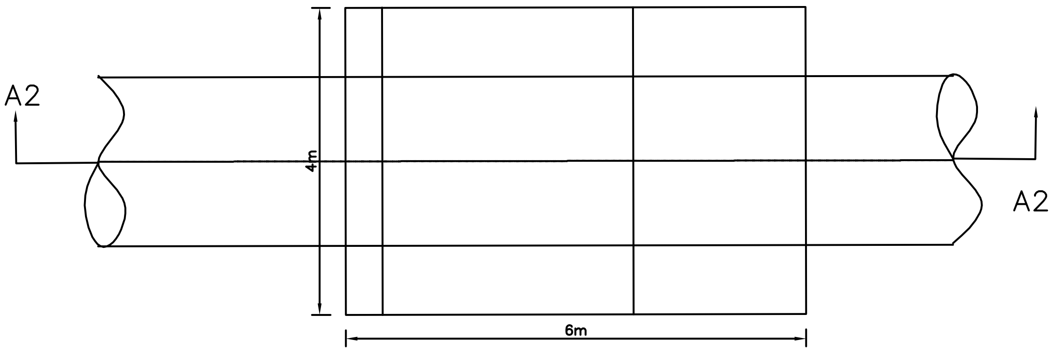
SCALE: 0 2 4 10m

CHECKED BY:

SHEET NO.: 9

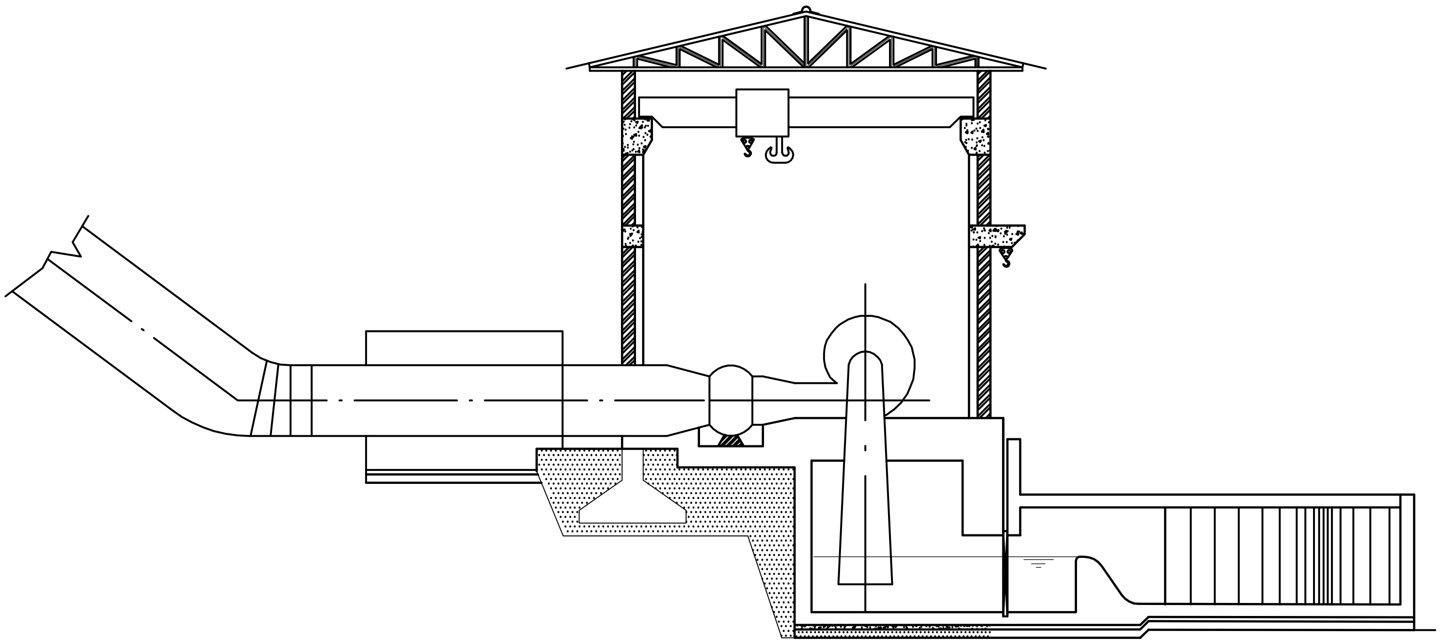


Anchor block section at A2-A2



Anchor Block Plan

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING ADVANCED COLLELGE OF ENGINEERING AND MANAGEMENT KUPANDOLE, LALITPUR	
HEWA KHOLA-B SMALL HYDROPOWER PROJECT	
TITLE: SECTION OF ANCHOR BLOCK	
SUBMITTED BY: 2064-BCE- 202, 204,205,206, 217,218	
DATE: 2068/07/20	SCALE: 0m 0.5m 1m 3m
CHECKED BY:	SHEET NO.: 10



SECTOPN AT A-A

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING ADVANCED COLLELGE OF ENGINEERING AND MANAGEMENT KUPANDOLE, LALITPUR	
HEWA KHOLA-B SMALL HYDROPOWER PROJECT	
TITLE: SECTION OF POWER HOUSE	
SUBMITTED BY: 2064-BCE- 202, 204,205,206, 217,218	
DATE: 2068/07/20	SCALE:
CHECKED BY:	SHEET NO.: 12