TRIBHUVAN UNIVERSITY
INSTITUTE OF ENGINEERING
PULCHOWK CAMPUS
DEPARTMENT OF CIVIL ENGINEERING

FINAL YEAR PROJECT REPORT on PRE-FEASIBILITY STUDY OF INDRAWATI-III RUN-OFF RIVER HYDROPOWER PROJECT

By:

Daman Raj Neupane	**(075BCE001)**
Anmol Yadav	**(075BCE021)**
Ashutosh Lamsal	**(075BCE029)**
Ashutosh Yadav	**(075BCE030)**
Basudev Chaudhary	**(075BCE032)**
Buddha Gubhaju	**(075BCE047)**

Supervisor:

Prof. Dr. Bhola Nath Sharma Ghimire

APRIL, 2023

TRIBHUVAN UNIVERSITY
INSTITUTE OF ENGINEERING
PULCHOWK CAMPUS
DEPARTMENT OF CIVIL ENGINEERING

FINAL YEAR PROJECT REPORT on PRE-FEASIBILITY STUDY OF INDRAWATI-III RUN-OFF RIVER HYDROPOWER PROJECT

IN PARTIAL FULFILMENT OF THE REQUIREMENT FOR THE AWARD OF BACHELOR DEGREE IN CIVIL ENGINEERING
(Course Code: CE755)

By:

Daman Raj Neupane	**(075BCE001)**
Anmol Yadav	**(075BCE021)**
Ashutosh Lamsal	**(075BCE029)**
Ashutosh Yadav	**(075BCE030)**
Basudev Chaudhary	**(075BCE032)**
Buddha Gubhaju	**(075BCE047)**

Supervisor:

Prof. Dr. Bhola Nath Sharma Ghimire

APRIL, 2023

COPYRIGHT

....

Head of Department
Department of Civil Engineering
Pulchowk Campus
Institute of Engineering
Lalitpur, Nepal

TRIBHUVAN UNIVERSITY
INSTITUTE OF ENGINEERING
PULCHOWK CAMPUS
DEPARTMENT OF CIVIL ENGINEERING

CERTIFICATE

This is to certify that this project work entitled "**PRE-FEASIBILITY STUDY OF INDRAWATI-III RUN-OFF RIVER HYDROPOWER PROJECT**" has been examined and declared successful for the fulfilment of academic requirement towards the completion of Bachelor Degree in Civil Engineering.

....	....
Project Supervisor	HOD, Department of Civil Engineering
Prof. Dr. Bhola Nath Sharma Ghimire	Prof. Dr. Gokarna Bahadur Motra
....	....
Internal Examiner	External Examiner
Mr. Suresh Manandhar	Prof. Dr. Prem Chandra Jha

Acknowledgement

We owe a debt of thanks to everyone who contributed their time, energy, and excitement to making this report possible. We want to thank you from the bottom of our hearts. We would like to take this opportunity to thank Pulchowk Campus for providing us with a conducive setting and an experienced facilitator. We would like to express our immense gratitude to our supervisor, Prof. Dr. Bhola Nath Sharma Ghimire, who warmly embraced us and helped us in every possible manner and provided us guidance and support leading us to complete this report on time. Similarly, to this project we owe a debt of thanks to Ms. Ishani Rijal, Civil Engineer at NHPC for helping us to access the technical details and ongoing conditions about project.

We are also thankful to our seniors for sharing with us their experience and guiding us with the preparation of the report.

Acknowledgement would be incomplete without mentioning our friends who have been constant source of inspiration during the preparation of the project.

Students' name

Daman Raj Neupane	PUL075BCE001
Anmol Yadav	PUL075BCE021
Ashutosh Lamsal	PUL075BCE029
Ashutosh Yadav	PUL075BCE030
Basudev Chaudhary	PUL075BCE032
Buddha Gubhaju	PUL075BCE047

Abstract

The study entitled "**A Pre-feasibility Study of Indrawati III ROR Hydropower Project**" is part of the partial fulfillment of a Bachelor's Degree of Civil Engineering. The Study focused on finding the technical and financial viability of power generation from Indrawati River located in Sindhupalchowk district in the Central Region of Nepal.

The Flow records and rainfall data of the nearby gauging stations and rainfall stations were obtained from the Department of Hydrology and Meteorology. The data are used to determine the flood discharge and average flow of the Indrawati River at the project Site. Using GIS software, the catchment area of the intake site of Indrawati was calculated, which is equal to **431.34 km^2**. The rainfall and discharge data obtained from the Department of Hydrology and Meteorology was analyzed and the flood discharge was derived using various methods. The flooding discharge is found **871.46 m^3/s** for 100 years return period. Based on the design discharge with Q_{40} (of 40% of discharge exceedance flow), the installed capacity is found to be **8.02 MW**. Three Francis turbines are used for the power generation. Using tunnel optimization techniques, the minimum diameter of the headrace tunnel is found **3.3 m.** Three nos. of penstocks pipes were proposed to connect the turbines. And penstock optimization techniques were applied to find the economical penstock diameter, which is 2m of each. The total annual energy production was found to be 47.686 GWh. Owing to the PPA provided by NEA, electricity will be sold at Rs. 4.8 per unit during wet season and Rs. 8.4 per unit during dry season. Based on this, the total annual revenue will be NRs 249 million.

The design life of the project is taken as 40 years. The preliminary cost estimation of the hydropower project is done for the economic analysis using Benefit Cost ratio (BCR) and IRR methods. Financial analysis shows the BCR is about 2.02 and IRR equal to 20.59% with discount period of 10 years. This suggests the economic viability of project. Hence, the project is recommended to further (feasibility) study and detail analysis as well as EIA is recommended as the capacity of the project exceeds 5MW.

Preface

This project work is not only focused on the curriculum of B.E. Civil final semester, but also available to the student community, a report that deals with various aspects under the heading "**Pre-Feasibility Study on Indrawati III Hydropower Project**".

Hydropower engineering includes great diversified nature of work from meteorological analysis to geological study, civil engineering structures, electromechanical installation, operation etc. To complete this project, the period of two semester inclusive of the regular classes, timely assessments, assignments and board exams are 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 Indrawati III hydropower station.

Every care has been taken to make this report free of errors, yet slip may occur. We warmly welcome constructive criticism and shall be obliged, if errors are brought to our notice.

Table of Contents

List of Tables

List of Figures

Abbreviations

°	Degree
°C	Degree Centigrade
amsl	above mean sea level
ArcGIS	Arc Geographic Information System
CAD	Computer Aided Design
CAR	Catchment Area Ratio
D/S	Downstream
DEM	Digital Elevation Model
DHM	Department of Hydrology and Meteorology
DoED	Department of Electricity Development
GoN	Government of Nepal
ESHA	Environmental, Safety and Health Affairs
HRT	Head Race Tunnel
GW	Gigawatt
GWh	Gigawatt hour
HFL	High Flood Level
hrs	hours
Km	Kilometre
Km^2	Square Kilometre
kW	Kilowatt
kWh	Kilowatt Hour
lps	litres per second
m	meter
m^3/s	cubic meter per second
masl	meters above sea level
MIP	Medium Irrigation Project
mm	millimeter
MW	Megawatt
MWh	Megawatt Hour
MWI	Monsoon Wetness Index
NEA	Nepal Electricity Authority
NPR	Nepalese Rupee
NRs.	Nepalese Rupees
No.	Number
PPA	Power Purchase Agreement
PRoR	Peaking Run of the River
RoR	Run of the River
Stn. No.	Station Number
U/S	Upstream
IW-III HEP	Indrawati-III Hydroelectric Project

Project Salient Features

Location		
District	:	Sindhupalchowk
Zone	:	Bagmati
Province No.	:	3
Latitude	:	27° 53’ 7” N to 27° 51’51” N
Longitude	:	85° 36’ 50” E to 85° 35’ 41” E
General		
Name of River	:	Indrawati Khola
Project Location	:	Melamchi Municipality
Type of Project	:	Run-of-River
Hydrology		
Catchment Area at Intake @ Indrawati Khola	:	431.34 km^2
Design Discharge	:	15.76 m^3/s at Q_{40}
Design Flood @ Intake (100 years Flood)	:	871.46 m^3/s
Diversion Weir		
Type	:	Broad Crested Weir
Weir Crest Level	:	923.0 m
Weir Length	:	40 m
Weir Width	:	2 m at top & 18.25 m at bottom
Intake		
Frontal Intake	:	2 m (H) x 2 m (W)
No. of openings	:	3
Intake Discharge	:	20.48 m^3/s
Settling Basin		
Type	:	Continuous
No. of Basins	:	one
Nominal Size of Trapped Particle	:	0.3 mm
Size (L x B x H)	:	88 m x10.3 m x8 m
Headrace Tunnel		
Material	:	Shotcrete
Economical Diameter	:	3.3 m
Effective thickness	:	0.05 m
Length	:	2934 m
Surge Tank		
Shape	:	Circular

Diameter	:	9 m
Height	:	16 m
Penstock Pipe		
Number of Penstock Pipe	:	3
Material	:	Steel
Economical Diameter	:	2.0 m
Length	:	300 m
Powerhouse		
Type	:	Sub-surface
Ext. Dimension (L x B x H)	:	40 m x 8 m x 15 m
Turbines		
Type of Turbine	:	Francis Turbine
No. of Units	:	Three
Rated net head	:	60 m
Rated discharge for each unit	:	5.25 m^3/s
Rated efficiency of turbine	:	0.90
Specific Speed	:	268 rpm
Diameter of turbine	:	0.7 m
Setting of turbine	:	3.58 m above TWL
Transmission Line		
Voltage	:	66 kV
Power and Energy Generation		
Gross Head	:	63 m
Rated Net Head	:	60 m
Installed Capacity	:	8.02 MW

1. Introduction

1.1 General

Hydropower specifically refers to any mechanical or electrical power that can be produced as a result of the energy head connected with moving or motionless water. According to the thesis of Dr. Hari Man Shrestha (M.Sc., hydropower engineering), Nepal has a theoretical hydropower capacity of 83,000 megawatts (MW), and its economically viable hydropower potential is 42,000 megawatts (MW). However, as of 2021, Nepal's installed hydroelectric capacity was only approximately 1,200 MW, or roughly 2% of the country's theoretical potential. But as per the latest statistics the installed capacity of the country is approximately 2500 MW where 100-500 MW of the power is being added on annual basis. Hydropower is an important sector for Nepal's economy, accounting for about a quarter of the country's total electricity generation. The sector has the potential to contribute significantly to Nepal's economic growth and development, as well as help address the country's energy needs. The recent example of Upper-Tamakoshi (456 MW) and its contribution to GDP has set an example regarding the immense potential of contribution to country’s GDP.

There are numerous on-going and upcoming hydroelectric projects in Nepal, which has a target of increasing the installed hydropower capacity to 5,000 MW by 2025. But the industry faces a number of difficulties, including a lack of investment, a weak transmission and distribution network, and unstable political conditions. In reality, unless urgent steps are done to entice both international and Nepali investors to build new hydropower projects, the situation won't get any better. After the government recently revised its hydropower policy, a new rule was established, allowing commercial hydropower developers to plan and build projects so that the dry energy should be 30% or more of the overall energy. This has given incentive to private developers to start off new projects and some projects are under construction.

Hydropower construction is capital intensive business so due to which financial resources from within the country is insufficient to think about the mega projects that can shape the future of nation towards the energy. Furthermore, the scenario of investment in the country is no that favorable for the investors (both national and foreign) because of the lack of stability indicators the foreigners seek. The clash issue of Arun-III project is a suitable example for it. Therefore, it would be definitely a wise act on the part of both the government and the national private sector to focus on the development of small and medium hydropower projects. Since these projects require less capital investment which can be generated from within the country and has a shorter gestation period. After having some experience in hydropower construction and fund management for one or two projects, achieving the dreams of mega projects would not be that far away as it is now for us.

In tune with this, the proposed Indrawati-III Hydroelectricity project is a run-of-river (RoR) type project located in Sindhupalchowk district, Central Development Region of Nepal. The project will have an installed capacity of 7.5 MW.

1.2 Need of Study

It's been nearly 25 years, since the existing Indrawati-III RoR Hydropower Project was established (2054 B.S.). There are lots of advancement in technology back then and now. The design principles and technologies may have been different. A similar project established at that very location may perform differently now as per new technology and varying design principles. Some variation could be made which could possibly increase the capacity of the plant. So, this study is of utmost as well as to determine whether any changes can be made to the existing project for better performance.

1.3 Objectives

The main Objectives of this project are:

- ✓ To learn to select and appraise possible projects of further considerations.
- ✓ To study about the prefeasibility analysis of the hydropower that is to be generated from Indrawati River.

1.4 Scope of the work

The main Scope of the project are:

- To analyze the existing hydrological and meteorological data of the project site.
- To prepare the layouts of the project at the pre-feasibility level.
- To design general components of hydropower at pre-feasibility level.
- To determine technical, economic and financial viability of the project.
- To establish the need and justification for the project.

1.5 Methodology

To prepare the report and for analysis, different methodologies have been adopted at different stage of project study.

Desk study: Under this phase we collected and reviewed topographical maps of the site, available reports, guidelines, secondary data and other information about the site. We also analyzed contour map of interval 10 m for topographical analysis.

Data collection: For gauged basin, data were collected from department of hydrology and meteorology (DHM).

Hydrological analysis: The design flow of river for hydropower design was computed. The flood of different years return period is determined by using empirical method and statistical methods.

Detailed design: The detailed design of all the hydraulic structures is to be carried out in accordance to the hydrological and topographical study.

Cost estimate of the project: The estimation of the overall cost of the project is to be done. Prefeasibility analysis must at least be done using B/C ratio, Discounted payback period and IRR method.

1.6 Limitations of the study:

- Project work is parallelly progressed with regular class and not able to be carried out the detailed field survey. Hence, the project is done based on Google map and GIS data, which might be less precise as compared to the actual field survey.
- DDC rate of all items of the VDC is unavailable and DDC rate of neighboring districts are used for rate analysis. Hence, the total cost of the project may differ than that of the real cost.
- Study of sediment data is not included. Similarly, high flood data and low flood data were not available able to be obtained due to limited timespan of the project period.
- To highlight the important project features, only a few structures (tunnel and penstocks) were optimized. Hence, project cost and benefits may differ from the real costs and benefits.

2. Literature Review

2.1 Hydropower

2.1.1 History of Hydropower

Hydropower, which is the use of water to generate electricity, has been used for thousands of years. Ancient Greece and Rome were the first civilizations to employ hydropower, turning waterwheels to grind grain and other materials. Hydropower was employed to produce electricity on a bigger scale in the late 1800s. In 1882, Appleton, Wisconsin saw the construction of the first hydroelectric power plant, while Niagara Falls saw the construction of the first significant hydroelectric power plant in 1895. During the 20th century, the development of hydropower expanded rapidly, especially in countries with abundant water resources such as the United States, Canada, Brazil, China, and Russia. Hydropower became an important source of renewable energy, and today it is the largest source of renewable electricity generation in the world.

2.1.2 Hydro Technical Review

The law of energy conservation may be used to explain how energy can be produced from water. In the penstock, the potential energy of moving water is transformed into kinetic energy. The turbine's blades rotate as a result of the water's kinetic energy, which is then transformed into mechanical energy. Electrical energy is ultimately produced when the turbine shaft turns the generator (Basnyat, 2006). The following formula describes the amount of electricity produced by utilizing the potential energy of flowing water:

$$P = \eta \times \gamma \times Q \times H$$

Where

P is the power in Kilo − Watts,

η is the general efficiency of the plant,

γ is the specific weight of water in KN/m^3,

Q is the discharge passing through the turbine in m/s.

H is the net head of the water in m

The principal requirements for electricity generation from water are given in ESHA (2005) as:

- Suitable rainfall catchment area
- Hydraulic Head
- Means of transporting water from intake to the turbine, such as pipe or millrace
- Turbine houses containing the power generation equipment and gate valve
- Tailrace to return the water to its natural course

2.1.3 Hydropower in the world

The most popular renewable energy source for energy production is hydropower. More than 150 nations across the world use hydropower. The installed capacity of 11000 stations with 27000 generating units is 860 GW, while the pumped storage plants add an additional 120–150 GW of capacity (IHA, 2010). In 63 nations, hydropower accounts for at least 50% of natural electricity production, while in 23 countries, it accounts for 90% (Yüksek et al., 2007).

2.1.4 Hydropower in Nepal

Large hydroelectric potential exists in Nepal. In reality, Nepal's steep gradient terrain and the perennial nature of its rivers make it possible for some of the biggest hydroelectric projects in the world to be built there. According to current estimations, Nepal has a hydropower potential of about 40,000 MW (Source: USAID, 2018). The technical and economic feasibility approximates to about 42 GW but the installed capacity at present day stands below 3000 MW. However, bulk of the economically feasible generation is yet to be standardized. Besides, the multipurpose, secondary and tertiary benefits have not been realized from the development of its rivers. Although bestowed with tremendous hydropower resources, about 43.6% of Nepal's population (NEA, 2009) has access to electricity. Most of the power plants in Nepal are run-of- river type with energy available in excess of the in-country demand during the monsoon season and deficit during the dry season. Nepal's electricity generation is dominated by hydropower, though in the entire scenario of energy use of the country, the electricity is a tiny fraction, only 1% energy need is fulfilled by electricity. With this scenario and having immense potential of hydropower development, it is important for Nepal to increase its energy dependency on electricity with hydropower development.

According to the Energy Demand Forecast study created by WECS in 2015, fuel wood has a percentage share of 70.47 in Nepal's energy consumption, while electricity only accounts for 3.48%. (WECS, 2015). As an alternative energy source, electricity has a very high potential, and Nepal has a large hydropower potential due to its more than 6000 rivers. The river network of Nepal counts to above 6000 rivers and rivulets whereas the lengths of these river morphology sums to about 45000 Kms (Joshi, 2016).

Table 1:Existing major hydroelectric power plants and their types (Source: NEA)

S.N	Generation Station Name	Installed Capacity in MW	Type
1.	Upper Tamakoshi	456	PROR
2.	Kaligandaki A	144	ROR
3.	Lower Marsyangdi	139	ROR
4.	Kulekhani I	60	Storage
5.	Kulekhani II	30	Storage
6.	Modi	24.8	ROR
7.	Trishuli and Chilime	82	ROR
8.	Devighat	14.1	ROR
9.	Sunkoshi	17.55	ROR

2.1.5 Pre-Feasibility and Feasibility Study of Hydropower Project

A. Pre-Feasibility Study

Pre-feasibility study is the second step of a hydropower project's design and construction process after reconnaissance research. Pre-feasibility studies establish the project's requirement and rationale through the creation of a development plan. The project's viability in terms of its technical, financial, and environmental aspects is assessed. The project's boundaries are also stated. The appropriate suggestion is made for further action and consideration following the successful pre-feasibility study. This stage gives the glimpse of the feasibility of the project.

The majority of the data used in this study came from unofficial polls and secondary sources. It is reasonable to anticipate that this research will provide information on the following: location and site, market, plant capacity, materials, project engineering and technology and equipment, manpower, financial analysis, project finance, investment, production costs, and commercial profitability.

B. Feasibility Study

The majority of the information utilized in this study was gathered from secondary sources and unofficial surveys. The following topics should be covered by this research: location and site, market, plant capacity, materials, project engineering and technology and equipment, manpower, financial analysis, project finance, investment, production costs, and commercial profitability.

All forms of research approaches are used for this study, including: To acquire full information, primary sources, including official and informal networks, are utilized. This research can be anticipated to provide information on location, technology, capacity, investment needed, production costs, sales return on investment, debt-to-equity ratio, estimated cash flows, cost-benefit analysis, pre-operative costs, and funding sources. This research report should also include a detailed implementation schedule with all relevant charts and milestones, justification of assumptions, and results gained. Pre-feasibility study preparation takes longer and costs more money than feasibility study preparation.

Feasibility stage resembles with the major process in EIA after scoping and screening. It involves the stage of preparing DPR on the basis of EIA report.

2.2 Types of Hydro-Electric Schemes

2.2.1 Run of River

Run of River plans were the name of the earliest hydroelectric plants. The plans make advantage of the river's current flow because there is no large water storage component. Run-of-river schemes are unable to produce electricity when rivers and streams are flowing insufficiently. A low-level diversion weir (a tiny dam) or a stream bed intake are typical components of run-of-river schemes, which are typically found on fast-moving streams. An intake structure may be placed on the riverside thanks to a low-level diversion weir, which elevates the water level in the river just enough. A garbage screen and submerged opening with an intake gate make up the intake. Having a streambed intake where the water descends via a screened input duct that has been built flush with the bottom of the riverbed is an alternate option that does not require a weir (see Figure below). A screen must be included in the design to remove the debris from the system since a streambed intake will enable pebbles and gravel to enter. In order to maximize the head of the turbine, water from the intake is often sent through a pipe (penstock) built downstream of the intake and downhill as much as feasible. In Nepal lot of hydropower are ROR projects. It’s because of the PPA policy and the variable rainfall distribution throughout the country. Also, the long term technical and economic benefit also suggests the construction of RoR project. Only trouble with the RoR is the low generation during the dry season which becomes as low as one third of the installed capacity.

Some of the RoR projects in Nepal are Khimti-I, Bhotekoshi, Indrawati III.

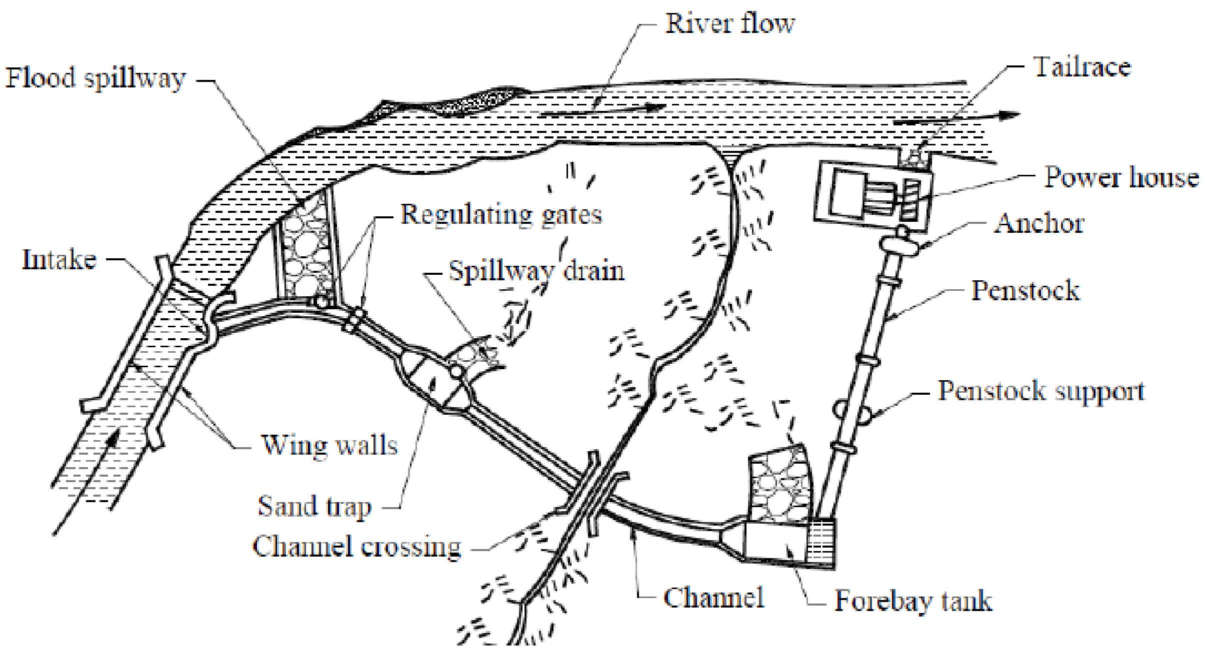


Figure 1:Layout of RoR Hydro Scheme (www.researchgate.net/General-Layout-of-the-MHP)

2.2.2 PRoR

Peaking Run-of-river (PRoR) hydropower projects must be planned to produce energy in accordance with changes in the amount of electricity required by regulating the river's daily flow on an hourly basis. When the river's flow is lower than the design discharge during dry seasons, this capacity must be achieved by regularly ponding water at the headworks. The plant must run like a RoR plant during the rainy season when the river flow exceeds the design discharge in order to assist bed load flushing. The installed capacity of PRoR systems must be sized for flows greater than the reliable river flow in order to provide peak load coverage. Peak needs should typically be established for 4 to 6 hours of plant operation during the dry season based on site characteristics and optimization.

The PRoR projects are more suited to Nepal's power system because of their increased operational flexibility, which enables them to close the gap between water supply and electricity demand. However, the expense of storage facilities added for PRoR projects means that their energy is more expensive than that produced by RoR plants. In Nepal PRoR projects are more technically and economically sustainable owing to the current policy of hydropower in Nepal.

Some of the PRoR projects in Nepal are: Marsyangdi (69 MW), Upper Tamakoshi (456MW) Sunkoshi (10 MW) etc.

2.2.3 Storage Type

The development of a sizable dam to retain water and provide enough head for the turbine is another possible foundation for hydropower. These water storage plans allow the power plant to produce during periods of peak electricity demand and then to let the water level increase again during off-peak hours. Larger, softly graded rivers are more suitable for plans including massive dams. This sort of plant has the advantage of having the capacity to store energy (water) and utilize it as needed. They are often used as the security for power generation. In Nepal, only one storage type project is running (Kulekhani-I) which is utilized for power generation during peak time and is also working as the voltage stabilizer.

2.2.4 Pumped Storage

Pumped storage facilities move water from a lower reservoir to an upper reservoir using a reversible pumping turbine to store hydro energy during off-peak electricity hours. When power is expensive to create during peak hours, this stored energy is subsequently utilised to generate electricity by transferring water from the higher to the lower reservoir. The possibility of pumped storage is found to be feasible in two famous lakes of Nepal i.e., Begnas lake and Rupa Lake where the two ponds can be used as the natural reservoirs.

2.3 Turbines

Impulse and reaction turbines are the two basic categories of hydro turbines. By hitting buckets or blades, impulse turbines transform the kinetic energy of a jet of water in the air into motion. In contrast, a reaction turbine's blades are completely submerged in the water flow, and both the angular and linear momentum of the water are transformed into shaft power. There is a more thorough discussion of turbine types.

2.3.1 Impulse Turbines

A. Pelton Turbines

The Pelton wheel was created in California during the 1850s Gold Rush. It is one of the types of the impulse turbine. The Pelton turbine is made up of a number of buckets with unique shapes positioned around the edge of a disc. Water jets that are released from one or more nozzles and strike the buckets rotate the disc. To prevent the center of the buckets from acting as a dead spot incapable of deflecting water away from the incoming jet, the buckets are divided into two parts. The lower lip's cutaway enables a smoother entry of the bucket into the jet as well as allowing the subsequent bucket to advance further before cutting off the jet. The highest angle that can be achieved without the return jet interfering with the next bucket for the approaching jet is 180 degrees, however the Pelton bucket is only intended to deflect the jet through 165 degrees. Although Pelton turbines are often only considered for heads above 150m in large-scale hydro, they may be utilized well in micro-hydro applications at heads as low as 20m. At lower heads, Pelton turbines are not employed because the needed runner is very big and cumbersome and the rotating speed becomes very sluggish. When there is a very high head available, this method extracts 90% of the energy from the water flow. The water flow must run between 70% and 80% of the maximum flow for the turbine to function at a 90% turbine efficiency.

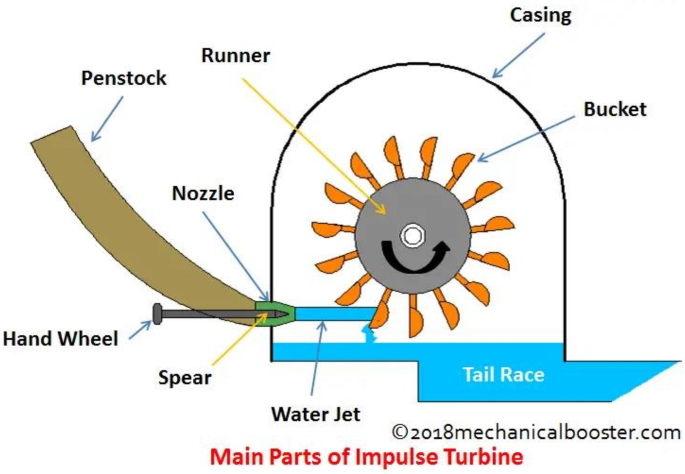


Figure 2:Main parts of Pelton turbine

B. Turgo Turbines

The Turgo turbine is similar in design to a Pelton turbine, but was designed to have a higher specific speed. In this case, the jets are aimed to strike the plane of the runner on one side and exit on the other. As a result, unlike Pelton turbines, the flow rate is not constrained by the discharged fluid interfering with the entering jet. As a result, a Turgo turbine can use a runner with a smaller diameter for similar power as a Pelton turbine. The Turgo has many of the main traits of a Pelton turbine, including the ability to be installed either horizontally or vertically, and is effective at a wide range of speeds.

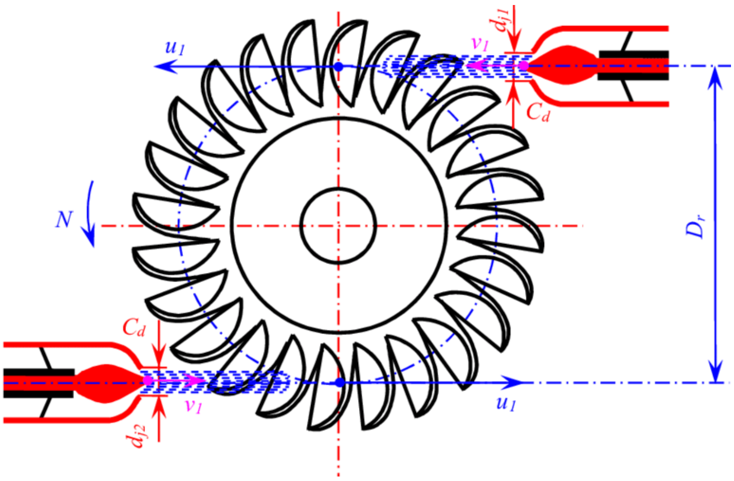


Figure 3:Turgo Turbine
(Source:www.researchgate.net/Design-parameters-of-the-hydro-Turgo-runner)

2.3.2 Reaction Turbine

A. Francis Turbine

Francis turbines may be split into two groups: vertical shaft and horizontal shaft. In reality, smaller turbines are often placed with a horizontal shaft, whereas bigger turbines are arranged with a

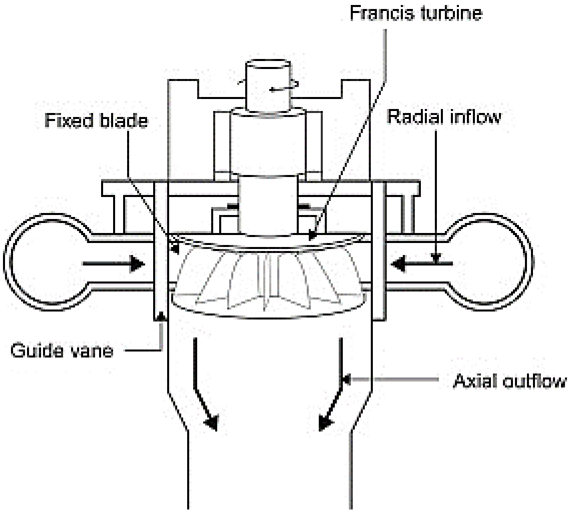


Figure 4:Francis Turbine (Source: www.sciencedirect.com)

vertical shaft. Francis turbines come in two different varieties: open-flume and volute-cased. The guiding vanes feed the water into the runner at the proper angle, and the spiral casing is tapered to evenly distribute water throughout the runner's full circumference. Typically, the Francis turbine has movable guiding vanes. The water is directed by the complexly shaped runner blades such that it leaves axially from the center of the runner. Before exiting the turbine through a draft tube, the water thus transfers the majority of its pressure energy to the runner. When the turbine's flow rate is reduced below 85% of the maximum flow, the efficiency of the turbine falls away.

B. Kaplan Propeller Turbine

The fundamental propeller turbine is made up of a propeller resembling a ship's propeller that is mounted inside a penstock tube extension (see Figure 9). At the point where the tube changes direction, the turbine shaft exits the tube. Just upstream of the propeller, there are often three to six blades or swivel gates. Since the pitch angle of the rotor blades cannot be altered, this type of propeller turbine is referred to as a fixed blade axial flow turbine. The part-flow efficiency of fixed blade propeller turbines tends to be very poor.

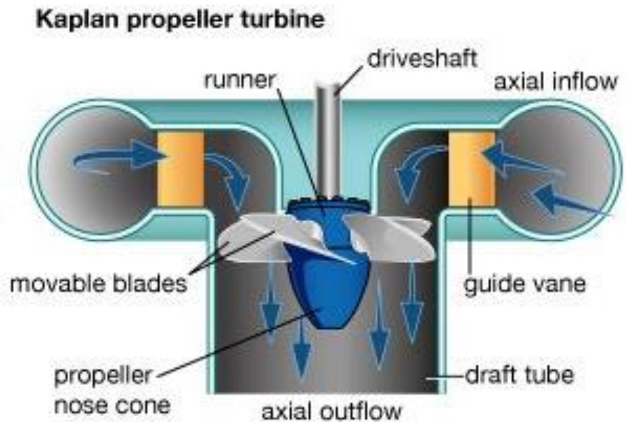


Figure 5: Kaplan Propeller turbine (Source: https://kids.britannica.com)

2.3.3 Turbine Selection Chart

The turbine selection chart below permits the user to select turbines for a given flow rate (m^3/s) and head (m).

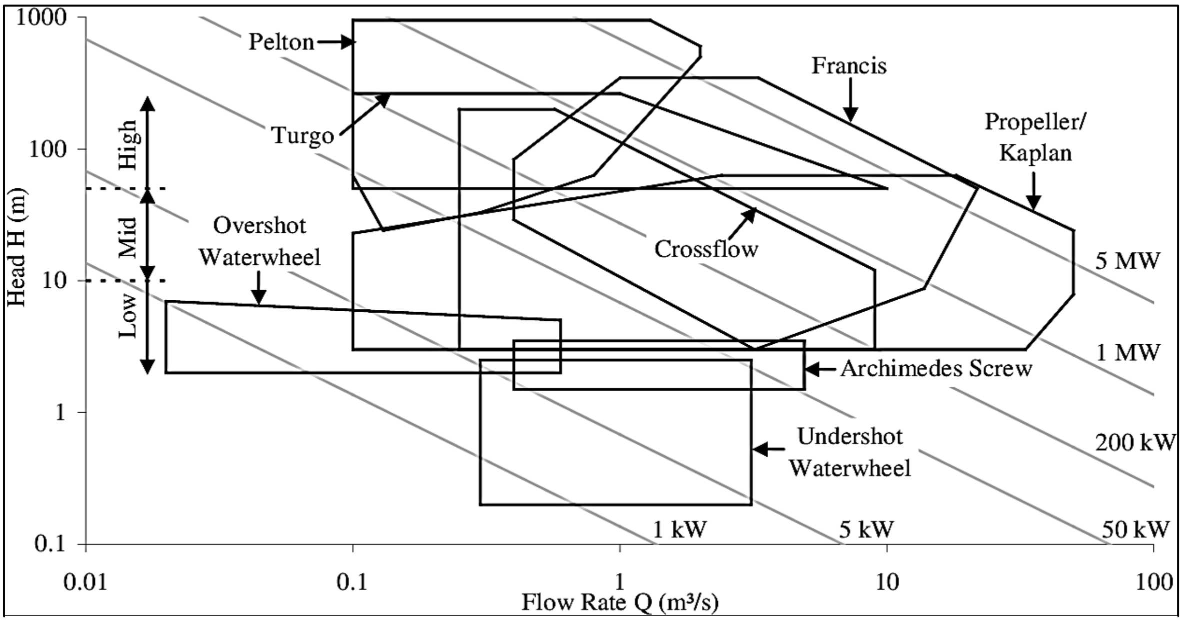


Figure 6: Turbine selection chart (Source: https://www.semanticscholar.org)

2.4 Governors

Turbine governors are tools for quick control and adjusting the output of the turbine and balancing variations between power and grid demand. Mechanical hydraulic, electrohydraulic, or digital hydraulic governor systems are all possible. No of the type, all systems have the following three parts:

- The controller, which is the unit used for control of the hydro installation.
- The servo system, which is an amplifier that carries out water admission changes determined by the controller.
- The pressure oil supply system, which is used to supply oil to the servo system.

The turbine governor's objective is to maintain the turbine generator's constant rotational speed under all situations of grid load and water flow. During load rejections or emergency stops, the turbine water entry must be turned off in line with the permitted limitations of the rotational speed. The pressure in the water conduit increased, as did the unit's ascent. The input reference signal and the speed feedback signal are contrasted. When the load varies, the generator's power output briefly deviates from the load. In reaction to the deviation, the unit inertia masses either accelerate or decelerate.

2.5 Hydropower as electrical commodity

Since there are makers, suppliers, and buyers of electricity, it may be traded like any other product we use on a daily basis. In the economic sense, electricity is a good that can be bought, sold, and exchanged. As a result, just like consumers of all other commodities, clients of electricity will raise their demand until their marginal gain from consuming the power equals their cost of acquisition. Nepal has been trading electricity since few years and will be able to establish as a net exporter in response to the business relationship with India.

Comparing to other products might not be the best approach. The fact that electricity is a real-time good, cannot be stored in large quantities due to high storage costs, and must instead be consumed as it is produced, cannot be separated from its transportation means—transmission and distribution lines, which are primarily owned by utility companies—and cannot be separated from its consumer demand curve are some of these distinguishing characteristics. It has been discovered that this inelastic feature applies to both commercial and residential power usage. Additionally, it cannot be in a queue of customers waiting for it during regular business hours, unlike other items. Even if electricity prices only marginally rise, the manufacturing sectors that rely on it for their production process won't cut off the supply.

Residential users, like business users, won't weigh cost vs benefit when turning on the lights in their residences or places of business as a result of the rise in power rates [10–13]. Power and energy are the two main types of commodities found in an electrical market.

3. Description of the Project

3.1 Introduction of the Project

Indrawati III Hydropower Station is run-of-river hydro-electric plant. It is the first hydro project developed by Private sector, National Hydropower Company Limited. The flow from Indrawati River, a tributary of Sunkoshi River, is used to generate 7.5 MW electricity with annual energy of 50 GWh. The design flow is 14 m3/s and design gross head is 65m. The plant is owned and developed by National Hydropower Company Limited, am IPP of Nepal. The plant started generating electricity since 2059-01-21 BS. The generation license will expire in 2104-09-29 BS, after which the plant will be handed over to the government. The power station is connected to the national grid and the electricity is sold to Nepal Electricity Authority.

3.2 Location of the Project Area

Indrawati III Hydropower Station lies on the northern part on Sindhupalchowk district and is located on Indrawati River. The catchment area of Indrawati river at the outlet point of sunkoshi river is about 1240 km^2

Figure 7: Location of Indrawati-III Hydropower Project

3.3 Accessibility of site

After going about 70 km towards east from Kathmandu in Arniko Highway, we reach a location called Melamchi. After that, by following a road of about 15km towards the north, we reach our site area.

3.4 Topography and Basin Physiography

The topography of the site area is somewhat diversified with hills on northern side. The Indrawati Khola originates from water falling on it and seepage from mountainous regions. The total catchment area at intake is 431.34 km^2 , calculated using ArcGIS.

3.5 Climate Characteristics

It has somewhat diversified climate characteristics. In wet season, the discharge occurs due to rainwater on its catchment whereas in dry season, the flow is due to the seepage from natural springs and snowmelt from mountainous regions.

3.6 Construction Material

Local construction materials like stone and aggregates are found locally whereas other industrial materials like cement are not available locally and should be important from city areas. Stone and aggregates can be found in large amount.

4. Hydrology and Sediment

4.1 General

A hydropower project's planning, construction, and effective operation are greatly influenced by the hydrological characteristics of a river. The hydrology of a river affects the kind and scale of hydroelectric plants, the design of their components, and their capacity to operate in a safe and cost-effective manner. While designing a hydropower project it should be noted that hydraulic structures are well affected by the sediments carried out by the rivers. So, these affecting parameters are to be well measured and mitigated properly.

It is located at the Indrawati Khola. The hydrological studies at proposed intake and powerhouse sites are carried out based on available regional stream flows and precipitation records near by the Project. The study includes hydro-meteorological characteristics of the watershed regional database for hydrological analysis, long term monthly flow and flow duration at intake, low flow analysis at intake, flood hydrology and hydraulics intake and powerhouse sites Indrawati Khola.

4.2 Objectives

The study's main goal is to review all previous research, update it whenever possible with new data, employ both stochastic and deterministic methods when the data allows, and suggest hydrologic design parameters like low flows and mean monthly flows for energy calculations and capacity optimizations as well as design floods at intake and tailrace sites.

4.3 Scope and Methodology

- Delineation of catchment area at intake and powerhouse sites
- Transposition of regional monthly flows to Intake site
- Comparative analysis of monthly flow results obtained by different sources, approaches
- and recommendation of appropriate method for long term design flows
- Development of Flow Duration Curve (FDC) and annual hydrograph from recommended
- long term flows
- Estimation of low flows (Drought analysis)
- Estimation of design floods at intake and powerhouse sites
- Developing rating curves and determining HFL at intake and powerhouse sites

4.4 Review old Past Studies, Reports and Literature

Following relevant past studies, reports and literature on regional hydrology have been collected and reviewed for the study and finalization of key hydrological parameters of the project.

- WECS/DHM, Methodologies for Estimating Hydrological Characteristics of Ungauged Locations in Nepal, 1990
- DHM, Hydrological Estimations in Nepal, 2015
- DHM, Stream Flow Summary, 2015
- DoED, Guidelines for the Design of Head works for Hydropower Projects in Nepal, 2018
- Design guidelines for water conveyance structures, DoED
- Water Resource Engineering Vol. II, S.K Garg, Khana Publishers
- Engineering Hydrology, K Subramanya, Tata McGraw-Hill Publishing Company
- Fundamentals of Hydropower Engineering, Er. Sanjeeb Baral

4.5 Hydro-Meteorological Characteristics and Database

4.5.1 Watershed Characteristics

The Indrawati-3 intake site in Indrawati River basin is located at Latitude of 27°53'6.64"N and Longitude of 85°36'49.68"E. The outlet elevation of Intake is about 928 m (from GIS) and the elevation of headwater (Highest Point) is about 5814 m. The catchment area up to Intake site estimated by GIS is about 431.34 km2 and main channel length is about 40.76 km. The catchment delineated by GIS is presented below in figure below:

Figure 8: Delineated catchment and stream flow lines using Arc-GIS

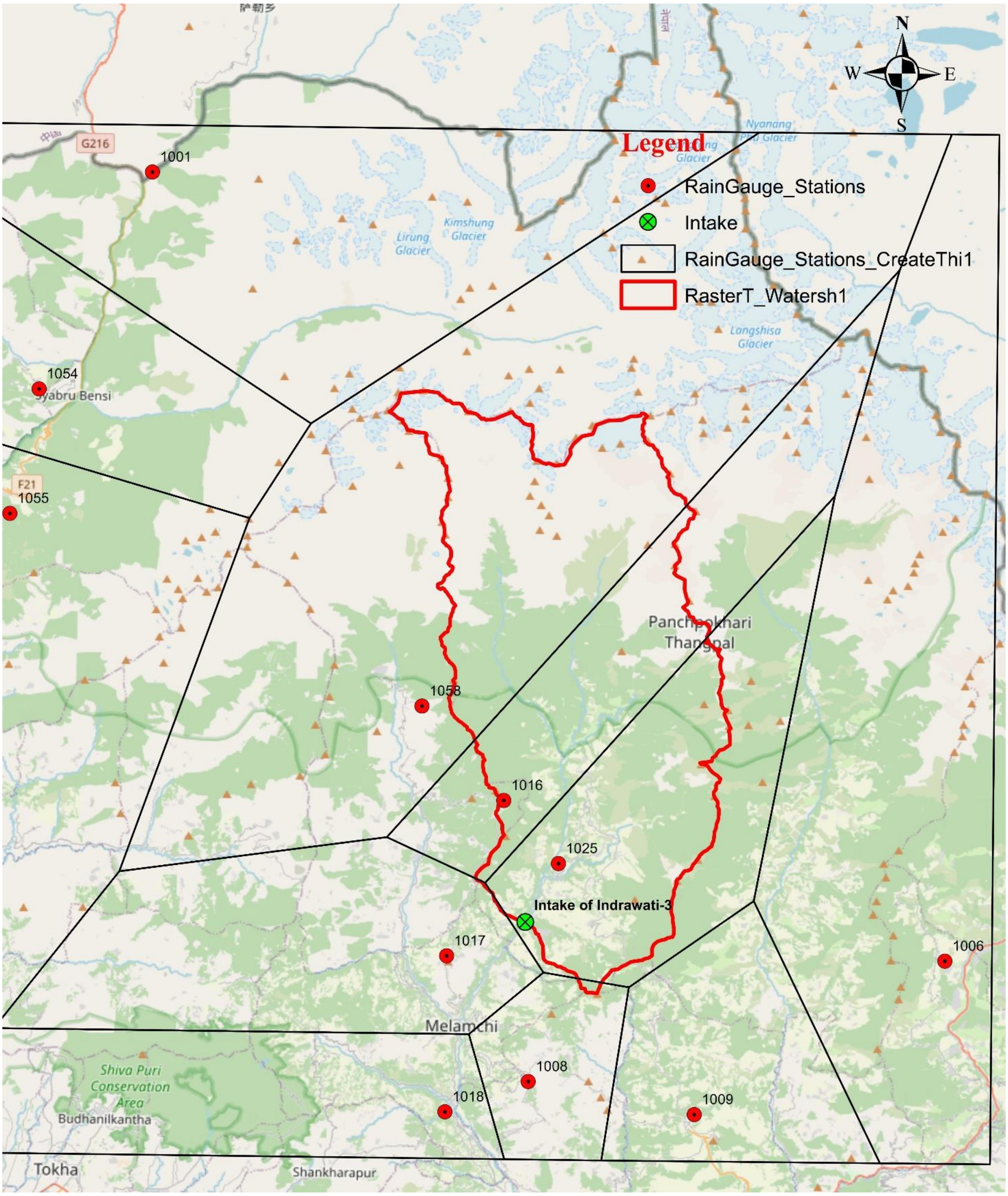


Figure 9: Delineated catchment with Thiessen polygon using Arc-GIS

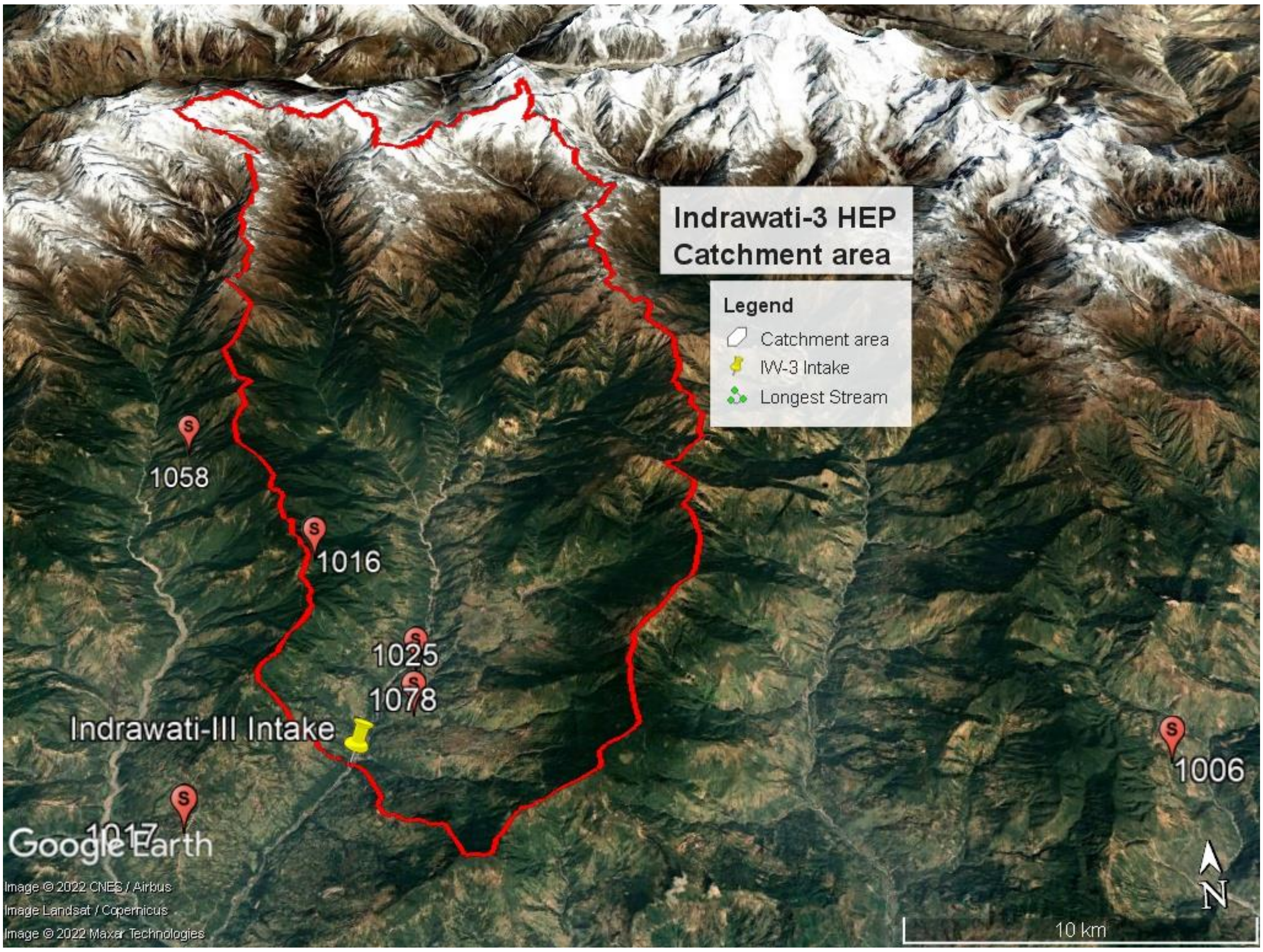


Figure 10: Delineated catchment using Google Earth Pro

Table 2: Hypsometric Data of Catchment abstracted using GIS

Total Catchment Area at intake site	431.34 km2
Total Catchment Area at Powerhouse site	453.36 km2
Catchment area above 5000m elevation	24.32 km2
Catchment area between 3000m and 5000m	188.62 km2
Catchment area below 3000m	218.39 km2
Average Catchment elevation	2631.6 m
Average Catchment Slope(S)	0.1286
Length of Major River(L)	40.76 km
Elevation difference between remotest point and outlet	5814 - 928 = 4886m

4.5.2 Meteorological Database (Precipitation)

No meteorological stations have been found to lie within the catchment of the Indrawati-3 intake site. However, according to the data available on the website of DHM, about 9 meteorological stations have been found to be situated in the periphery of the catchment. Meanwhile, data from few of the stations could not be acquired and data available for some of the stations have been found to be below 25 years. Among the stations with available data, only four stations have been found to have certain weightage in Thiessen Polygon Method using GIS.

Table 3:Meteorological Stations nearby catchment, DHM

Index No.	Name of Stations	Altitude (m)	Latitude (°N)	Longitude (°E)	Annual rainfall(mm)	Data Records
	Intake Indrawati-3	928	27.88	85.61		
1008	Nawalpur	1592	27.80	85.62	2461.4	1982-2011
1016	Sarmathang	2625	27.95	85.6	3737.3	1982-2011
1025	Dhap	1240	27.92	85.83	2791.4	1982-2011
1058	Tarke Ghyang	2480	28.00	85.55	3504.2	1982-2011

4.5.3 Precipitation in Basin

Estimation of basin precipitation was done by Thiessen polygon method. Thiessen polygon is a graphical technique for which calculates station weights based on the relative areas of each measurement station in the Thiessen polygon network. The individual weights are multiplied by the station observation and the values are summed to obtain the basin precipitation. If P_1,P_2, …P_n are the rainfall magnitudes recorded by the stations 1,2,…n respectively and A_1,A_2,…A_n are the respective areas of the Thiessen polygons, then the average rainfall over the catchment P is given by:

$$P = \frac{P_1A_1 + P_2A_2 + \cdots + P_nA_n}{A_1 + A_2 + \cdots + A_n}$$

The Rainfall frequency analysis for 100-year return period is done using two methods: **Logarithmic (Excel) analysis** and **Gumbel Method.**

In both method, four meteorological stations have been used as reference.

Nawalpur Station (1008)
Sarmathang Station (1016)
Dhap Station (1025)
Tarke Ghyang (1058)

Table 4: Weightage factor of Thiessen Polygon using ArcMap 10.5

Stations	Nawalpur	Sarmathang	Dhap	Tarke Ghyang
Thiessen Polygon Weightage Factor	0.0023	0.2500	0.3406	0.4071

A. **Logarithmic (Excel) analysis**

Design Rainfall for 100 years Return Period at Nawalpur Station = 160.968 mm **(Appendix-A, Figure 20)**

Design Rainfall for 100 years Return Period at Sarmathang Station = 236.645 mm **(Appendix-A, Figure 21)**

Design Rainfall for 100 years Return Period at Dhap Station = 270.933 mm **(Appendix-A, Figure 22)**

Design Rainfall for 100 years Return Period at Tarke Ghyang Station = 274.284 mm **(Appendix-A, Figure 23)**

Weighted Mean of Design Rainfall from 4 stations

= 0.4071 × 274.284 + 0.25× 236.645 + 0.3406 × 270.933 + 0.0023 × 160.968

= 263.472 mm

B. **By Gumbel Method**

For N= 30 years, Y_N =0.5362 and S_n=1.1124

Also, for Return period (T=100 years),

$Y_T = -\left(\ln.\ln\frac{T}{T-1}\right) = 4.6 \text{ and } K_T = \frac{Y_T - Y_N}{S_N} = 3.653$

So, $X_T = X_{Mean} + K_T \times \text{Stdev.}\,S$ (**Calculation in Appendix – A, Table – 24**)

Weighted Mean of Design Rainfall from 4 stations = 253.397 mm

(From Appendix-A, Table 24)

4.5.4 Hydrological Database (Stream Flows)

There are a lot of hydrometric stations in different tributaries of Sunkoshi and around the intake site of Indrawati-3. Hence for regional analysis of hydrology it is imperative to use flow data of these stations.

Table 5: Stream flow data selected for regional analysis of Indrawati-III HEP

St. No	River (Location)	Lat.(°N)	Long.(E)	Elevation(m)	Area(km2)	Records
	Intake Indrawati-III	27.89	85.61	928	431.34	
	Powerhouse Indrawati-III	27.86	85.59	865	453.36	
620	Balephi (Jalbire)	27.80	85.77	793	629	1977-2006
447	Trisuli (Betrawati)	27.97	85.18	600	4110	1977-2006
505	Bagmati (Sundarijal)	27.77	85.42	1600	17	1977-2006
630	Sunkoshi (Pachuwarghat)	27.55	85.75	602	4920	1977-2006
647	Tamakoshi (Busti)	27.63	86.06	849	2753	1977-2006

4.5.5 Long term Stream Flow Analysis

The process of long-term streamflow analysis is looking at the water flow patterns in a specific stream or river over an extended period of time, generally decades or even centuries. Understanding how water resources behave in a specific area is crucial for forecasting future water availability and formulating water management plans. Long-term streamflow analysis begins with the collection and compilation of historical streamflow data, which generally include measurements of water flow rates conducted over a number of years at regular intervals. Using this information, streamflow records may be made that demonstrate how changes in water levels and flows have changed through time.

The streamflow data may be examined for trends using statistical methods after the historical data has been gathered and collated. To find trends or patterns in streamflow data, such as seasonal fluctuations or long-term changes across time, researchers may, for instance, utilize time-series analysis.

4.5.6 Mean Monthly Flows at Intake

The term "mean monthly flows" describes the usual, long-term average of the water flow in a stream or river during a specific month. In order to comprehend seasonal trends of water availability and to plan water management strategies, mean monthly flows are a valuable measure. Historical streamflow data is often gathered over a number of years, averaged for each month of the year, and then used to derive mean monthly flows. For instance, the average flow rate for all Januarys in the historical data set may be used to compute the mean monthly flows for January.

The availability of water resources in a certain area may be determined by mean monthly flows, which can offer valuable information. For instance, it could be required to establish water conservation measures or to restrict water use during the summer months if a specific river has low mean monthly flows. On the other side, it could be conceivable to collect and store part of a river's high mean monthly flows during the winter to use during the dry summer months.

It is possible to determine long-term patterns in the availability of water by looking at mean monthly flows. For instance, if a river's mean monthly flows have been dropping over a number of years, it may be important to look into the reasons why and to come up with plans for managing water resources more sustainably.

4.5.6.1 Regional Analysis (Hydrologically Similar Catchment)

There is not any gauging site at Indrawati River downstream from the intake site. So, **Regional analysis** is done to derive stream flow at intake using Hydrologically Similar Catchments (HSC) method.

Five gauging stations having Hydrologically Similar Catchments (HSC) found nearby the **Indrawati-III** catchment are selected for regional analysis. The long term mean monthly and yearly flows at these stations with their respective catchments are presented in **Appendix-A, Table 30**

Transposition of Stream flow from 5 HSC to Intake Site of Indrawati-III HEP

The unit area flows at these stations have been derived by dividing the flows by respective catchment area and presented in **Appendix-A, Table 31**. In the last column of **Table 31**, the average values of unit area flow of all regional stations in each month have been computed and assumed that these averaged unit area flows should represent the unit area flow for Indrawati-III HEP intake.

Table 6: Long term monthly flows (m^3/s) at IW-3 intake derived from UAF of 5 HSC

River (Location)	**UAF of 5 HSC**	**IW-3 Intake**
Area, Km^2	1.000	431.340
Jan	0.0158	6.815
Feb	0.0135	5.823
Mar	0.0129	5.564
Apr	0.0144	6.211
May	0.0220	9.489
Jun	0.0584	25.190
Jul	0.1610	69.446
Aug	0.1942	83.766
Sep	0.1357	58.533
Oct	0.0602	25.967
Nov	0.0294	12.681
Dec	0.0197	8.497

4.5.6.2 HYDEST Method (DHM-2004)

Table 7: Long term monthly flows (m^3/s) at IW-3 intake derived from HYDEST Method

Month	**Mean monthly flow (m^3/s)**
Jan	7.73
Feb	6.53
Mar	4.93
Apr	5.19
May	7.53
Jun	28.02
Jul	68.44
Aug	94.08
Sep	63.6
Oct	30.08
Nov	14
Dec	9.52

4.5.6.3 MHSP NEA 1997

Table 8:Long term monthly flows (m3/s) at IW-3 intake derived from MHSP NEA 1997 Method

Month	Mean monthly flow (m^3/s)
Jan	5.906
Feb	4.890
Mar	4.549
Apr	5.882
May	6.761
Jun	23.190
Jul	69.854
Aug	81.586
Sep	62.851
Oct	28.592
Nov	13.748
Dec	8.909

4.5.7 Comparisons of Mean Monthly Flows

The comparison table for mean monthly flow is shown in **Appendix-A, Table 32**

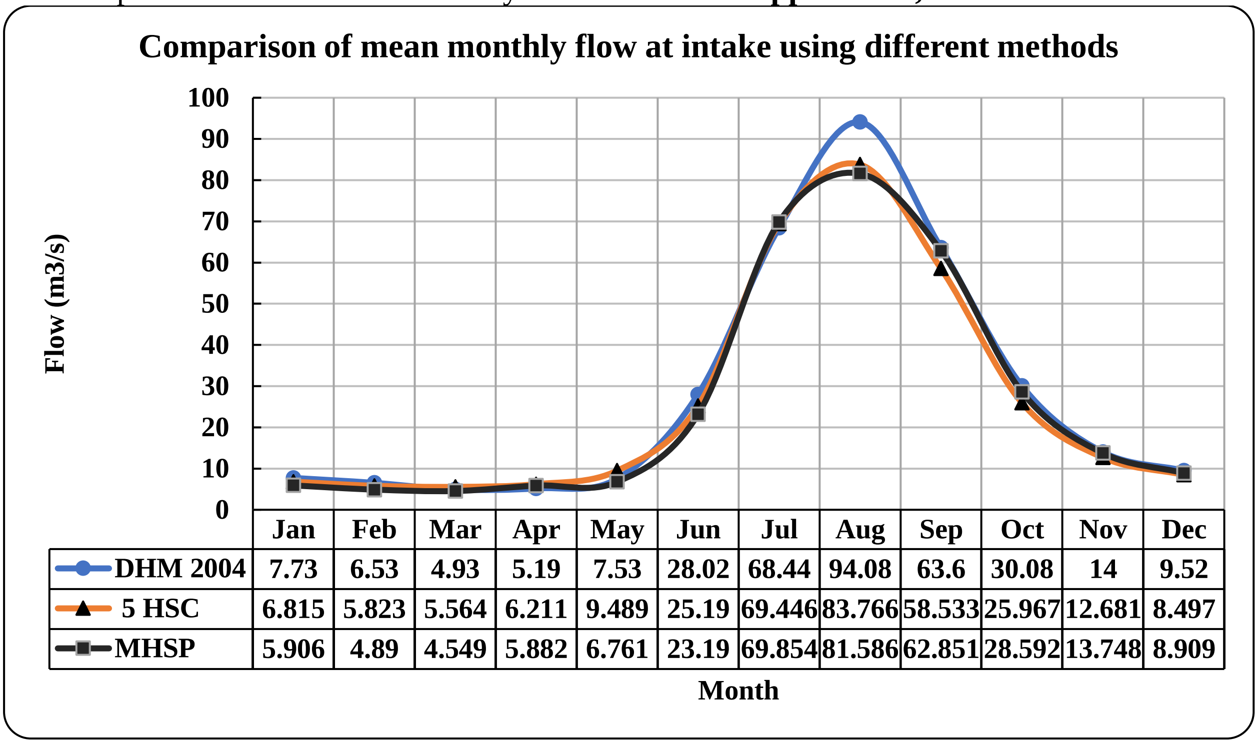
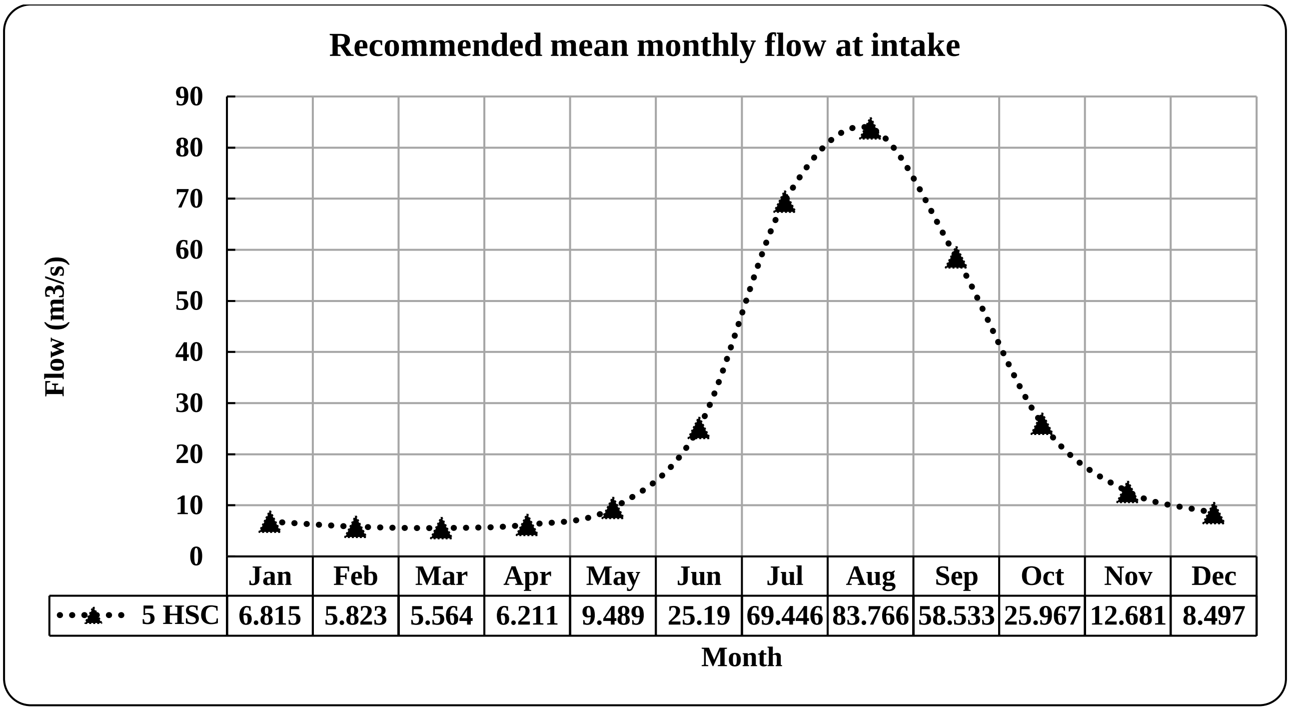


Figure 11: Comparison of Mean monthly flow

4.5.8 Recommended Mean Monthly Flows

Transposed flow of 5 HSC was taken using the catchment area ratio method. The monthly flows are presented in **Appendix-A, Table 33**



	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
5 HSC	6.815	5.823	5.564	6.211	9.489	25.19	69.446	83.766	58.533	25.967	12.681	8.497

Figure 12:Recommended mean monthly flow at intake (m3/s)

4.5.9 Flow Duration Curve

It is simply obtained by plotting the discharge as ordinate and the percentage of time duration for which that magnitude or more is available as abscissa. The design discharge was determined to be **15.76 m^3/s**. The long-term daily flow is shown in **Appendix-A, Table 34**. Flow duration curve is shown below:

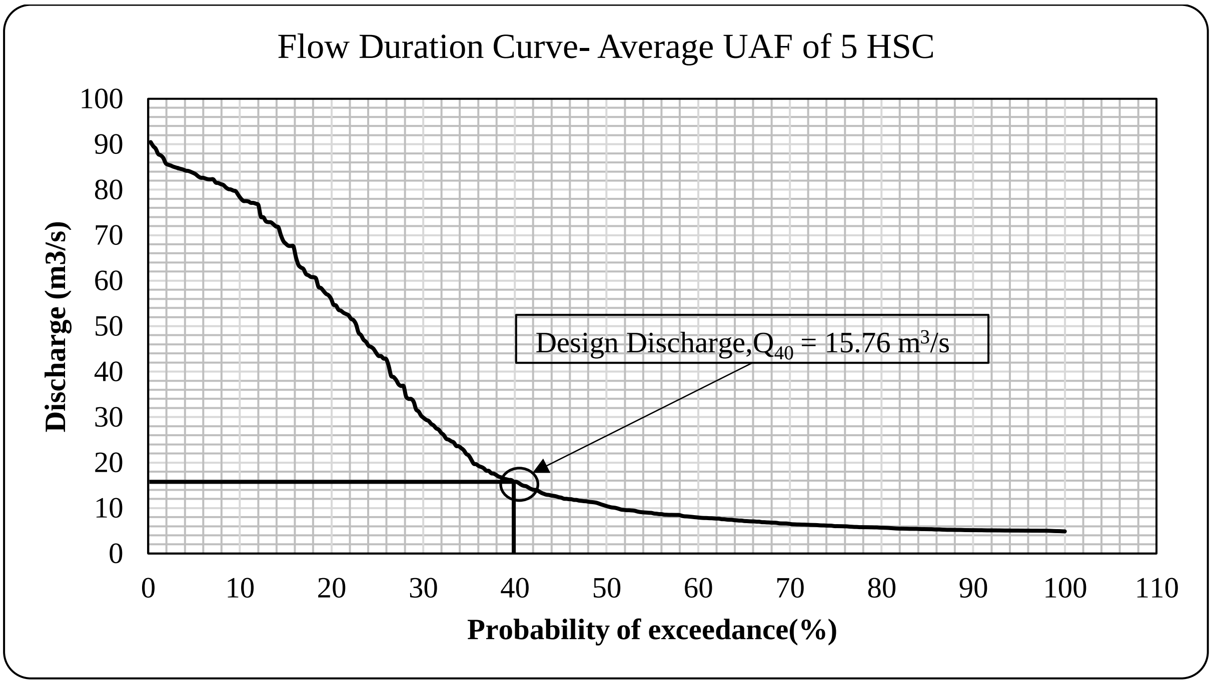


Figure 13:Flow duration curve by regional analysis of 5-HSC

4.5.10 Power duration curve

The available power from a run of river plant could be represented by a power duration curve similar to the flow duration curve. In the existing project of Indrawati-III hydropower, the net available head is 60 m. The firm power generated at the hydropower plant at design discharge of **15.76 m^3/s** would be **8.02 MW**. Power duration curve is shown in below:

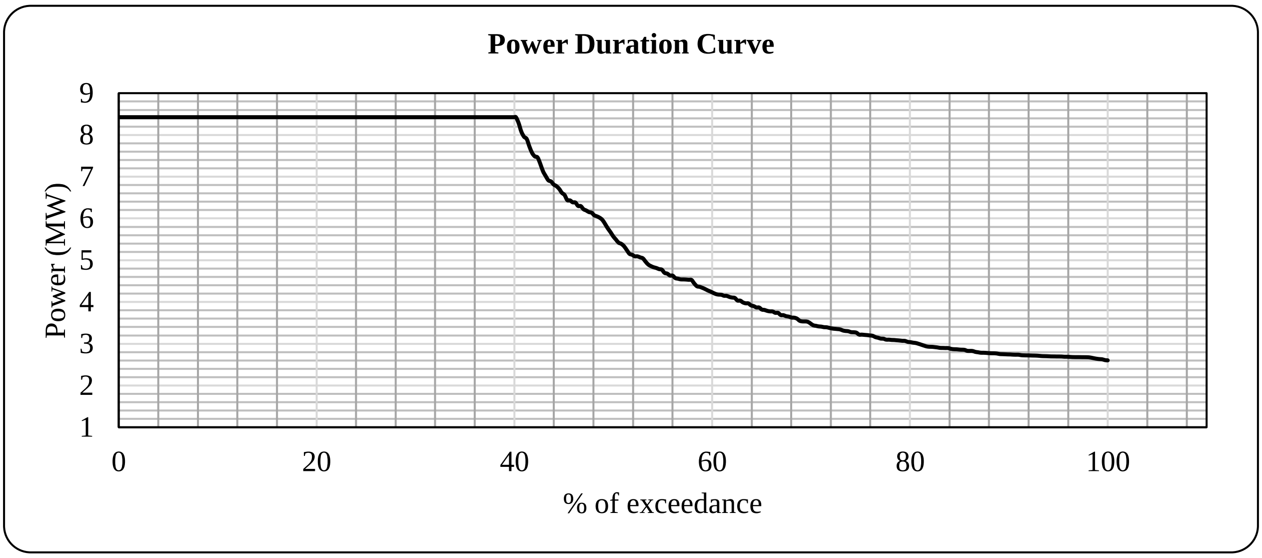


Figure 14: Power duration curve by regional analysis of 5-HSC

4.5.11 Compensation Flows (Environmental release) and Net Available Flows at Intake

The compensation flows or environmental release downstream for Indrawati-3 HEP has been decided according to DoED norms as 10 % of minimum monthly flow. This release will be used by flora and fauna of the area but does not include the present and future demand for irrigation.

4.5.12 Flood Hydrology

4.5.12.1 Estimation of Flood at Intake by Different Methods

4.5.12.1.1 Transposition through Average Unit Area Floods (UAF)

Five stream gauging stations comprising small and medium catchments found nearby the Indrawati catchment are selected for regional flood analysis. The unit area floods at these stations have been derived by dividing the instantaneous floods by respective catchment areas. The average values of unit area floods of all regional stations in each year have been computed and assumed that these averaged unit area floods (annual series) should represent the unit area floods for intake. The annual flood series for intake have been derived by multiplying the annual series of unit area floods by the catchment area of Indrawati at intake. Extreme (Maximum Instantaneous) Floods in 5 HSC are given in **Appendix-A, Table 35**. Unit Area Floods (UAF) at these HSC and annual floods at Indrawati intake estimated using average UAF are shown in **Appendix-A, Table 36**.

From Appendix-A, Table 37 and Figure 24;

Table 9: Estimated floods of different return periods at intake (Logarithmic (Excel) Analysis's 5 HSC)

Return Period (T, years)	**10**	**50**	**100**	**200**
Design Flood(m^3/s)	**381.127**	**545.305**	**616.013**	**686.721**

Table 10: Estimated floods of different return periods at intake (Gumbel Analysis's 5 HSC)

N=30 years			
X_{mean}=	242.67	Y_N=	0.5362
Stdeviation=	88.6416	S_N=	1.1124
Return Period	Y_T	K_T	**X_T**
10	2.250	1.540	**379.265**
50	3.901	3.025	**510.871**
100	4.600	3.653	**566.508**
200	5.295	4.278	**621.942**

4.5.12.1.2 MHSP, NEA (1997) (Regional Approach)

The flood peaks (m3/s) of different return periods are developed using the relation:

$Q_T = k \times (\text{Area below 3000m})^b$

Where k and b are constants which depend on the return periods considered.

k is equal to 7.4008, 13.0848, 17.6058, 21.5181, 39.9035, 69.7807 and b is equal to 0.7862, 0.7535, 0.738, 0.7281, 0.6969, 0.6695 for 5, 20, 50, 100, 1000, 10000 years respectively.

Table 11: Floods obtained at intake by MHSP-1997

Return period (T, Years)	10	50	100	200	**Cat. Area, Km^2**
Flood, m3/s	334.396	610.270	718.260	944.930	**431.34**

4.5.12.1.3 Dickens Modified (Irrigation Research Institute, Roorkee, India)

Irrigation Research Institute, Roorkee, has conducted frequency studies on Himalayan Rivers and suggested the following relationship to compute Dickens constant C_T for desired return period (T):

$$C_T = 2.342\log(0.6T)\log\frac{1185}{p} + 4$$

$$p = \frac{a+6}{A+a}\times 100$$

Where a = perpetual snow area in sq. km; A = total basin area in sq. km.

Now, T year flood discharge (Q_T) in m^3/sec is determined by:

$$Q_T = C_T \times A^{0.75}$$

Table 12: Flood frequency results by Dickens Modified

T, Years	10	50	100	200	Cat. Area, Km^2
Q_T, m^3/s	614.427	840.752	938.225	1035.698	431.34

4.5.12.1.4 PCJ 1996 method (Intensity based regional)

This method is an outcome of research conducted by **Prof. Dr. Prem Chandra Jha** during his Ph.D. studies at a university in Moscow, Russia. The PCJ method calculates design peak flood discharge based on hourly rainfall intensity. The formula for calculation of maximum rainfall discharge is:

$$\mathbf{Q_P = 16.67 a_p o_p \phi F K_F + Q_S}$$

Where,

Q_p= Maximum rainfall design discharge for required exceedance probability (p) in m^3/sec

a_p= Maximum rainfall design intensity for required exceedance probability (p) in mm/min

$a_p = a_{hr}k_t$, where, a_{hr} = Hourly rainfall intensity for required exceedance probability (p) in mm/min at selected rainfall stations

k_t = Reduction coefficient of hourly rainfall intensity (depends on the size of catchment area)

o_p = Infiltration coefficient of the basin derived as the function of exceedance probability (p)

Φ = Areal reduction coefficient of maximum rainfall discharge (depends on the size of catchment)

F = Catchment area of the drainage basin in sq. km.

K_F = Coefficient for unequal distribution of rainfall in different sizes of the basin, captured by one rain.

Q_S = Discharge by melting of snow can be taken as 0 to 10% of Q_P in the absence of data.

Table: Flood Result from PCJ 1996 (From 3 Rainfall stations:- 1008,1009,1035)

T, Years	10	50	100	200	Cat. Area, Km^2
Q_T, m^3/s	358.954	732.789	871.462	929.676	431.34

4.5.12.1.5 Rational method

This method is widely used for small catchments (up to 50 km^2) where the time of concentration is small and required flood frequency is less (up to 50 years). Due to this limitation, it has not been used in the case of Indrawati-III intake flood analysis

4.5.12.1.6 WECS/DHM 1990 (HYDEST)

In Nepalese context, Water and Energy Commission Secretariat (WECS)/Department of Hydrology and Meteorology (DHM) developed empirical relationships for analyzing flood of different frequencies. It is the modification of WECS approach of 1982.

The formula for 2-year return period is given by,

$$Q_2 = 1.8767(A_{3000} + 1)^{0.8737}$$

The formula for 100-year return period is given by,

$$Q_{100} = 14.63(A_{3000} + 1)^{0.7342}$$

Where,

Q is design flood in m3/s

A3000 is basin area (in km2) below 3000 m elevation.

For other return period,

$$Q_T = e^{\ln Q_2} + S\sigma$$

Where,

S= Standard Normal Variate

$$\sigma = \frac{\ln(Q_{100}/Q_2)}{2.32}$$

Table 13: Flood frequency results by WECS/DHM -1990 Method

Return Period (T, Years)	10	50	100	200	**Cat. Area, Km^2**
Flood Discharge (m^3/s)	431.811	659.668	765.889	878.540	**431.34**

4.5.12.1.7 DHM-2004

The DHM (2004) method is an update to the WESCS/DHM 1990 method.

The formula for 2-year return period is given by,

$$Q_2 = 2.29(A_{3000})^{0.86}$$

The formula for 100-year return period is given by,

$$Q_{100} = 20.7(A_{3000})^{0.72}$$

Where,

Q is design flood in m3/s

A3000 is basin area (in km2) below 3000 m elevation.

For other return period,

$$Q_T = e^{\ln Q_2} + S\sigma$$

Where,

s = Standard Normal Variate

$$\sigma = \frac{\ln(Q_{100}/Q_2)}{2.32}$$

Table 14: Flood frequency results by WECS/DHM -2004 Method

Return Period (T, Years)	10	50	100	200	**Cat. Area, Km^2**
Flood Discharge (m^3/s)	522	845	1001	1169	**431.34**

4.5.12.1.8 Comparisons of Flood by Different Methods

Catchment Area= 431.34 km^2

Table 15: Comparison of floods (m^3/s) at intake by different methods

Methods	**Return Period in Years**			
	10	**50**	**100**	**200**
DHM 2004, (m^3/s)	522.000	845.000	**1001.000**	**1169.000**
Modified Dicken's , (m^3/s)	614.427	840.752	**938.225**	**1035.698**
PCJ 1996, (m^3/s)	358.954	732.789	**871.462**	**929.676**
WECS 1990, (m^3/s)	431.811	659.668	**765.889**	**878.540**
MHSP 1997, (m^3/s)	334.396	610.270	**718.260**	**944.930**
Logarithmic (Excel) Analysis, (m^3/s)	381.127	545.305	**616.013**	686.721
Gumbel's-5 HSC, (m^3/s)	379.265	510.871	**566.508**	621.942
Average value=			**782.48**	**991.57**

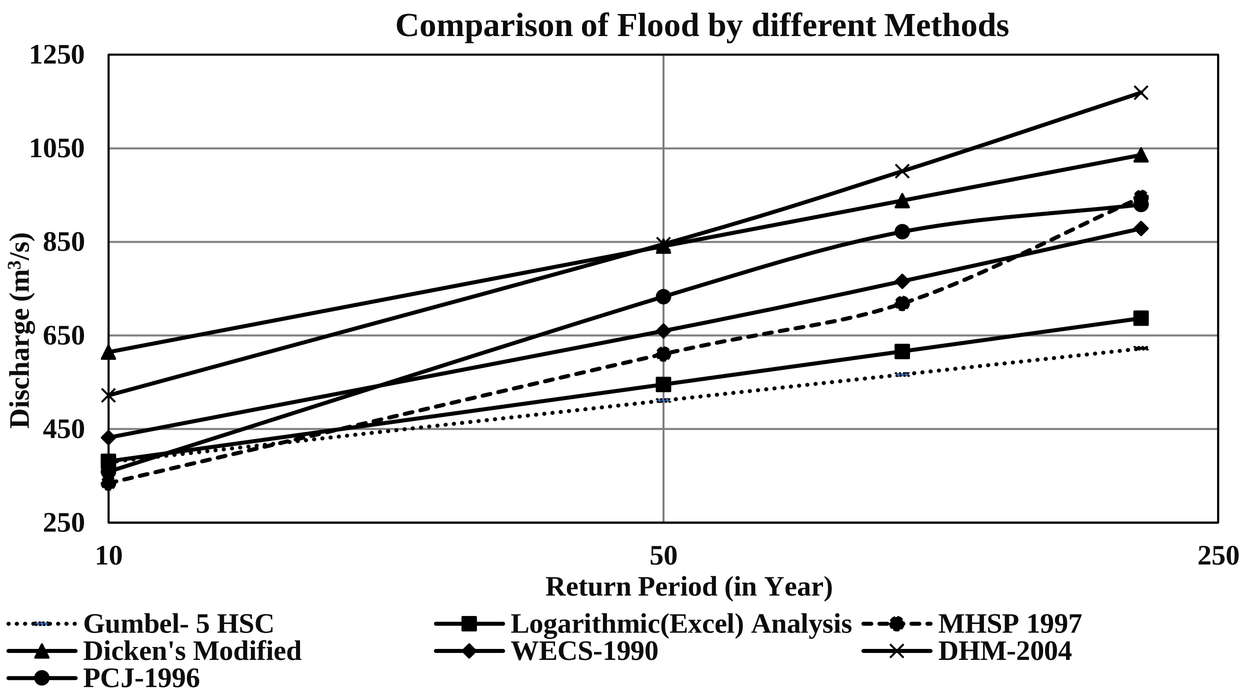


Figure 15:Comparison of floods at intake obtained by different method

4.5.12.1.9 Recommended Design Flood

Taking average of nearest values (which is in the variation of 20% with each other) as shown in comparison chart, we get the average value of **782.48 m^3/s for 100-year return period flood.** Average value is found near to the **WECS 1990 and PCJ 1996**. So, we **adopted PCJ 1996** method for design flood for extra factor of safety for design of headworks structures and components of Powerhouse.

$$\mathbf{Q_{100} = 871.46\ m^3/s}$$

$$\mathbf{Q_{200} = 929.67\ m^3/s}$$

4.5.12.1.10 Design Flood at Powerhouse

Floods of different return periods at Powerhouse of Indrawati-III HEP are directly estimated by applying catchment area ratio,

Catchment area at powerhouse= **453.36 km^2**

Catchment area at Intake Site = **453.36 km^2**

$$Q_{100} = \frac{871.46}{431.34} \times 453.36 = 915.95\ m^3/s$$

$$Q_{200} = \frac{929.67}{431.34} \times 453.36 = 977.13\ m^3/s$$

4.6 Sediment Yield

The sediment load for a hydropower project refers to the amount of sediment, such as sand, silt, and clay, that is carried by a river or stream and that can accumulate in the reservoir of the hydropower dam. Sedimentation can reduce the capacity of the reservoir and the efficiency of the power generation system, and can also cause environmental problems downstream of the dam.

To minimize the impact of sedimentation on a hydropower project, engineers and planners need to estimate the sediment load of the river or stream where the project is located. This involves analyzing data on the flow rate, erosion, and sediment transport of the river, as well as the geological and climatic conditions in the area.

DHM is not collecting sediment data for Indrawati River or any other hydrologically similar river. We couldn’t collect data ourselves and we didn’t get any data for further calculations **so sediment analysis is not covered in this report**. We adopted sediment data of similar project for our project study.

5. Project Optimization Studies

5.1 General

The optimization study of a project component is carried out to analyze available opportunities at an optimal cost to arrive at the best possible option. The optimal diameters of the main elements of the water conveyance system/ waterways are determined using an in-house optimization spreadsheet program. The detail of optimization is discussion on respective subchapters below.

5.2 Headrace Tunnel (HRT) Optimization

In general, the optimal diameter of the headrace tunnel is found out considering the cost of the tunnel of different diameters as well as corresponding revenue loss due to the head loss. The diameter of the tunnel having total minimum combined cost (tunnel cost and revenue loss) was considered to be the optimum diameter. The tunnel is assumed to get constructed by conventional drilling and blasting methods. Limiting velocity of flow in HRT below 2 m/s, for design discharge of 15.76 m3/s, the minimum tunnel diameter of 3.3 m (Inverted D-shaped) is required. For different diameters the headrace tunnel optimization is shown in Appendix B. Their respective cost calculation and cost of energy loss are shown in Appendix B.

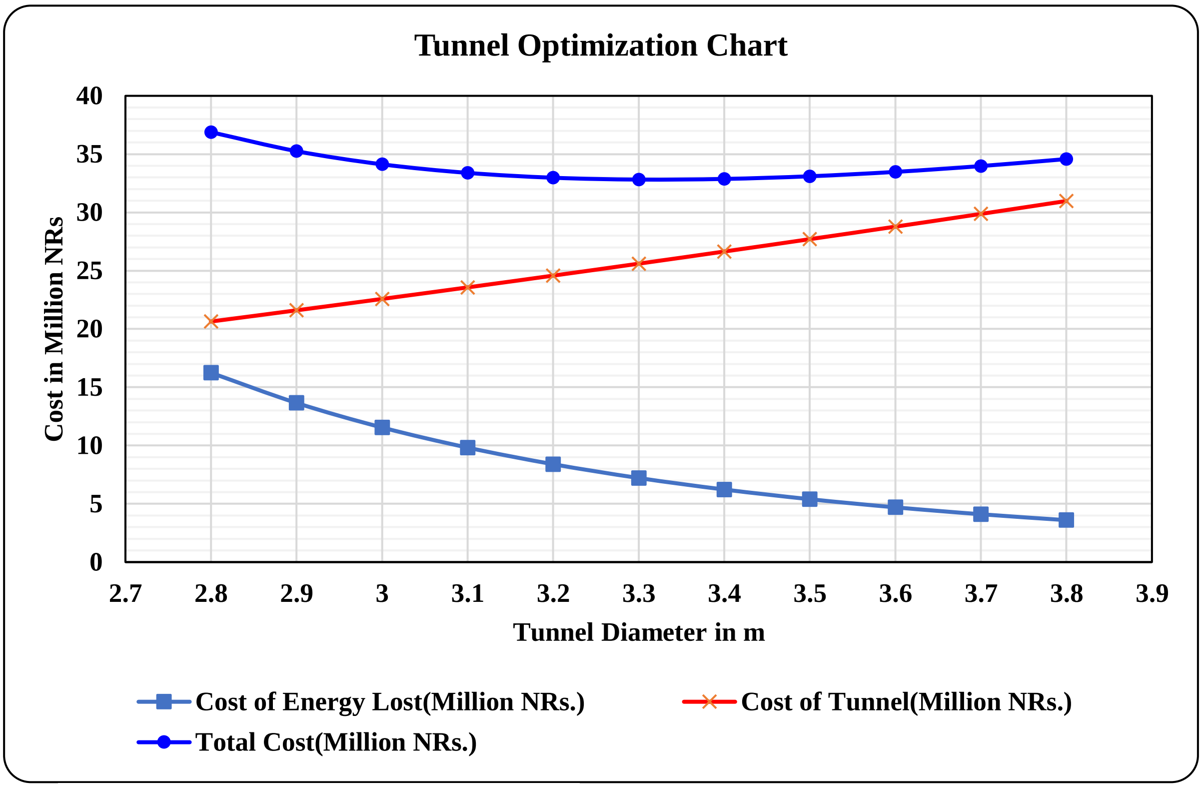
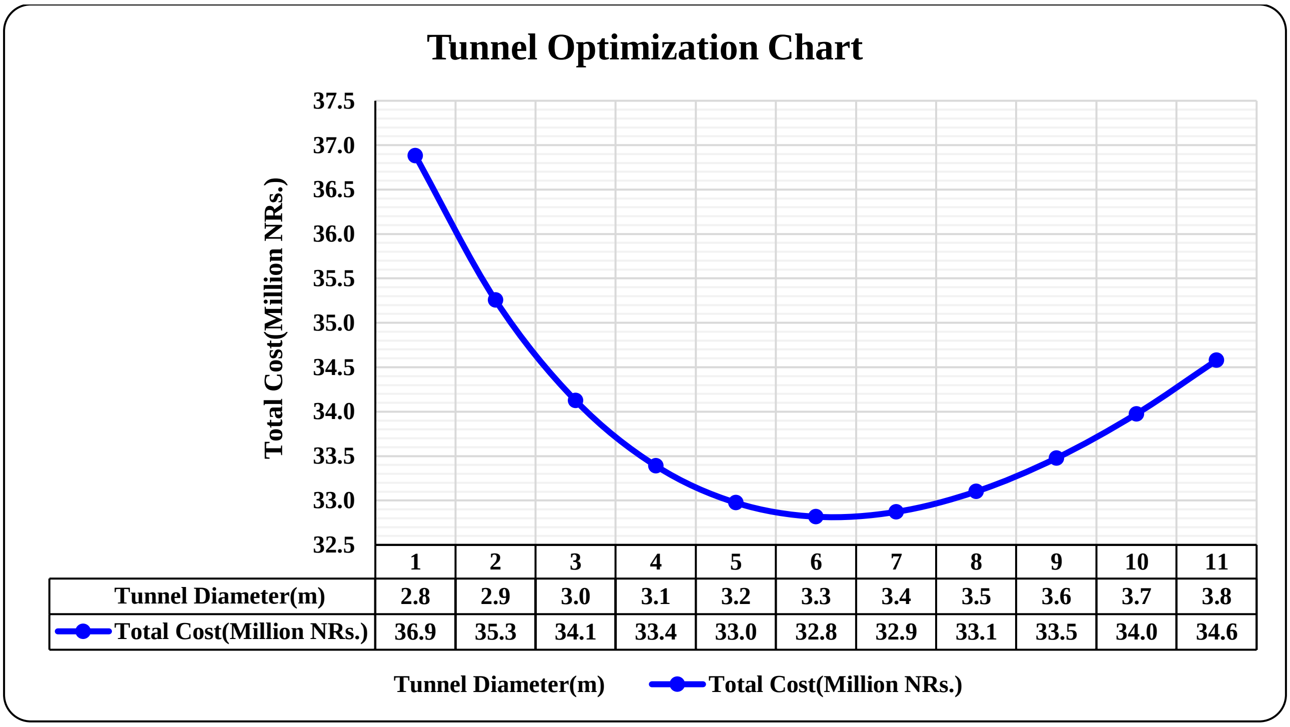


Figure 16: Tunnel Optimization Chart



	1	2	3	4	5	6	7	8	9	10	11
Tunnel Diameter(m)	2.8	2.9	3.0	3.1	3.2	3.3	3.4	3.5	3.6	3.7	3.8
Total Cost(Million NRs.)	36.9	35.3	34.1	33.4	33.0	32.8	32.9	33.1	33.5	34.0	34.6

Figure 17: Tunnel Optimization Chart (enlarged view of Total Cost)

5.3 Penstock Pipe Optimization

A similar approach has been made to optimize penstock pipe diameter considering different pipe diameter and corresponding revenue loss due to head loss. Steel lining has been considered throughout the length of the pressure shaft. The project has discharge of 15.76 m^3/s due to which three turbines are operated with 5.25 m^3/s on each. Twelve different diameters of the penstock pipe ranging from 1.5 m to 2.6 m with an increment as 0.1 m were considered for optimization. However, for optimization, the cost of steel is estimated based on current purchasing cost, transportation cost, fabrication and installation cost including site arrangement. The analysis based on the mentioned assumptions resulted in an optimum diameter of 2.0 m. For different diameters the penstock optimization is shown in Appendix B. Their respective cost calculation and cost of energy loss are shown in Appendix B.

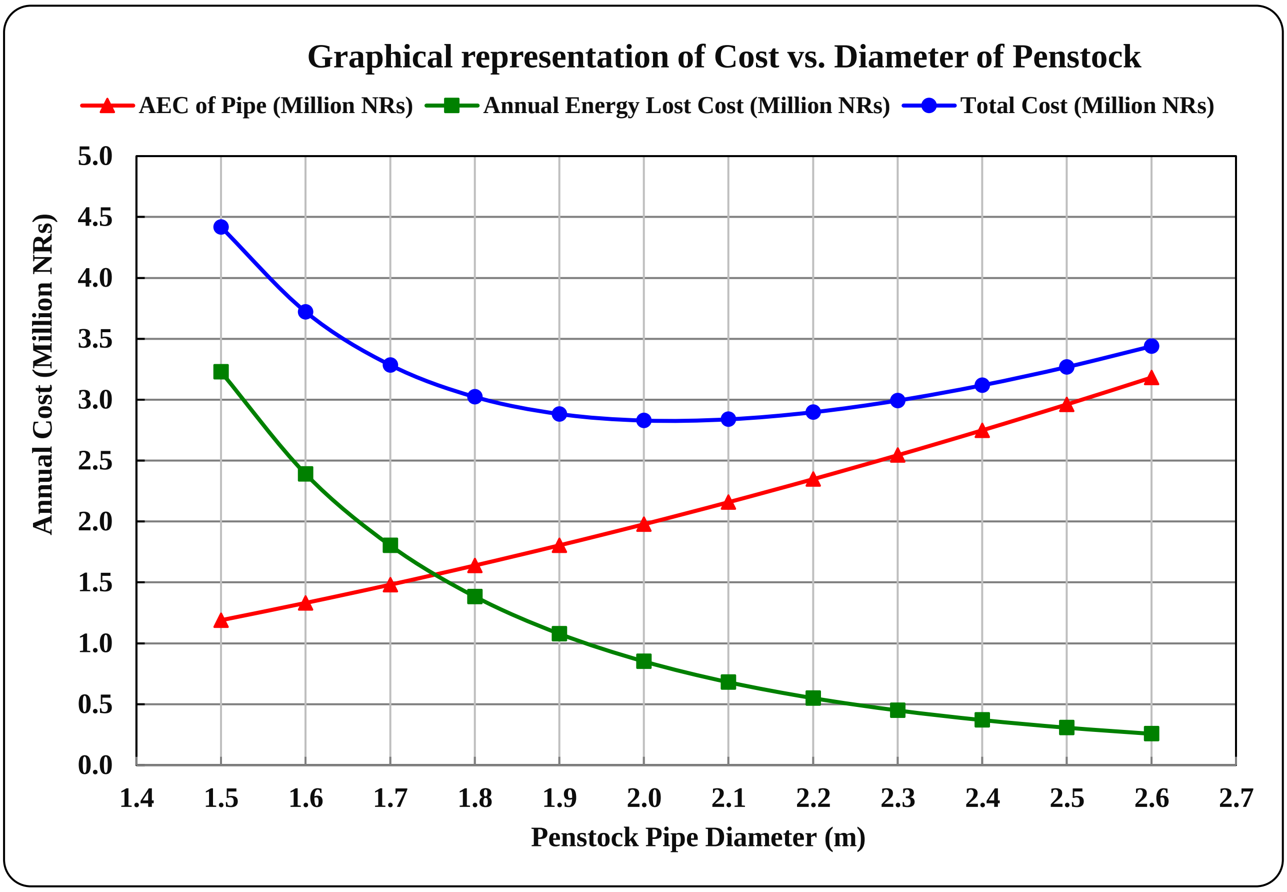
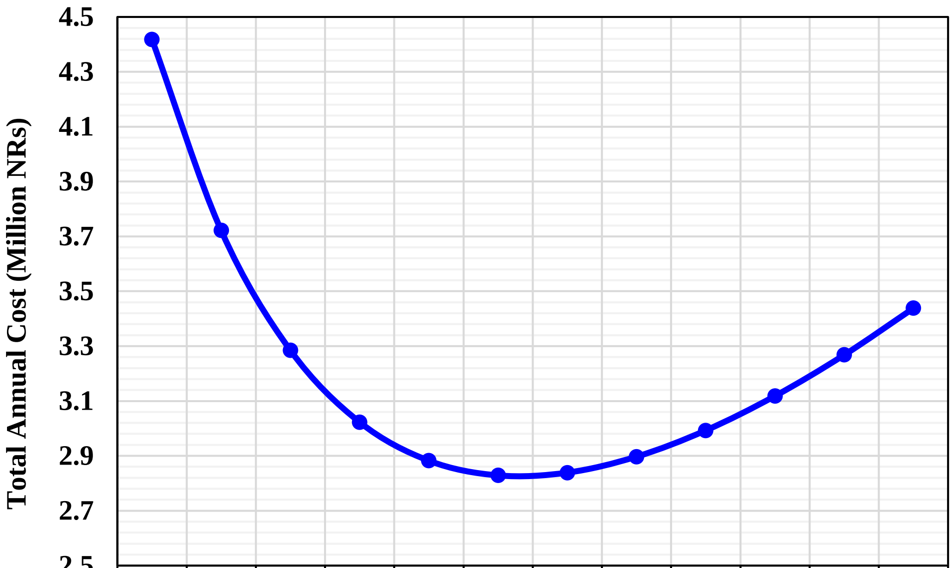


Figure 18: Penstock Optimization Chart



	1	2	3	4	5	6	7	8	9	10	11	12
Penstock Diameter (m)	**1.5**	**1.6**	**1.7**	**1.8**	**1.9**	**2.0**	**2.1**	**2.2**	**2.3**	**2.4**	**2.5**	**2.6**
Total Annual Cost (Million NRs)	**4.4**	**3.7**	**3.3**	**3.0**	**2.9**	**2.8**	**2.8**	**2.9**	**3.0**	**3.1**	**3.3**	**3.4**

Penstock Diameter (m) **Total Annual Cost (Million NRs)**

Figure 19:Penstock Optimization Chart (enlarged view of Total Cost)

6. Project Design

6.1 Introduction of Components

6.1.1 Headworks

Any construction at the head or divergence point of a canal is referred to as a headwork in civil engineering. It is used to redirect water from a river into a canal or from a major canal into a smaller canal. It is smaller than a barrage. Weir, intake, under sluice, division wall, flood wall, gravel trap, approach canal, and settling basin are the main parts of a headwork.

Functional requirements of headworks:

The headworks of run-of-river hydropower projects shall be planned and designed to ensure safe and regular power generation from the hydropower plant under normal conditions. For this purpose, the headworks shall fulfill the following functional requirements:

a. Withdrawal of desired quantity of water from the river for power generation.
b. Safe passage of flood flows.
c. Passage of trash, floating debris and ice.
d. Passage of sediments.
e. Bed control at the intake.
f. Exclusion of suspended sediments.
g. Flushing of settled sediments.

Selection of headworks site

Based on how well suited it is to the main elements that make up the headworks concept, the headworks location must be chosen. Alternative headworks layouts must be created for this purpose and compared based on careful consideration of technical, economic, and environmental factors. The site with the most environmentally friendly headworks layout that is also technically and economically feasible will be chosen.

The hydropower components for RoR and PROR projects are

- ✓ Diversion weir
- ✓ Undersluice
- ✓ Intake
- ✓ Gravel trap
- ✓ Approach canal
- ✓ Settling basin
- ✓ Conveyance system (Canal, Pipe or Tunnel)
- ✓ Surge tank / Forebay

- ✓ Penstock pipe
- ✓ Anchor blocks and support piers
- ✓ Powerhouse
- ✓ Tailrace

6.1.2 Weir

A weir is a structure used to divert water that is often built over a river outflow to provide enough water for the intake. There are several varieties of weirs, and each type's use is influenced by factors such as geography, geology, discharge, river morphology, etc. Weirs are created when a high crest creates the majority of the water ponding and shutters only create a little or nonexistent portion of it. It is referred to as a barrage or river regulator if gates perform the majority of the ponding and a minor or nonexistent portion of it is performed by the higher crest.

6.1.3 Trash Racks

To stop debris, ice, etc. from entering the conduit, the intakes and dam outputs are often covered with garbage racks. Based on the largest size of the trash that must be prevented from entering the conduit, these racks are nothing more than steel bars spaced 5 to 15 cm apart (in both directions). To reduce losses, the flow through the rack is maintained at a low velocity (often less than 0.62 m/s). When necessary, physical work is used to remove the floating debris, ice, etc. that the racks halt and gather on them.

6.1.4 Intake Structures

The building known as an intake is used to draw the necessary volume of water from a river or reservoir for a variety of engineering purposes, including irrigation, power generating, and water supply, among others. It is a structure designed to direct water into a conduit that leads to the power plant. A water intake structure should guarantee excellent quality water in enough quantities and control over the water supply. Weir and intake structure preparations must be made in order to evacuate the required volume of water to the channel at any time. It is necessary to securely and damage-free evacuate the peak discharge. To stop rubbish from entering the conveyance canal and potential damage to it, garbage racks should be installed at the intake.

Selection of type of intake

The most suitable type of intake for a particular site shall be selected considering the following factors:

- ✓ Nature of river.
- ✓ Nature and scale of hydropower development.
- ✓ Sediment, trash, and debris content.
- ✓ Construction considerations.
- ✓ Operation and maintenance considerations.

The type of intake selected based on the above considerations should generally be verified through model studies.

6.1.5 Settling basin

The settling basin is a structure used to filter suspended particles out of the water used for power plant transportation. Settling basins must be built to prevent sediments from entering the water conveyance system, which might abrasively harm the turbine runners and penstock. This will be accomplished by lowering the water flow's turbulence level to enable suspended sediment particles to settle out of the water body and deposit on the basin's floor.

The design shall consist of the following activities:

a. General arrangement of the settling basin, its flushing structures and its inlet and outlet transitions.

b. Hydraulic design of the settling basin, its flushing structures and its inlet and outlet transitions.

Selection of type of settling basin

The choice between settling basins with periodic or continuous flushing shall be made based on the following factors:

a. Topography.

b. Availability of water.

c. Type and size of power plant.

d. Cost of construction.

e. Ease of operation and maintenance.

f. Power outage or reduction.

6.1.6 Headrace Tunnel

Headrace tunnels (HRT) are defined differently depending on the situation. Depending on the project and site requirements, "headrace tunnel takes water from connecting channels and conveys it to the fore bay or directly to the penstock provided with surge shaft." Headrace tunnels are occasionally also referred to as power tunnels. When the terrain is extremely steep, it is typically preferable to the canal system.

6.1.7 Surge Tank

Surge tank is located between the headrace pressure conduit and the steeply sloping penstock pipe and is designed either as a chamber excavated in the mountain or as a tower raising high above the surrounding terrain.

The important functions of surge tank are as follows:

- ✓ Upon the rapid closure of the turbine in case of load rejection, protects the conduit system from high internal pressures.
- ✓ The surge tank provides protection to the penstock against the detrimental effects of water hammer if no bypass valve is installed or if the bypass valve fails to operate.
- ✓ When the load decreases, the water moves backwards and gets stored in it.
- ✓ When the load increases, additional supply of water will be provided by surge tank.

Location of Surge Tanks

- ✓ Surge tanks are located near to the powerhouse to reduce length of penstocks.
- ✓ Location at which flat sloped conduit and steep sloped penstock meets.

6.1.8 Penstock

A penstock is a sluice, gate, intake structure, or enclosed conduit that regulates water flow and supplies water to hydro turbines and sewage systems. The word is a holdover from previous watermill and mill pond technologies. Penstocks for hydroelectric plants often have a surge tank and a gate mechanism. Depending on the application, they may consist of a variety of parts, including anchor blocks, drain valves, air bleed valves, and support piers. Flow is controlled by the functioning of the turbines, and it is zero when the turbines are not in use. Penstocks need to be maintained by hot water washing, hand cleaning, antifouling coatings, and desiccation, especially when utilized in dirty water systems.

6.1.9 Powerhouse

The powerhouse is a part of the hydroelectric system that contains the turbines, generators, draft tubes, and penstocks that are required for producing hydropower. Its primary goal is to adequately store all the equipment while also providing some degree of visual attractiveness. It may be roughly divided into two categories.

Surface Power Station

These are the power stations where turbine is located on the surface or on the ground level.

Underground Power Station

These are the power stations where turbine is located below the surface or below the ground level.

Mainly, three divisions of powerhouse can be explained. They are:

Sub-structure: It is that part of powerhouse which is situated below the turbine level. It includes draft tube, tail water channel, natural drainage pipes of wastewater, drainage galleries, etc. It transmits the load of the structures it to foundation strata and is usually a massive concrete structure.

Indeterminate structure: It extends from the top of the draft tube to the top of the generator foundation. It includes scroll casing, galleries for auxiliary machines and the governor survo- motor systems. The turbine floor is below the generator floor and is accessible through stairs from the generator floor.

Super-structure: It is the portion extending from the generator floor, called the main floor, up to the roof top. It includes generators and governors, control room, the exits and the auxiliary equipment such as needed for ventilation and cooling. It also consists of walls and the roof with a main travelling gantry crane at the roof level.

6.1.10 Tailrace

The tail race, containing tail water, is a channel that carries water away from a hydroelectric plant or water wheel. The water in this channel has already been used to rotate turbine blades or the water wheel itself. This water has served its purpose and leaves the power generation unit or water wheel area.

In hydroelectric dams, the tail race is at a much lower level than the height of the reservoir behind the dam. This difference in height corresponds to the amount of hydropower that can be obtained from the water, and the height difference is known as the hydraulic head. This change in height corresponds to a change in gravitational potential energy. Some of the gravitational potential energy from the water above the dam was used to spin the turbines and generate electricity. Water flowing from a hydroelectric plant in the tail race eventually joins the natural flow of water.

6.2 Basis of Design

The project components are designed based on the following:

- ✓ The design discharge of the existing Indrawati-III HEP is 14 m^3/s and its installed capacity is 7.5 MW.
- ✓ The weir height is about 5 meters to accommodate sufficient pondage during dry months.
- ✓ Settling Basin for the Indrawati-III is designed to settle particle size greater than 0.3 mm. The settling basin is continuous flushing type of length 88 m with flush desander.
- ✓ The thickness and diameter of penstock pipe will be determined by considering the pipe strength and available water head and water hammer considerations.
- ✓ A surge tank is designed based on Thoma criteria considering water hammer effects.
- ✓ The powerhouse is designed to accommodate three units of generating machines and auxiliary equipment's. Furthermore, spaces are provided for the repair and maintenance of the power plant.
- ✓ The powerhouse is constructed above the surface.

6.3 General arrangement of Project Components

The 7.5 MW installed capacity Indrawati-III Hydropower Project is a run-of-river type hydropower scheme which utilizes water from the Indrawati-III River. The design discharge of the project is 15.76 m^3/s. Similarly, headworks are designed for 100 years of flood event discharges of 871.46 m^3/s.

The Crest level of weir of this hydropower project is at an elevation of 923 m amsl. The total adopted length of the weir is 40 m.

The intake invert level is proposed at 920 m amsl with minimum submergence of 2.2 m. There will be three intake openings. The intake opening will be 2 m (H) by 2 m (W). The approach velocity at the intake is designed to be 1.169 m^3/s. These designs are carried out according to DoED guidelines.

The length of the reinforced concrete settling basin in the Indrawati-III is 88 m. The settling basin is designed to settle particle size of up to 0.3 mm. This is designed based on high sediment concentration of 2000 mg/liter. The sediment deposition depth of 1.45 m is provided. Flushing is carried out continuously.

The headrace pressure tunnel has diameter of 3.3 m. The total length of the HRT is 2.934 km. The whole length of tunnel is shotcrete.

The penstock pipe length from the surge tank to the powerhouse is 300 m. Three penstockpipes feed the design discharge to three Francis turbine units. The average/economic diameter ofpenstock pipe is 2 m, with thickness 7 mm.

6.4 Description of Project Components

The main project components are briefly described below:

6.4.1 Headworks

The headwork structure for the Indrawati-III HEP consists of:

- ✓ Weir
- ✓ Frontal Intake
- ✓ Settling Basin

6.4.1.1 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.

6.4.1.2 Elevation of weir crest

There are numerous 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 **918 m**. The crest level of weir provided is **923 m**.

6.4.1.3 Length of weir

The waterway's width at the intake point determines the weir's length. The average wetted breadth during the flood should be calculated using the crest length. For protection purposes, the upstream and downstream should be thoroughly inspected. Aflux is the term used to describe the rise in water levels upstream of the buildings following the installation of the weir. Aflux fixation is influenced by topographic and geomorphologic variables. The length of the weir is shortened by a large afflux, but the expense of the river training and river protection works is increased. Although it is often limited to 1m for alluvial reaches, it may be higher in hilly areas. High floods must be able to travel through the waterway with the required efflux.

Generally, the waterway is calculated by

Lacey's perimeter Formula: $P=4.75\times\sqrt{Q}$ for alluvial channel.

It may be taken just as 60 % of "P" calculated above for boulder reaches.

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.

6.4.1.4 Major 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:

$$P=0.5\times\gamma_w\times H^2\times b$$

Where, γ_w=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 seepage from the bottom junction between the weir and its foundation, via the weir body itself, and through foundation material holes, fissures, and pores causes an uplift pressure on the weir base. The uplift pressure effectively lessens the weir's downward weight, which works against the stability of the dam. The Khosla Theory is used to analyze seepage. The Laplacian equation has a mathematical solution known as Khosla's Theory, which is a quick and reliable approach for seepage investigation. According to the USBR, a straight line connecting the uplift pressure intensity at the heel and toe should be considered as equal to each location's hydrostatic pressure.

Weight of weir

The weight of weir and its foundation is the major stabilizing/ resisting force. While calculating the weight, the cross section is splitted 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.

6.4.1.5 Stability Analysis of Weir

The stability of dam is checked for following conditions:

a) Sliding of Weir

$$\text{Sliding Factor} = \mu \times \frac{\text{Net Vertical Forces}}{\text{Net Horizontal Forces}} > 1$$

Where,

μ = Coefficient of friction

The sliding factor should be more than 1 for stability.

b) **Overturning of Dam:**

$$\text{Factor of Safety against overturning} = \frac{\text{Resisting Moment with respect to toe(Mr)}}{\text{Overturning Moment with respect to toe(Mo)}} > 1.5$$

The overturning factor of safety should be more than 1.5 for stability.

c) Tension Failure:

$$\text{Centroid of all forces from toe, } \bar{x} = \frac{\Sigma M}{\Sigma V}$$

Where,

∑M = the summation of moment of forces about toe

∑V = the summation of all vertical forces

$$\text{Eccentricity from the center of base} = \frac{B}{2} - \bar{x}$$

$$\text{Eccentricity (e)} < \frac{B}{6} \text{ (safe in tension failure)}$$

d) Crushing Failure:

The maximum value of compressive value is obtained as:

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

Where,

$\sum V$= Net vertical force

B= Width of weir

e = eccentricity from center of base

The feature of weir and intake is tabulated in Table 6-1.

Table 16: Salient features of diversion works

S. No	Description	Indrawati-III HEP
1.	Length of weir	40 m
2.	Width of weir	2.00 m at Top and 18.25 m at bottom
3.	Number of gates of intake	3
4.	Number of Intake	3
5.	Intake Size	2.0 (B) m X 2.0 (H) m
6.	Normal Operating Level	922.1 m amsl

6.4.1.6 Intake

We selected frontal intake at our dam site since the side area for side intake was not enough. The invert level is 1.0 m above the bed level, thus, the problem of debris and boulders is less and we had enough area along dam axis for intake. Hence, frontal intake seemed appropriate choice.

Discharge at intake:

$$Q = C_d A\sqrt{2gH_l}$$

C_d= coefficient of discharge of the orifice=0.6 for the sharp edge and roughly finished concrete

A= area of the orifice

H_1 = head between weir crest level and canal water level

We have put invert of intake 1 m above the bed level of dam. The design discharge of intake is 1.3*15.76 cumecs (i.e. 20.488 cumecs). The flow velocity at the entrance is 1.169 m/s. The intake is 2m wide with 2m height of each. There are piers of 1m at the edges and at middle. Trash rack with slope of 3V:1H is provided.

The detail drawing of intake is shown in Appendix D and the design calculation in Appendix B.

6.4.1.7 Trash rack design

Trash rack is provided to prevent the coarse boulder, wooden log entering from the headrace.
Head Loss through trash rack is given by,

$$h_t = k \times \left(\frac{t}{a}\right)^{\frac{4}{3}} \times \frac{v^2}{2g} \times \sin\alpha$$

Where,

k =1.67 for round edge bars

a = Clear spacing between rack bars (mm)

t = thickness of the bars (mm)

v = velocity of flow through trash rack (m/s)

α = angle of bar inclination to the horizontal

The trash rack rests at the slope 3V:1H with the flow velocity of 1.169 m/s.

The trash rack is provided with 20 mm diameter bar at the spacing of 100mm.

6.4.1.8 Settling Basin:

The settling basin shall be planned and designed such that power generation is not interrupted, or reduced, during flushing operations.

Fall Velocity Calculation:

For falling velocity (V_f) in laminar flow (Re, Reynolds Number, up to 1),

$$V_f = 418 \times (S - 1) \times d^2 \times \frac{3T + 70}{100}$$

Where,

S = specific gravity

d = diameter in mm

T= temperature in degree centigrade

For transition flow (Re = 1-10000),

$$V_f = \sqrt{\frac{4}{3} \times \frac{g}{C_D} \times (S - 1)D} \ ; \ \text{Where, } C_D = \frac{24}{Re} + \frac{3}{\sqrt{Re}} + 0.34$$

Where,

C_D = Drag Coefficient

Plan Area is obtained as $A = \frac{K \times Q}{\omega}$

Where,

K = turbulence factor

Q = design discharge

w = fall velocity

A = L *B

(L/B should be from 4 to 10)

Depth of settling basin is obtained from the formula given below:

$$\text{Depth (H)} = \frac{Q}{V \times B}$$

Where,

V= flow velocity obtained as

V= a√d (mm) where a = 0.44 for diameter between 0.1 to 1 mm

Sediment depth is obtained for design discharge Q_d m³/sec with Sediment Concentration (C kg/m³) and detention time (T).

$$\text{Silt load (in kg)} = Q_d \times T \times C$$

$$\text{Volume of sediment} = \frac{\text{Sediment Load}}{\text{Density of sediment} \times \text{PF}}$$

From the plan area and volume, sediment depth is obtained as

$$\text{Depth of Sediment} = \frac{\text{Volume of sediment}}{\text{plan area}}$$

Inlet Transition:

A symmetrical and smooth layout of the inlet expansion shall be designed to prevent the flow from separating from the sidewalls and bottom of the transition. This shall be achieved by providing an opening angle of the inlet transition in the range of 7° to 10º.

Outlet Transition:

A closing angle of the outlet transition in the range of 10° to 15° is provided.

The length of settling basin is designed to be 88 m and the width is designed to be 10.3 m. The depth is designed to be 8 m including freeboard.

The design calculation for settling basin is shown in Appendix B. The design drawings for settling basin is shown in Appendix D.

Check for efficiency:

1. Using Camp's Graph:

According to Camp's, efficiency depends upon two dimensionless parameters namely $\frac{\omega}{u_*}$ and $\frac{\omega A_s}{Q}$

Shear velocity, $u_* = \sqrt{gRS_e}$ or

For the practical case, the shear velocity shall be determined from the following formula

$$u_* = \frac{0.042 \times v_m}{R^{\frac{1}{6}}}$$

(Source: Guidelines for Settling Basin)

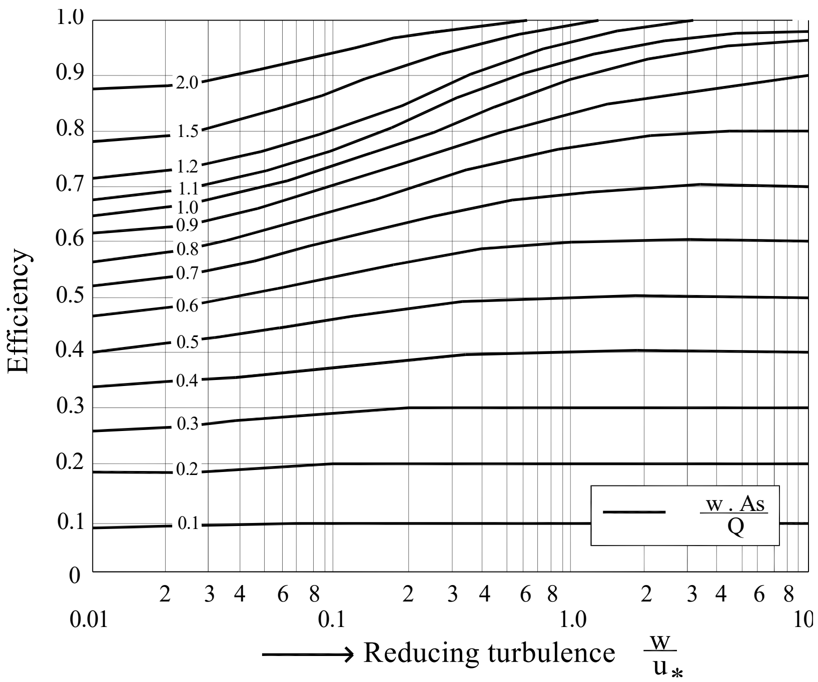


Figure 20:Trap efficiency for a rectangular reservoir under turbulent conditions by T.R. Camp

2. Using Vetter's equation:

$$\eta = 1 - e^{-\frac{\omega A_s}{Q}}$$

This formula does not consider the effect of turbulence in flow of the basin on the sediment trapping efficiency in it.

6.4.2 Water Conveyance

The proposed waterway structures for conveying water are:

- Headrace Tunnel
- Penstock pipe
- Tailrace

6.4.2.1 Headrace Tunnel

The headrace tunnel is designed for discharge of 15.76 m3/s. This tunnel is designed to cater the pressure imposed by flowing water. Therefore, the whole section, 2934 m is lined with shotcrete. The diameter of tunnel is 3.3 m. These excavations are proposed by drilling and blasting methods.

6.4.2.2 Surge Tank

Approximate solution by JAEGER

$$\frac{Z_{upsurge}}{Z_{max}} = 1 - \frac{2P_0}{3} + \frac{P_0^2}{9}$$

Where $Z_{upsurge}$ is the maximum surge height in the tank with friction.

$$Z_{max} = \frac{Q_T}{A_{st}}\sqrt{\frac{A_{st} \times L}{A_t \times g}}$$

i.e., maximum surge height without friction.

$$P_0 = \frac{h_{f_0}}{Z_{max}}$$

$h_{f_0} =$ friction headloss from reservoir to surge tank (friction headloss in tunnel).

$$\frac{Z_{downsurge}}{Z_{max}} = -1 + 2P_0$$

Thoma formula of surge tank area

$$A_{st,min} \geq \frac{A_T L V_0^2}{2gh_f\left(H_g - h_f\right)}$$

Where,

V_o = velocity of flow at the headrace tunnel

L= Length of headrace tunnel

A_T = Area of headrace tunnel

hf = head loss from reservoir to surge tank (i.e., in tunnel)

Hg = static head available between the reservoir level and the powerhouse (gross head).

6.4.2.3 Penstock Pipe

Hydraulic design considerations of penstock

The thickness of penstock pipe will be determined by using the thickness formula considering the pipe strength and available water head and water hammer considerations. The formula so needed is

$$t = \frac{PR}{\sigma_{st} \times \eta - 0.6P} + 0.15 \text{ cm}$$

Where,

t = thickness of pipe

P = internal pressure, kg/cm^2

R = internal radius of pipe

σ_{st} = safe stress allowable

η = joint efficiency factor

Dynamic head is taken = 20% of static head for thickness calculation

Design parameters of penstock

The penstock pipe is designed for a discharge of 5.25 m^3/s. Length of penstock is 300 m, internal diameter is 2 m and effective thickness of penstock is 7 mm. The joint efficiency has been adopted as 90%. There are total of 3 penstock pipes coming from surge tank, each of which leads to three turbines in the powerhouse.

Anchor blocks are provided for horizontal bend and vertical bends as well.

Anchor block design has not been done in this study.

Powerhouse Dimensions:

Length: It depends upon the number of units.

Center-center distance between units = 5D+2.5 m where D is turbine outlet diameter.

Width = 5D+2.5; Here, 2.5 m is extra passage from the wall.

Height: Depends upon height of machine usually taken as height of machine + clearance from the ground to lifting objects

The design calculation for powerhouse is shown in Appendix B

6.4.3 Powerhouse Hydro-Mechanical Installation

6.4.3.1 Unit Selection

The selection of unit capacity assumes that a minimum number of units could be installed for the more economic development of the Project, reliability of generation, and minimum loss of power during maintenance and operation at different stages of time.

The selection of units of turbine is carried out considering 3 no. of unit of vertical axis Francis turbine as the net available head for the project is found to be 60 m and design discharge to be 15.76 m3/s (Q40) resulting the total installed capacity to be 8.02 MW. With 3 no. of units, the required length of the surface powerhouse will be appropriate that means the appreciable amount in the construction cost. The rated discharge for each unit is calculated to be 5.25 m^3/s. The general trend of selecting the turbine type is as per available net head and available discharge at the site. The more scientific approach for selection of turbine is a consideration of the specific speed and speed number of the turbine.

6.4.3.2 Characteristics of turbine summarized below

- Rated net head = 60 m
- Rated efficiency = 86 %
- Rate flow/unit = 5.25 m3/s
- Synchronous speed = 750 rpm
- No. of poles = 8
- Specific Speed = 268 rpm
- Diameter of turbine = 0.7 m
- Setting of turbine = 3.58 m above TWL

[The design calculations of turbine is shown in Appendix B.]

7. Power and Energy Benefits

7.1 Introduction

Nepal has abundant water resources, and hydropower development has recently accelerated. The government has also identified solar, wind, and geo-thermal as potential future energy sources. In this regard, the government has long allocated funds and offered incentives for solar PV systems erected in remote locations where grid power supplies would not soon be available.

Energy has been referred to be the engine of economic development because it powers the machinery that multiplies human labor and raises productivity. Instead, load shedding in the nation has been eliminated thanks to the addition of 372 MW from India to the 372 MW of NEA-owned projects now in operation (with the exception of the 104 MW Kulekhani reservoir plan) and 220 MW from independent power providers. However, a lack of 300MW of power to meet the needs of industrial users has significantly reduced the country's industrial production. All facets of the economy, including the general public, have been severely impacted by the energy deficit. The country's failure to produce 668 MW of installed capacity during the 13th periodic plan (for F/Y 2070/71 to 2072/73) and the private sector's limited ability to add 93 MW to the national grid during the same period are the main causes of the present power problem. Additionally, hydropower growth in the nation has been hindered by its greater initial cost and a lack of a favorable investment climate. The 14th periodic plan aimed to reach 2301 MW of total installed capacity and begin work on hydroelectric projects with a 2552 MW total installed capacity in order to address the aforementioned challenges.

7.2 Present status

Fuel-wood is the main source of biomass, which provides more than 80% of the nation's energy needs. The industrial and transportation sectors mostly use imported hydrocarbon fuel, which comes in second place and accounts for around 19% of total fuel use. For a nation like Nepal, where the hydro energy potential is 83,000 MW with a technical and economic realistic capacity of 42,000 MW, the contribution of domestically produced electrical energy is a pitiful less than 2 percent. Currently, the installed capacity in the nation is at 851 MW, with hydropower facilities contributing 94% and diesel power plants the remaining portion. Independent Power Producers (IPP) account for around 41% of the world's power production. Eighty-eight percent of the electricity produced by hydropower plants comes from run-off-river-type projects. As a result, during the winter, both their peak serving capacity and energy output drop by up to 60%, causing a prolonged power outage. However, the improved management of the NEA and sufficient production of hydroelectricity in wet season Nepal is slowly moving to be the net exporter of power.

7.3 Integrated Nepal Power System (INPS)

In the country, the Integrated Nepal Power System and Isolated System are the two existing power systems. The integrated system accounts for 99 percent of the supply, whereas the remaining one percent is an isolated supply system that includes various captive generation and distribution systems supplying electricity to district headquarters and outlying areas. The power evacuated to the Integrated Nepal Power System (INPS) is about 851 MW and presented in Table 17.

Table 17: Existing Electricity Generating Facilities in Nepal

A. NEA Operated (MW)

Hydroelectric Plant	Storage	104 MW
Hydroelectric Plant	Run-off-river	1008 MW
Petroleum/Diesel Plant		53 MW

B. Independent Power Producers (IPPs)

Hydroelectric Plant	Run-off-river	1370 MW

Total- 2492 MW

In the country, the existing transmission line system is 2,970 circuit kilometers long. It primarily comprises a 1,000 kilometer long 132 kV national grid running horizontally across the country along the East-West Highway. Moreover, 66 kV and 132 kV transmission lines are connected to the generating plant and/or regional major distribution systems and/or load centers such as Kathmandu and Pokhara.

The development of small hydropower projects has been hampered by a lack of transmission line networks to transport generated electricity to the national grid. Therefore, cost-effective energy generation, transmission, and distribution in middle mountains where demand is rising and settlements are dispersed will be achieved by thorough transmission line planning focused on small hydropower projects.

7.4 Demand Forecast

NEA annually revises and publishes demand forecast, which is an integral part of the medium- and long-term development. The forecast for Fiscal Year 2010/11 is presented in Table 18 and the load forecast is around 10%. Electrification demand in rural areas is growing and this can be met cost effectively by the construction of small hydropower projects in the surrounding areas. This on one hand will promote small hydropower and on the other hand will reduce transmission losses.

Table 18: Load Forecast in Nepal

Year	Peak Load (MW)	Energy (GWh)	Energy Growth %
2010-11	967.1	4430.7	10.26
2011-12	1056.9	4851.3	9.49
2012-13	1163.2	5349.6	10.27
2013-14	1271.7	5859.9	9.54
2014-15	1387.2	6403.8	9.28
2015-16	1510.0	6984.1	9.06
2016-17	1640.8	7603.7	8.87
2017-18	1770.2	8218.2	8.08
2018-19	1906.9	8870.2	7.93
2019-20	2052.0	9662.9	8.94
2020-21	2206.0	10300.1	6.59
2021-22	2363.0	11053.6	7.32
2022-23	2525.4	11929.1	7.92
2023-24	2741.1	12870.2	7.89
2024-25	2951.1	13882.4	7.86
2025-26	3176.7	14971.2	7.84
2026-27	3418.9	16142.7	7.82
2027-28	3679.1	17403.60	7.81

7.5 Hydropower Projects under Construction

900 MW of Arun-III HEP is being constructed both by the effort of Nepal government and Indian Government. Construction of 456 MW Upper Tamakoshi Hydropower project has been recently completed. Also, 140 MW of the Tanahun Hydropower is also running at the full speed and expected to be completed by 2027. Likewise, private developers have started the construction of 9 small hydropower projects with a total capacity of 51.75 MW. In pipeline, are other mega projects such as 600 MW Lower Tamakoshi, 402 MW Upper Karnali, and 600 MW Upper Marshyangdi Project.

7.6 Energy Computation

The energy computation result for the ideal case of operation of the plant is given in Table Appendix C. However, the financial analysis of the project is carried out considering the 4% forced and scheduled outages of the plant.

The input data and assumptions made for computation of energy for Indrawati-III Hydropower are given in Table 19.

Table 19: Input Parameters and Assumptions

Parameters	**Values**
Design Discharge	15.75 m^3/s
Design Net head	60 m
Average Forced Outage Rate	4%
Number of Units	3
Installed capacity	8.42 MW
Overall efficiency	85.54 %

7.7 Assessment of Power and Energy benefits

For private small Independent Power Producers, NEA does a Power Purchase Agreement (PPA) at NRs 4.8 per kWh during wet period and NRs. 8.4 per kWh during dry period. The rate as per NEA for peaking is taken as Nrs. 10.55 per kWh.

8. Conclusion and Recommendations

8.1 Conclusion

The following main conclusions are drawn from pre-feasibility study of Indrawati- III Hydroelectricity Project:

- The Extreme flood at intake site for 100 years return period is about 871.46 m^3/s, while the design discharge for 40 percentile is around 15.76 m3/s.
- The optimum installed capacity is 8.02 MW comprising 3 generating units by vertical axis Francis turbines operating at an average net head 60 m.
- Based on estimated cost and benefits, the BC ratio of the project is equal to 2.01 and IRR equal to 20.62 % for Payback Period of 10 years. This suggests the project is financially attractive. Further, the project can be recommended for feasibility analysis.

8.2 Recommendations

The following recommendations are made:

- Proper geological survey and mapping should be carried out at headwork area and powerhouse area. Also tunneling is proposed so proper geological investigation should be carried out.
- Physical hydraulic modelling of the components of hydropower should be conducted in order to validate the design.
- Since the capacity of projects stands out to be more than 5 MW so EIA should be performed in order to avoid any environmental issues.
- Since the project involves the livelihood of the people in the project area directly so the participation of the local should be addressed and encouraged to focus on sustainability of the project as well as the involved environment.

9. References

Acts & Regulation, Electricity Act 2049. Kathmandu, Nepal (1992).

Acts & Regulation, Environment Protection Act, KTM, Nepal (1996).

Acts & Regulation, Water resources act 2049. Kathmandu, Nepal (1992).

Civil Works Guidelines for MICRO-HYDROPOWER IN NEPAL, Intermediate Technology Development Group (ITDG), Nepal, 2002.

Climatological Records of Nepal 1999-2000, Department of Hydrology and Meteorology, Ministry of Science and Technology, GoN, 2002

Design Guidelines for Headworks of Hydropower Projects, Department of Electricity Development, MoWR, GoN, 2006.

Guide on how to Develop a Small Hydropower Plant, European Small Hydropower Association (ESHA), 2004.

Guidelines for Study of Hydropower Projects, MoWR, Department of Electricity Development

High-Head Power Plants, Emil Mosonyi, Water Power Development, Volume (Tow/A and Two/B), 1991

Hydraulic Design Criteria, Corps of Engineers

Hydrological Records of Nepal, Stream flow Summary, Department of Hydrology and Meteorology, Ministry of Science and Technology, GoN, 1998.

Hydropower Development, Volume N0. 8, Hydraulic Design, Dagfinn Lysne, Brian Glover, Hakon Stole, Einar Tesaker, 2003, Norwegian University of Science and Technology,

Hydropower Structures (Including Micro, Mini and Small Power Stations), R.S. Varshney, 1986 NDP .Nepal District Profile, National Research Associated, Kathmandu Nepal. 1999

P. S. Nigam, Handbook of Hydro Electric Engineering S. K. Garg, Irrigation Engineering & Hydraulic Structures, Water Power Engineering, *M.M. Dandekar, 2000.*

WECS/DHM Report on Methodologies for Estimating Hydrologic Characteristics of Ungauged Locations in Nepal, July 1990, ReportNo.4/4/240990/1/1 SEQ.NO. 331

Appendix-A (Hydrological Analysis)

A. Hydro-Meteorological Database

A.1 Rainfall Frequency Analysis

A.1.1 By Logarithmic (Excel) Method

In this method, four meteorological stations have been used as reference.

Nawalpur Station (1008)

Table 20: 1008: Nawalpur Station's Rainfall Frequency analysis

Year	Extreme Rainfall (mm)	Extreme Rainfall in descending order(mm)	Rank(m)	Return period T=(n+1)/m	Predicted value from curve (mm)
1982	119.70	131.80	1	31.000	143.554
1983	79.00	131.40	2	15.500	133.248
1984	97.30	125.50	3	10.333	127.219
1985	114.20	121.80	4	7.750	122.941
1986	118.00	119.70	5	6.200	119.623
1987	106.00	119.40	6	5.167	116.912
1988	119.40	118.00	7	4.429	114.620
1989	87.60	115.20	8	3.875	112.635
1990	115.00	115.00	9	3.444	110.883
1991	89.70	114.20	10	3.100	109.317
1992	78.60	112.80	11	2.818	107.900
1993	115.20	109.60	12	2.583	106.606
1994	131.80	109.00	13	2.385	105.416
1995	102.00	107.60	15	2.067	103.288
1996	105.00	107.60	15	2.067	103.288
1997	121.80	106.00	17	1.824	101.427
1998	131.40	106.00	17	1.824	101.427
1999	102.00	105.60	18	1.722	100.577
2000	106.00	105.20	19	1.632	99.773
2001	112.80	105.00	20	1.550	99.010
2002	109.60	102.00	22	1.409	97.593
2003	109.00	102.00	22	1.409	97.593
2004	107.60	97.30	23	1.348	96.932
2005	107.60	96.20	24	1.292	96.299
2006	96.20	89.90	25	1.240	95.692
2007	85.20	89.70	26	1.192	95.109
2008	105.60	87.60	27	1.148	94.548
2009	89.90	85.20	28	1.107	94.007
2010	125.50	79.00	29	1.069	93.486
2011	105.20	78.60	30	1.033	92.982

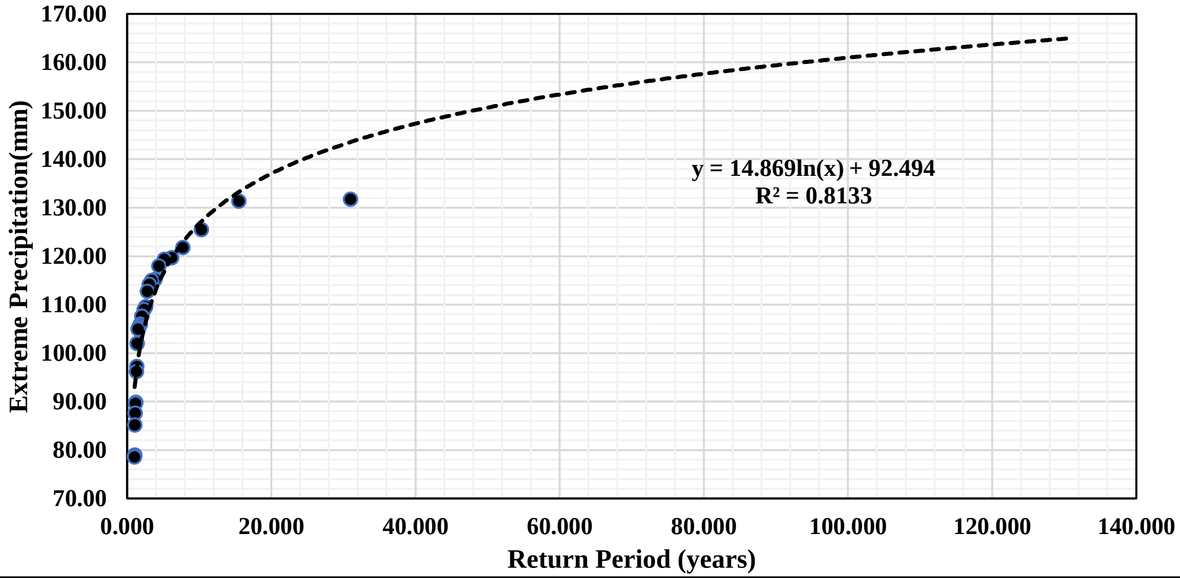


Figure 21: Return period vs Rainfall of Nawalpur's Station

Design Rainfall for 100 years Return Period = 160.968 mm

Sarmathang Station (1016)

Table 21: 1016: Sarmathang station's Rainfall Frequency analysis

Year	Extreme Rainfall (mm)	Extreme Rainfall in descending order(mm)	Rank(m)	Return period T=(n+1)/m	Predicted value from curve (mm)
1982	127.60	177.00	1	31.000	192.377
1983	133.20	160.10	2	15.500	166.179
1984	160.10	133.20	3	10.333	150.853
1985	123.20	127.60	4	7.750	139.980
1986	123.20	124.80	5	6.200	131.545
1987	177.00	124.53	6	5.167	124.654
1988	DN	123.20	7	4.429	118.828
1989	DN	123.20	8	3.875	113.781
1990	20.40	123.20	9	3.444	109.329
1991	84.00	121.47	10	3.100	105.347
1992	40.00	120.20	11	2.818	101.744
1993	DN	116.60	12	2.583	98.455
1994	DN	114.20	13	2.385	95.430
1995	90.10	106.00	14	2.214	92.629
1996	123.20	97.80	15	2.067	90.021
1997	120.20	95.00	16	1.938	87.582
1998	92.80	94.07	17	1.824	85.290
1999	116.60	92.80	18	1.722	83.130
2000	46.00	90.10	19	1.632	81.086
2001	16.80	90.00	20	1.550	79.148
2002	97.80	84.00	22	1.409	75.545
2003	84.00	84.00	22	1.409	75.545
2004	83.00	83.00	23	1.348	73.865
2005	77.20	80.80	24	1.292	72.257
2006	62.20	77.20	25	1.240	70.714

2007	80.80	62.20	26	1.192	69.231
2008	90.00	46.00	27	1.148	67.805
2009	114.20	40.00	28	1.107	66.430
2010	95.00	20.40	29	1.069	65.104
2011	106.00	16.80	30	1.033	63.822

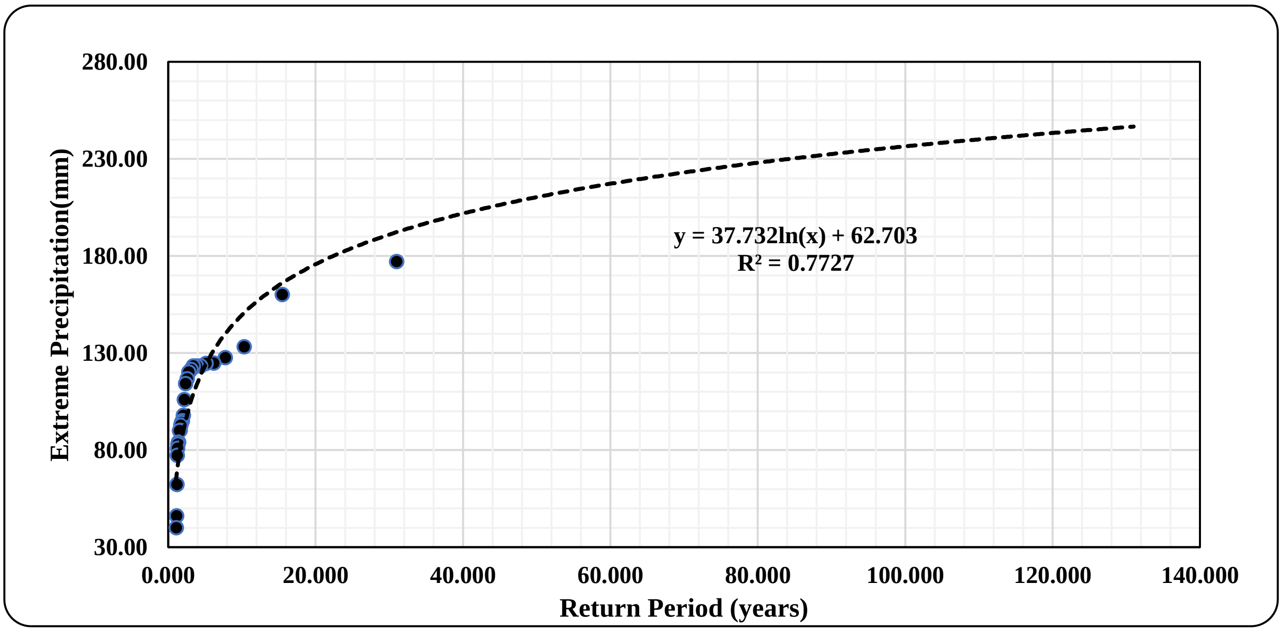


Figure 22: Return period vs Rainfall of Sarmathang Station’s

Design Rainfall for 100 years Return Period = 236.645 mm

Dhap Station (1025)

Table 22: 1025: Dhap station's Rainfall Frequency analysis

Year	Extreme Rainfall (mm)	Extreme Rainfall in descending order(mm)	Rank(m)	Return period T=(n+1)/m	Predicted value from curve (mm)
1982	115.80	280.00	1	31.000	215.975
1983	119.00	151.00	2	15.500	183.449
1984	116.60	130.00	3	10.333	164.422
1985	130.00	122.20	4	7.750	150.923
1986	91.00	120.50	5	6.200	140.452
1987	93.20	119.50	6	5.167	131.897
1988	96.00	119.00	7	4.429	124.663
1989	94.00	116.60	8	3.875	118.397
1990	110.80	115.80	9	3.444	112.870
1991	119.50	112.40	10	3.100	107.926
1992	82.30	110.80	11	2.818	103.454
1993	100.00	109.80	12	2.583	99.371
1994	93.70	100.00	13	2.385	95.615
1995	109.80	96.00	14	2.214	92.137
1996	120.50	94.00	15	2.067	88.900
1997	90.50	93.70	16	1.938	85.871
1998	112.40	93.20	17	1.824	83.026
1999	151.00	91.00	18	1.722	80.344

2000	82.50	90.50	19	1.632	77.807
2001	89.00	89.00	20	1.550	75.400
2002	80.80	82.50	21	1.476	73.111
2003	70.00	82.40	22	1.409	70.928
2004	49.10	82.30	23	1.348	68.842
2005	53.10	80.80	24	1.292	66.845
2006	40.10	70.00	25	1.240	64.929
2007	280.00	53.10	26	1.192	63.089
2008	50.10	50.10	27	1.148	61.318
2009	30.50	49.10	28	1.107	59.611
2010	82.40	40.10	29	1.069	57.964
2011	122.20	30.50	30	1.033	56.374

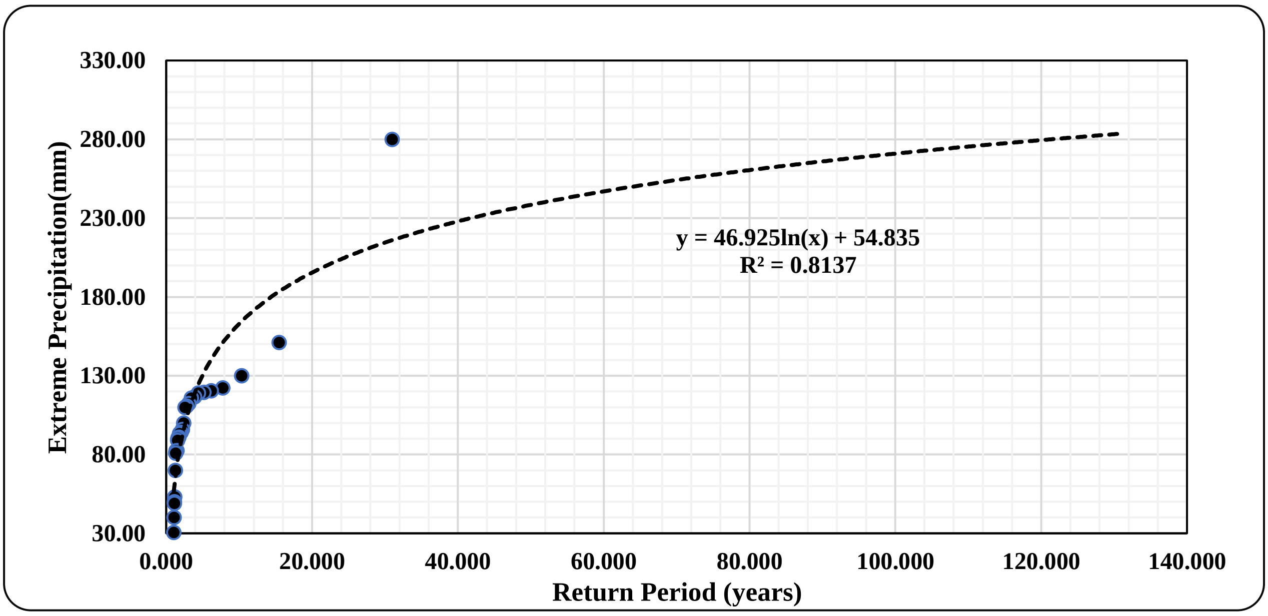


Figure 23: Return period vs Rainfall of Dhap Station's
Design Rainfall for 100 years Return Period = 270.933 mm

Tarke Ghyang (1058)

Table 23: 1058: Tarke Ghyang station's Rainfall Frequency analysis

Year	Extreme Rainfall (mm)	Extreme Rainfall in descending order(mm)	Rank(m)	Return period T=(n+1)/m	Predicted value from curve (mm)
1982	95.50	234.90	1	31.000	224.603
1983	181.60	181.60	2	15.500	195.201
1984	135.20	159.00	3	10.333	178.001
1985	107.40	153.10	4	7.750	165.798
1986	112.00	152.00	5	6.200	156.333
1987	120.00	149.20	6	5.167	148.599
1988	159.00	148.10	7	4.429	142.060
1989	100.60	142.50	8	3.875	136.395
1990	153.10	135.60	9	3.444	131.399
1991	16.80	135.20	10	3.100	126.930
1992	234.90	131.00	11	2.818	122.887

1993	149.20	121.00	12	2.583	119.196
1994	148.10	120.00	13	2.385	115.801
1995	102.60	119.70	14	2.214	112.657
1996	121.00	113.30	15	2.067	109.731
1997	112.50	112.50	16	1.938	106.993
1998	113.30	112.00	17	1.824	104.421
1999	103.00	107.40	18	1.722	101.997
2000	119.70	104.90	20	1.550	97.527
2001	135.60	104.90	20	1.550	97.527
2002	85.00	103.00	21	1.476	95.458
2003	87.40	102.60	22	1.409	93.484
2004	63.50	100.60	23	1.348	91.599
2005	104.90	95.50	24	1.292	89.793
2006	88.20	88.50	25	1.240	88.062
2007	104.90	88.20	26	1.192	86.398
2008	131.00	87.40	27	1.148	84.797
2009	142.50	85.00	28	1.107	83.255
2010	88.50	63.50	29	1.069	81.766
2011	152.00	16.80	30	1.033	80.328

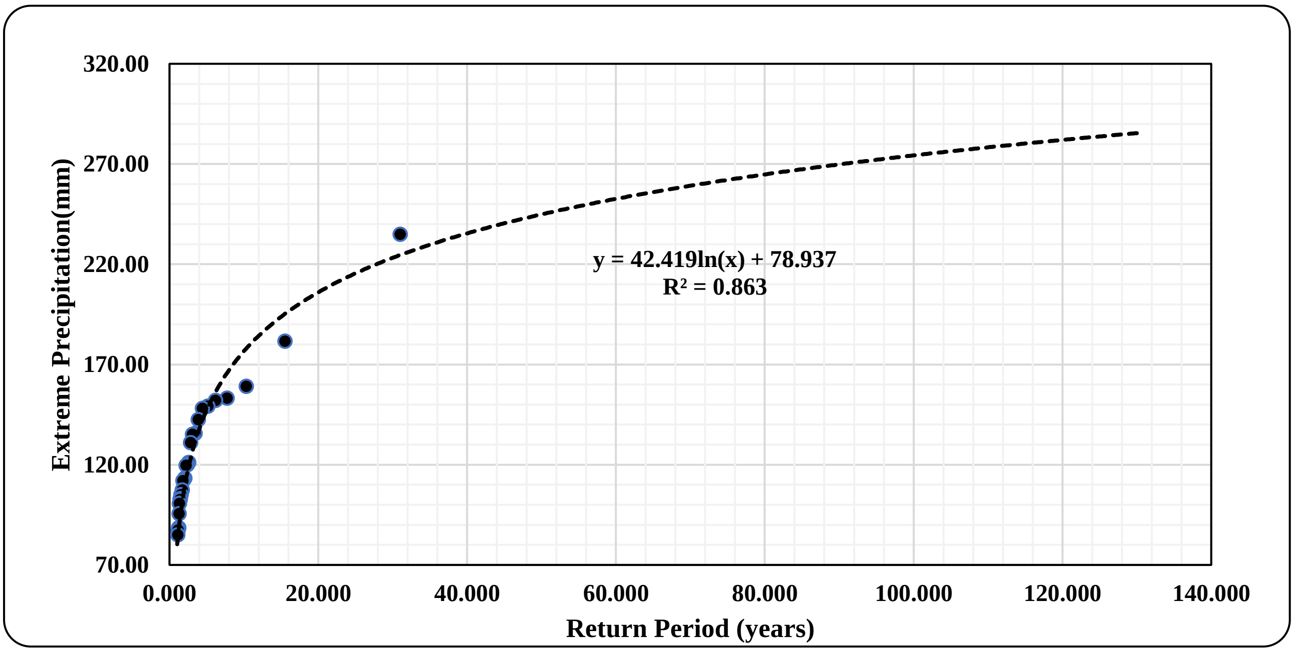


Figure 24: Return period vs Rainfall of Tarke Ghyang Station's
Design Rainfall for 100 years Return Period = 274.284 mm

A.1.2 By Gumbel Method

Table 24: Gumbel Method for Rainfall frequency Analysis

Year	Extreme Rainfall(mm)			
	1008-Nawalpur	1016-Sarmathang	1025-Dhap	1058-Tarke Ghyang
1982	119.70	127.60	115.80	95.50
1983	79.00	133.20	119.00	181.60
1984	97.30	160.10	116.60	135.20
1985	114.20	123.20	130.00	107.40
1986	118.00	123.20	91.00	112.00
1987	106.00	177.00	93.20	120.00

1988	119.40	124.80	96.00	159.00
1989	87.60	94.07	94.00	100.60
1990	115.00	20.40	110.80	153.10
1991	89.70	84.00	119.50	16.80
1992	78.60	40.00	82.30	234.90
1993	115.20	121.47	100.00	149.20
1994	131.80	124.53	93.70	148.10
1995	102.00	90.10	109.80	102.60
1996	105.00	123.20	120.50	121.00
1997	121.80	120.20	90.50	112.50
1998	131.40	92.80	112.40	113.30
1999	102.00	116.60	151.00	103.00
2000	106.00	46.00	82.50	119.70
2001	112.80	16.80	89.00	135.60
2002	109.60	97.80	80.80	85.00
2003	109.00	84.00	70.00	87.40
2004	107.60	83.00	49.10	63.50
2005	107.60	77.20	53.10	104.90
2006	96.20	62.20	40.10	88.20
2007	85.20	80.80	280.00	104.90
2008	105.60	90.00	50.10	131.00
2009	89.90	114.20	30.50	142.50
2010	125.50	95.00	82.40	88.50
2011	105.20	106.00	122.20	152.00
X_mean	**106.463**	**98.316**	**99.197**	**118.967**
Stdev.S	**14.060**	**36.550**	**44.239**	**38.877**
X_100	**157.826**	**231.833**	**260.800**	**260.986**

A.2 Stream Database

Table 25:Mean monthly discharge at station 447

Mean Daily Discharge from Trishuli (Betrawati)												
Days	**Jan**	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**	**Oct**	**Nov**	**Dec**
1	54.31	44.51	43.34	46.89	67.12	140.64	452.43	667.70	542.22	263.78	114.30	71.53
2	54.00	43.99	43.36	46.71	71.87	144.88	486.70	698.39	523.35	257.70	111.95	70.29
3	53.19	44.10	42.98	46.77	74.70	148.63	474.57	678.91	510.09	253.30	108.30	68.99
4	52.67	43.97	43.31	46.20	76.69	153.77	503.48	623.61	506.35	240.30	105.87	67.93
5	52.01	43.80	43.44	45.79	79.30	156.13	479.91	619.09	508.17	239.09	104.07	67.00
6	51.50	43.59	43.01	47.75	79.02	162.93	479.74	638.65	499.65	241.04	101.71	66.53
7	51.10	43.65	42.93	47.89	78.22	185.41	491.87	619.13	486.22	233.70	98.97	65.72
8	50.49	44.18	42.73	48.06	80.16	192.85	508.52	624.57	484.30	219.35	97.24	64.90
9	50.00	44.29	43.13	48.53	80.04	204.22	482.39	622.39	485.83	210.26	95.52	64.18
10	49.87	43.67	43.61	48.54	78.30	195.50	494.96	658.52	482.78	212.74	93.61	65.02
11	49.36	43.39	43.96	49.35	76.03	199.60	523.57	630.65	473.39	201.83	92.47	63.51
12	49.24	43.32	44.72	49.99	74.81	195.65	503.52	655.74	470.83	198.65	91.26	62.81
13	49.07	43.43	44.21	52.23	76.27	229.96	520.04	659.87	462.74	187.61	89.70	61.93
14	48.62	43.28	43.90	52.42	82.96	209.78	561.26	648.09	449.74	177.13	88.03	61.58
15	48.92	43.37	44.21	53.53	88.14	215.52	576.09	639.13	432.74	169.48	87.33	60.71
16	48.03	43.78	44.60	53.61	91.84	238.43	610.26	603.17	416.30	162.61	86.01	60.03
17	47.74	43.37	44.31	54.45	91.60	256.00	601.35	625.35	403.78	163.87	85.20	59.64
18	47.70	43.20	45.66	55.50	96.61	300.04	596.26	628.70	376.13	166.10	84.20	58.98
19	47.15	43.03	45.51	54.09	97.39	293.74	595.00	646.13	372.09	161.38	82.89	58.65

20	46.81	43.10	45.81	55.87	104.32	300.09	603.78	646.61	368.22	159.11	81.50	58.17
21	46.23	42.62	45.59	56.52	104.10	331.70	592.87	636.87	353.22	153.55	80.55	57.10
22	46.29	42.75	45.80	59.53	109.60	360.48	610.04	645.39	338.70	145.38	78.71	56.50
23	46.73	42.84	45.90	61.31	110.05	364.65	616.00	648.61	334.57	138.51	77.72	56.03
24	46.38	43.48	45.83	58.33	115.31	379.35	645.78	607.30	328.57	134.80	76.47	55.60
25	46.50	43.40	45.32	58.66	122.85	375.52	635.04	608.43	334.00	131.67	75.98	54.90
26	45.78	43.59	46.00	58.20	140.31	384.83	671.87	597.74	314.30	128.77	75.82	54.20
27	45.44	43.62	45.13	60.71	145.60	432.13	689.83	563.52	301.39	125.43	74.87	55.10
28	45.22	43.48	45.05	62.70	137.42	425.65	699.91	567.00	293.48	121.90	73.46	54.22
29	45.34	39.65	45.35	65.34	136.46	427.83	683.70	560.52	290.74	121.71	72.29	53.75
30	44.89		47.07	64.41	138.06	420.13	683.17	552.48	267.43	118.85	71.77	53.37
31	44.79		47.31		136.86		702.43	532.61		117.00		52.79

Table 26: Mean monthly discharge at station 505

Mean Daily Discharge from Bagmati (Sundarijal)												
Days	**Jan**	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**	**Oct**	**Nov**	**Dec**
1	0.53	0.36	0.31	0.30	0.41	0.43	1.86	3.81	3.50	2.22	0.94	0.57
2	0.43	0.36	0.31	0.30	0.38	0.50	1.96	3.82	3.61	2.12	0.92	0.57
3	0.43	0.35	0.31	0.29	0.39	0.51	2.08	3.70	3.63	2.17	0.90	0.53
4	0.42	0.35	0.31	0.29	0.40	0.51	1.86	3.67	3.48	2.05	0.87	0.52
5	0.44	0.34	0.31	0.31	0.40	0.50	1.93	3.60	3.60	1.98	0.85	0.52
6	0.42	0.34	0.31	0.30	0.49	0.56	2.03	3.53	3.44	1.92	0.84	0.51
7	0.42	0.34	0.30	0.30	0.44	0.57	1.98	3.56	3.46	1.83	0.84	0.50
8	0.43	0.34	0.29	0.30	0.42	0.55	2.05	3.69	3.25	1.79	0.82	0.49
9	0.42	0.37	0.30	0.29	0.51	0.60	2.33	3.99	3.21	1.72	0.79	0.49
10	0.41	0.34	0.29	0.31	0.54	0.62	2.33	3.66	3.20	1.70	0.78	0.49
11	0.40	0.35	0.29	0.31	0.44	0.56	2.37	3.66	3.06	1.70	0.77	0.48
12	0.40	0.33	0.31	0.30	0.44	0.58	2.38	3.75	3.01	1.71	0.76	0.47
13	0.40	0.34	0.31	0.31	0.40	0.65	2.43	3.72	3.09	1.62	0.75	0.47
14	0.39	0.35	0.30	0.31	0.38	0.77	2.77	4.19	3.02	1.55	0.74	0.47
15	0.40	0.34	0.31	0.44	0.43	0.78	2.82	3.60	2.93	1.50	0.71	0.46
16	0.43	0.34	0.30	0.33	0.38	0.73	3.04	3.63	2.87	1.48	0.71	0.46
17	0.41	0.33	0.30	0.34	0.38	0.75	2.94	3.62	2.76	1.38	0.70	0.46
18	0.40	0.32	0.31	0.31	0.45	0.77	2.82	3.79	2.73	1.34	0.69	0.45
19	0.40	0.32	0.31	0.31	0.41	0.97	3.21	3.80	2.72	1.29	0.67	0.45
20	0.40	0.32	0.31	0.34	0.41	0.97	3.36	3.61	2.91	1.26	0.66	0.44
21	0.39	0.32	0.31	0.31	0.42	0.84	3.24	3.66	2.80	1.21	0.65	0.44
22	0.39	0.32	0.30	0.31	0.40	0.93	3.25	3.67	2.68	1.18	0.64	0.43
23	0.39	0.31	0.30	0.31	0.40	0.90	3.19	3.87	2.63	1.15	0.63	0.43
24	0.39	0.31	0.30	0.32	0.39	1.03	2.89	3.76	2.65	1.10	0.62	0.43
25	0.38	0.31	0.30	0.37	0.38	1.28	3.04	3.91	2.56	1.11	0.61	0.42
26	0.38	0.31	0.31	0.36	0.38	1.23	3.41	3.95	2.55	1.07	0.60	0.42
27	0.37	0.31	0.31	0.37	0.42	1.33	3.35	3.68	2.54	1.04	0.59	0.42
28	0.37	0.31	0.30	0.35	0.40	1.57	3.49	3.77	2.46	1.02	0.58	0.42
29	0.37	0.46	0.32	0.37	0.41	1.55	3.53	3.84	2.62	1.00	0.57	0.42
30	0.37		0.32	0.39	0.41	1.55	3.62	3.69	2.71	0.98	0.56	0.41
31	0.36		0.33		0.43		4.37	3.56		0.96		0.41

Table 27: Mean monthly discharge at station 620

Mean Daily Discharge from Balephi (Jalbire)												
Days	**Jan**	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**	**Oct**	**Nov**	**Dec**
1	14.57	12.50	11.27	11.57	14.00	21.87	105.80	172.90	177.88	79.59	32.70	20.07
2	14.49	12.39	11.24	11.72	14.63	21.87	110.50	159.44	155.23	80.67	32.17	19.90
3	14.36	12.27	11.24	11.58	14.55	22.12	118.24	156.43	160.06	78.20	31.45	19.81
4	14.23	12.14	11.18	11.47	14.73	22.90	117.11	158.35	174.01	72.01	31.06	19.44

5	14.06	12.08	11.23	11.48	15.08	23.72	107.80	159.33	167.43	71.70	30.37	19.17
6	14.12	12.01	11.20	11.54	15.37	25.65	130.80	160.78	160.42	72.16	29.61	18.90
7	14.05	11.99	11.17	11.46	15.94	27.58	116.28	155.60	146.03	66.22	29.03	18.74
8	13.90	12.26	11.02	11.43	15.75	31.03	144.52	152.30	151.48	61.88	28.49	18.59
9	13.79	12.13	10.97	11.54	17.51	34.83	121.14	155.12	151.04	60.02	27.86	18.70
10	13.67	12.06	10.97	11.59	15.97	29.84	124.67	166.10	139.37	63.66	27.30	18.63
11	13.60	12.03	11.20	11.81	15.46	33.43	137.07	165.54	132.33	58.82	26.90	18.25
12	13.52	11.92	11.33	11.84	15.42	31.15	125.56	167.39	135.30	56.80	26.55	18.03
13	13.43	11.90	11.20	12.08	15.70	36.42	129.60	174.97	128.58	53.58	26.15	17.79
14	13.36	11.87	10.93	12.06	15.70	35.07	141.70	175.40	129.68	51.09	25.87	17.60
15	13.40	11.79	11.08	12.53	17.08	37.35	147.84	184.12	122.06	49.87	25.30	17.43
16	13.57	11.87	11.23	12.56	16.95	45.88	143.39	182.74	123.35	47.84	25.03	17.04
17	13.41	11.73	11.20	12.37	17.03	57.29	152.46	190.36	138.24	50.90	24.81	17.02
18	13.21	11.66	11.39	12.60	17.43	53.09	146.56	213.77	116.56	57.43	24.58	16.96
19	13.09	11.60	11.32	12.36	17.16	54.76	152.36	206.15	113.84	51.60	23.99	16.84
20	13.07	11.54	11.19	12.29	17.26	53.63	159.17	199.07	115.91	50.15	23.53	16.59
21	12.97	11.46	11.23	12.56	17.07	59.11	163.28	196.54	109.30	47.49	23.13	16.41
22	12.95	11.44	11.32	12.79	17.26	63.64	159.81	176.97	103.83	44.74	22.76	16.13
23	12.78	11.40	11.20	13.86	17.82	72.11	155.73	190.17	102.81	42.20	22.38	16.01
24	12.72	11.40	11.17	13.14	18.43	80.14	166.28	176.17	99.52	40.41	21.92	15.90
25	12.72	11.47	11.22	13.94	19.97	83.64	172.26	189.27	99.65	39.33	21.72	15.73
26	12.68	11.35	11.22	12.83	26.51	83.00	165.82	174.83	90.07	38.38	21.49	15.67
27	12.62	11.34	11.07	13.37	24.42	94.11	170.57	197.40	90.31	37.10	21.09	15.87
28	12.58	11.36	11.07	13.42	23.79	90.57	170.78	189.57	88.56	36.04	20.77	15.47
29	12.67	10.08	11.30	13.56	22.19	101.83	157.40	180.93	90.64	35.13	20.43	15.28
30	12.58		11.78	13.76	21.13	93.92	157.80	173.57	81.62	34.05	20.32	15.20
31	12.48		11.70		21.54		169.70	176.90		33.26		15.03

Table 28: Mean monthly discharge at station 630

Mean Daily Discharge from Sunkoshi (Pachuwarghat)												
Days	**Jan**	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**	**Oct**	**Nov**	**Dec**
1	63.93	51.90	46.10	48.89	61.19	99.40	437.61	741.00	648.61	287.13	135.24	84.70
2	63.79	51.45	45.69	48.20	60.46	97.66	433.78	730.78	606.22	274.48	132.97	83.76
3	62.77	51.20	44.88	47.73	62.91	96.27	449.04	752.26	580.96	271.57	129.78	82.66
4	62.59	50.48	44.79	48.82	65.65	99.59	437.13	671.39	592.61	258.83	128.58	81.65
5	61.68	50.37	44.79	49.45	65.40	104.98	417.30	665.00	555.30	251.00	124.97	81.16
6	60.43	49.51	45.10	47.93	69.30	120.67	431.65	688.65	582.04	266.83	123.03	80.20
7	60.11	49.27	45.44	48.31	69.97	116.00	454.04	643.74	545.04	240.35	121.54	79.18
8	59.75	50.91	45.37	46.59	70.09	124.47	489.04	652.52	525.74	235.00	120.00	77.99
9	59.37	49.90	44.63	46.14	72.78	160.96	446.13	643.13	542.00	226.91	118.33	77.38
10	59.15	49.48	44.53	46.33	66.83	128.57	463.65	715.87	519.26	233.22	117.55	78.15
11	58.84	50.20	44.65	48.29	66.48	140.76	518.13	697.96	493.52	224.52	115.71	76.46
12	58.09	49.75	47.56	47.47	66.30	135.33	501.61	704.70	479.61	218.91	113.47	75.79
13	57.92	49.09	45.33	47.96	68.81	131.87	499.61	731.00	456.91	203.57	111.82	75.03
14	57.08	48.63	45.50	48.54	70.21	154.41	551.17	689.61	475.39	194.87	110.16	74.34
15	56.56	48.60	45.52	51.05	72.55	149.56	564.61	715.96	454.74	191.96	107.30	74.14
16	57.31	48.17	45.69	51.70	80.54	170.92	594.57	659.35	451.00	185.52	106.52	72.33
17	56.96	48.41	46.83	52.06	79.29	194.38	590.13	694.04	446.70	184.74	104.69	71.46
18	56.20	47.71	45.19	53.78	78.42	221.23	571.48	769.30	416.00	213.70	103.76	70.77
19	55.57	47.31	45.78	52.78	73.56	215.43	601.70	740.78	407.65	195.35	101.54	70.10
20	55.34	47.03	44.50	51.77	76.00	211.77	653.39	725.61	392.52	195.17	99.57	69.38
21	54.77	47.11	45.49	51.91	76.99	238.30	607.17	760.78	375.65	179.74	98.01	68.43
22	54.49	46.72	45.29	51.79	79.61	251.65	638.83	711.96	365.91	171.35	96.44	67.61
23	54.60	46.63	45.58	55.55	79.73	275.48	666.26	719.35	374.48	164.70	94.70	67.48
24	53.87	46.58	45.09	54.55	82.77	305.39	746.35	664.48	359.61	162.50	93.52	66.39

25	53.63	46.06	44.43	55.06	83.96	322.65	729.04	684.43	344.52	158.37	91.43	65.67
26	53.43	45.75	45.47	55.00	91.37	325.48	687.43	655.83	343.39	153.91	89.99	65.20
27	52.69	45.57	45.19	55.32	98.78	340.04	744.13	669.30	326.43	150.04	88.88	66.14
28	52.66	45.35	45.14	55.83	99.02	348.83	746.57	631.22	316.09	148.20	87.51	66.05
29	51.94	44.58	45.74	58.70	101.83	357.78	688.43	637.00	313.83	144.19	85.82	64.87
30	52.10	0.00	47.48	58.95	97.03	363.91	699.13	645.39	291.91	140.84	84.65	64.05
31	52.32	0.00	49.43	0.00	94.60	0.00	748.83	609.91	0.00	138.69	0.00	63.70

Table 29:Mean monthly discharge at station 647

Mean Daily Discharge from Tamakoshi (Busti)												
Days	**Jan**	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**	**Oct**	**Nov**	**Dec**
1	32.40	26.75	23.92	24.98	33.69	71.68	342.39	504.70	406.30	189.74	73.71	45.83
2	32.29	26.59	24.07	25.39	34.90	72.89	328.48	494.74	395.22	181.96	72.56	45.04
3	31.90	26.44	23.95	25.37	36.05	76.14	334.61	519.13	387.87	177.30	70.93	44.48
4	31.67	26.20	23.99	25.23	36.62	80.07	332.26	483.43	381.22	164.83	69.29	44.09
5	31.46	26.11	24.02	25.41	37.63	80.16	325.30	504.91	374.57	166.83	67.95	43.79
6	31.07	25.91	23.86	25.41	39.43	84.52	341.87	486.70	392.83	165.13	66.40	42.94
7	30.87	25.86	23.90	24.93	40.03	90.16	350.52	445.96	360.61	150.78	65.30	42.36
8	30.58	25.76	23.76	24.80	39.24	96.68	369.22	461.17	365.26	145.04	63.93	41.48
9	30.30	25.68	23.66	24.98	40.33	101.08	359.78	485.09	367.22	138.56	63.00	40.63
10	30.05	25.59	23.53	25.40	40.60	100.05	374.26	486.74	354.61	142.16	61.84	40.16
11	29.77	25.49	24.00	25.70	37.78	102.15	418.17	512.83	356.52	133.51	60.87	39.53
12	29.54	25.37	24.33	25.67	39.03	101.44	405.00	530.65	350.65	134.58	60.03	39.03
13	29.34	25.35	23.56	26.19	39.33	106.08	390.39	497.09	340.78	127.44	59.18	38.83
14	29.20	25.34	23.58	26.83	41.49	125.94	422.17	490.09	332.83	120.47	58.45	38.57
15	29.00	25.18	23.53	26.83	46.15	125.97	404.13	503.78	306.83	114.82	57.37	38.03
16	28.77	25.17	23.79	28.05	45.21	142.23	445.61	482.70	293.74	110.34	56.46	37.66
17	28.78	25.11	23.75	27.43	46.57	163.72	451.83	505.61	308.09	110.73	55.75	37.48
18	28.26	25.01	24.10	27.81	49.51	167.39	433.52	503.70	284.17	105.86	54.91	37.07
19	28.06	24.84	23.87	27.82	48.71	172.55	458.04	511.43	273.09	104.46	54.13	36.75
20	27.86	24.90	23.87	28.32	48.66	184.38	479.87	526.13	261.87	103.10	53.20	36.35
21	27.90	24.58	24.10	28.48	50.12	205.13	511.87	506.30	246.87	97.79	52.28	35.86
22	27.70	24.47	24.09	28.70	55.32	202.51	499.91	496.22	246.91	96.00	51.30	35.27
23	27.77	24.43	24.29	29.96	58.68	222.87	479.57	498.04	239.87	91.27	50.87	35.13
24	27.73	24.51	24.37	29.27	56.16	249.65	513.13	461.74	231.70	88.27	49.98	34.65
25	27.50	24.29	24.25	29.84	60.93	250.07	537.57	493.52	234.96	86.59	49.41	34.27
26	27.34	24.29	24.10	30.40	70.43	261.26	541.87	491.35	257.04	83.61	48.47	33.98
27	27.15	24.14	24.00	31.11	70.82	278.13	544.09	490.22	235.09	81.82	48.04	33.86
28	26.95	24.05	23.90	31.26	68.27	272.78	522.22	480.48	223.13	80.21	47.40	33.63
29	27.02	39.28	24.08	31.87	71.77	292.04	539.43	458.00	211.57	78.50	46.80	33.32
30	26.79		24.79	34.27	70.81	306.26	483.61	444.26	197.57	76.70	46.28	32.87
31	26.85		24.57		69.90		518.30	423.22		75.21		32.55

A.2.1 Regional Analysis- 5 HSC

Table 30: Long term mean monthly flows (m^3/s) at 5 Hydrologically Similar Catchments regions (5 HSC)

River (location)	Balephi (Jalbire)	Trisuli (Betrawati)	Bagmati (Sundarijal)	Sunkoshi (Pachuwarghat)	Tamakoshi (Busti)
Jan	13.38	48.56	0.405	57.09	29.09
Feb	11.76	43.39	0.338	45.28	25.75
Mar	11.22	44.61	0.306	45.55	23.99
Apr	12.37	53.66	0.326	49.56	27.59
May	17.70	98.13	0.418	76.85	49.17
Jun	50.72	267.53	0.836	193.67	159.53

Jul	143.29	573.43	2.771	574.45	434.16
Aug	176.72	624.35	3.734	691.04	489.67
Sep	126.50	413.71	2.989	438.18	307.30
Oct	53.62	179.25	1.488	202.13	120.12
Nov	25.62	88.59	0.725	104.44	57.87
Dec	17.36	60.70	0.466	72.98	38.24
Yearly	55.02	207.99	1.233	212.60	146.87

Table 31: Long term monthly UAF ($m^3/s/km^2$) in region and for Indrawati-3 Intake

River(location)	**Balephi (Jalbire)**	**Trisuli (Betrawati)**	**Bagmati (Sundarijal)**	**Sunkoshi (Pachuwarghat)**	**Tamakoshi (Busti)**	**Average (For IW-3 intake)**
Jan	0.0213	0.0118	0.0238	0.0116	0.0106	0.0158
Feb	0.0187	0.0106	0.0199	0.0092	0.0094	0.0135
Mar	0.0178	0.0109	0.0180	0.0093	0.0087	0.0129
Apr	0.0197	0.0131	0.0192	0.0101	0.0100	0.0144
May	0.0281	0.0239	0.0246	0.0156	0.0179	0.0220
Jun	0.0806	0.0651	0.0491	0.0394	0.0579	0.0584
Jul	0.2278	0.1395	0.1630	0.1168	0.1577	0.1610
Aug	0.2809	0.1519	0.2196	0.1405	0.1779	0.1942
Sep	0.2011	0.1007	0.1758	0.0891	0.1116	0.1357
Oct	0.0853	0.0436	0.0875	0.0411	0.0436	0.0602
Nov	0.0407	0.0216	0.0427	0.0212	0.0210	0.0294
Dec	0.0276	0.0148	0.0274	0.0148	0.0139	0.0197
Yearly	0.0875	0.0506	0.0726	0.0432	0.0534	0.0614

A.2.2 Comparison of mean monthly flow at intake using different methods

Table 32: Long term monthly flows (m^3/s) at IW-3 intake from different approaches

Approach Method	DHM 2004	5 HSC	MHSP
Jan	7.73	6.815	5.906
Feb	6.53	5.823	4.89
Mar	4.93	5.564	4.549
Apr	5.19	6.211	5.882
May	7.53	9.489	6.761
Jun	28.02	25.19	23.19
Jul	68.44	69.446	69.854
Aug	94.08	83.766	81.586
Sep	63.6	58.533	62.851
Oct	30.08	25.967	28.592
Nov	14	12.681	13.748
Dec	9.52	8.497	8.909
Yearly	28.3	26.484	26.393

A.2.3 Recommended mean monthly flow at intake

Table 33: Recommended mean monthly flow at intake (m^3/s)

Approach Method	5 HSC
Jan	6.815
Feb	5.823
Mar	5.564
Apr	6.211
May	9.489
Jun	25.19
Jul	69.446
Aug	83.766
Sep	58.533
Oct	25.967
Nov	12.681
Dec	8.497
Yearly	26.484

A.2.4 Long term daily flow Analysis

Table 34: Long term daily flow analysis

Days	**Long term average flow(m^3/s)**	**Available flow(m^3/s)**	**Probability of exceedence**	**Flow in descending order (m^3/s)**	**Design Discharge (m^3/s)**	**Actual Discharge (m^3/s)**	**Power MW**	**Energy (GWh)**
1	7.964	7.423	0.27	90.480	15.76	15.76	8.02	0.197
2	7.433	6.892	0.55	89.593	15.76	15.76	8.02	0.192
3	7.368	6.827	0.82	89.025	15.76	15.76	8.02	0.192
4	7.278	6.737	1.09	87.863	15.76	15.76	8.02	0.192
5	7.320	6.779	1.37	87.560	15.76	15.76	8.02	0.192
6	7.182	6.641	1.64	87.014	15.76	15.76	8.02	0.192
7	7.152	6.611	1.91	85.830	15.76	15.76	8.02	0.192
8	7.154	6.613	2.19	85.496	15.76	15.76	8.02	0.192
9	7.063	6.522	2.46	85.330	15.76	15.76	8.02	0.192
10	6.981	6.440	2.73	85.076	15.76	15.76	8.02	0.192
11	6.896	6.355	3.01	84.738	15.76	15.76	8.02	0.192
12	6.862	6.321	3.28	84.735	15.76	15.76	8.02	0.192
13	6.837	6.296	3.55	84.579	15.76	15.76	8.02	0.192
14	6.748	6.207	3.83	84.429	15.76	15.76	8.02	0.192
15	6.795	6.254	4.10	84.195	15.76	15.76	8.02	0.192
16	6.958	6.417	4.37	84.132	15.76	15.76	8.02	0.192
17	6.822	6.281	4.64	83.934	15.76	15.76	8.02	0.192
18	6.714	6.173	4.92	83.687	15.76	15.76	8.02	0.192
19	6.668	6.127	5.19	83.397	15.76	15.76	8.02	0.192
20	6.648	6.107	5.46	82.925	15.76	15.76	8.02	0.192
21	6.563	6.022	5.74	82.642	15.76	15.76	8.02	0.192
22	6.550	6.009	6.01	82.615	15.76	15.76	8.02	0.192
23	6.540	5.999	6.28	82.440	15.76	15.76	8.02	0.192
24	6.511	5.970	6.56	82.290	15.76	15.76	8.02	0.192
25	6.451	5.910	6.83	82.285	15.76	15.76	8.02	0.192
26	6.422	5.881	7.10	82.281	15.76	15.76	8.02	0.192
27	6.337	5.796	7.38	81.569	15.76	15.76	8.02	0.192

28	6.320	5.779	7.65	81.469	15.76	15.76	8.02	0.192
29	6.324	5.783	7.92	81.224	15.76	15.76	8.02	0.192
30	6.298	5.757	8.20	81.046	15.76	15.76	8.02	0.192
31	6.237	5.696	8.47	80.523	15.76	15.76	8.02	0.192
32	6.224	5.683	8.74	80.155	15.76	15.76	8.02	0.192
33	6.185	5.644	9.02	80.056	15.76	15.76	8.02	0.192
34	6.111	5.570	9.29	79.811	15.76	15.76	8.02	0.192
35	6.070	5.529	9.56	79.651	15.76	15.76	8.02	0.192
36	6.003	5.462	9.84	78.824	15.76	15.76	8.02	0.192
37	5.968	5.427	10.11	78.075	15.76	15.76	8.02	0.192
38	5.960	5.419	10.38	77.541	15.76	15.76	8.02	0.192
39	6.034	5.493	10.66	77.507	15.76	15.76	8.02	0.192
40	6.151	5.610	10.93	77.425	15.76	15.76	8.02	0.192
41	5.966	5.425	11.20	77.123	15.76	15.76	8.02	0.192
42	6.016	5.475	11.48	77.102	15.76	15.76	8.02	0.192
43	5.886	5.345	11.75	76.922	15.76	15.76	8.02	0.192
44	5.924	5.383	12.02	76.645	15.76	15.76	8.02	0.192
45	5.959	5.418	12.30	74.037	15.76	15.76	8.02	0.192
46	5.894	5.353	12.57	73.949	15.76	15.76	8.02	0.192
47	5.906	5.365	12.84	73.067	15.76	15.76	8.02	0.192
48	5.829	5.288	13.11	72.880	15.76	15.76	8.02	0.192
49	5.750	5.209	13.39	72.831	15.76	15.76	8.02	0.192
50	5.726	5.185	13.66	72.417	15.76	15.76	8.02	0.192
51	5.716	5.175	13.93	71.920	15.76	15.76	8.02	0.192
52	5.686	5.145	14.21	71.750	15.76	15.76	8.02	0.192
53	5.676	5.135	14.48	69.991	15.76	15.76	8.02	0.192
54	5.619	5.078	14.75	68.702	15.76	15.76	8.02	0.192
55	5.634	5.093	15.03	68.108	15.76	15.76	8.02	0.192
56	5.626	5.085	15.30	67.672	15.76	15.76	8.02	0.192
57	5.608	5.067	15.57	67.624	15.76	15.76	8.02	0.192
58	5.599	5.058	15.85	67.557	15.76	15.76	8.02	0.192
59	5.593	5.052	16.12	65.163	15.76	15.76	8.02	0.192
60	6.562	6.021	16.39	63.463	15.76	15.76	8.02	0.192
61	5.586	5.045	16.67	62.898	15.76	15.76	8.02	0.192
62	5.580	5.039	16.94	62.570	15.76	15.76	8.02	0.192
63	5.554	5.013	17.21	61.474	15.76	15.76	8.02	0.192
64	5.553	5.012	17.49	61.177	15.76	15.76	8.02	0.192
65	5.563	5.022	17.76	60.812	15.76	15.76	8.02	0.192
66	5.550	5.009	18.03	60.775	15.76	15.76	8.02	0.192
67	5.501	4.960	18.31	60.501	15.76	15.76	8.02	0.192
68	5.420	4.879	18.58	58.628	15.76	15.76	8.02	0.192
69	5.456	4.915	18.85	58.387	15.76	15.76	8.02	0.192
70	5.410	4.869	19.13	57.740	15.76	15.76	8.02	0.192
71	5.465	4.924	19.40	57.119	15.76	15.76	8.02	0.192
72	5.662	5.121	19.67	56.791	15.76	15.76	8.02	0.192
73	5.570	5.029	19.95	56.051	15.76	15.76	8.02	0.192
74	5.480	4.939	20.22	54.724	15.76	15.76	8.02	0.192
75	5.556	5.015	20.49	54.551	15.76	15.76	8.02	0.192

76	5.545	5.004	20.77	53.604	15.76	15.76	8.02	0.192
77	5.554	5.013	21.04	53.393	15.76	15.76	8.02	0.192
78	5.641	5.100	21.31	52.941	15.76	15.76	8.02	0.192
79	5.632	5.091	21.58	52.675	15.76	15.76	8.02	0.192
80	5.598	5.057	21.86	52.391	15.76	15.76	8.02	0.192
81	5.623	5.082	22.13	51.622	15.76	15.76	8.02	0.192
82	5.585	5.044	22.40	51.307	15.76	15.76	8.02	0.192
83	5.582	5.041	22.68	50.395	15.76	15.76	8.02	0.192
84	5.571	5.030	22.95	48.562	15.76	15.76	8.02	0.192
85	5.551	5.010	23.22	48.011	15.76	15.76	8.02	0.192
86	5.630	5.089	23.50	47.019	15.76	15.76	8.02	0.192
87	5.583	5.042	23.77	46.531	15.76	15.76	8.02	0.192
88	5.527	4.986	24.04	45.695	15.76	15.76	8.02	0.192
89	5.682	5.141	24.32	45.425	15.76	15.76	8.02	0.192
90	5.837	5.296	24.59	45.002	15.76	15.76	8.02	0.192
91	5.909	5.368	24.86	44.151	15.76	15.76	8.02	0.192
92	5.733	5.192	25.14	43.446	15.76	15.76	8.02	0.192
93	5.751	5.210	25.41	43.421	15.76	15.76	8.02	0.192
94	5.673	5.132	25.68	42.864	15.76	15.76	8.02	0.192
95	5.661	5.120	25.96	42.783	15.76	15.76	8.02	0.192
96	5.772	5.231	26.23	41.328	15.76	15.76	8.02	0.192
97	5.744	5.203	26.50	39.056	15.76	15.76	8.02	0.192
98	5.728	5.187	26.78	38.801	15.76	15.76	8.02	0.192
99	5.693	5.152	27.05	38.157	15.76	15.76	8.02	0.192
100	5.665	5.124	27.32	37.205	15.76	15.76	8.02	0.192
101	5.790	5.249	27.60	36.830	15.76	15.76	8.02	0.192
102	5.881	5.340	27.87	36.817	15.76	15.76	8.02	0.192
103	5.832	5.291	28.14	34.485	15.76	15.76	8.02	0.192
104	5.988	5.447	28.42	34.012	15.76	15.76	8.02	0.192
105	6.019	5.478	28.69	33.988	15.76	15.76	8.02	0.192
106	6.811	6.270	28.96	33.384	15.76	15.76	8.02	0.192
107	6.308	5.767	29.23	31.672	15.76	15.76	8.02	0.192
108	6.337	5.796	29.51	31.231	15.76	15.76	8.02	0.192
109	6.281	5.740	29.78	30.299	15.76	15.76	8.02	0.192
110	6.201	5.660	30.05	29.826	15.76	15.76	8.02	0.192
111	6.379	5.838	30.33	29.397	15.76	15.76	8.02	0.192
112	6.285	5.744	30.60	29.153	15.76	15.76	8.02	0.192
113	6.384	5.843	30.87	28.510	15.76	15.76	8.02	0.192
114	6.674	6.133	31.15	28.152	15.76	15.76	8.02	0.192
115	6.524	5.983	31.42	27.527	15.76	15.76	8.02	0.192
116	6.921	6.380	31.69	27.242	15.76	15.76	8.02	0.192
117	6.725	6.184	31.97	26.529	15.76	15.76	8.02	0.192
118	6.930	6.389	32.24	26.070	15.76	15.76	8.02	0.192
119	6.891	6.350	32.51	25.242	15.76	15.76	8.02	0.192
120	7.137	6.596	32.79	25.034	15.76	15.76	8.02	0.192
121	7.326	6.785	33.06	24.686	15.76	15.76	8.02	0.192
122	7.538	6.997	33.33	24.432	15.76	15.76	8.02	0.192
123	7.597	7.056	33.61	23.654	15.76	15.76	8.02	0.192

124	7.775	7.234	33.88	23.592	15.76	15.76	8.02	0.192
125	7.958	7.417	34.15	23.168	15.76	15.76	8.02	0.192
126	8.088	7.547	34.43	22.724	15.76	15.76	8.02	0.192
127	8.704	8.163	34.70	21.914	15.76	15.76	8.02	0.192
128	8.542	8.001	34.97	21.551	15.76	15.76	8.02	0.192
129	8.433	7.892	35.25	20.647	15.76	15.76	8.02	0.192
130	9.209	8.668	35.52	19.738	15.76	15.76	8.02	0.192
131	9.018	8.477	35.79	19.633	15.76	15.76	8.02	0.192
132	8.299	7.758	36.07	19.233	15.76	15.76	8.02	0.192
133	8.303	7.762	36.34	19.028	15.76	15.76	8.02	0.192
134	8.223	7.682	36.61	18.740	15.76	15.76	8.02	0.192
135	8.354	7.813	36.89	18.216	15.76	15.76	8.02	0.192
136	9.093	8.552	37.16	18.174	15.76	15.76	8.02	0.192
137	9.010	8.469	37.43	17.652	15.76	15.76	8.02	0.192
138	9.036	8.495	37.70	17.557	15.76	15.76	8.02	0.192
139	9.628	9.087	37.98	17.249	15.76	15.76	8.02	0.192
140	9.294	8.753	38.25	16.894	15.76	15.76	8.02	0.192
141	9.495	8.954	38.52	16.744	15.76	15.76	8.02	0.192
142	9.578	9.037	38.80	16.470	15.76	15.76	8.02	0.192
143	9.827	9.286	39.07	16.333	15.76	15.76	8.02	0.192
144	10.021	9.480	39.34	16.191	15.76	15.76	8.02	0.192
145	10.138	9.597	39.62	16.137	15.76	15.76	8.02	0.192
146	10.627	10.086	39.89	15.794	15.76	15.76	8.02	0.192
			40.00		15.76	15.76	8.02	
147	12.318	11.777	40.16	15.766	15.76	15.76	8.02	0.115
148	12.488	11.947	40.44	15.495	15.76	15.49	7.89	0.190
149	12.053	11.512	40.71	15.111	15.76	15.11	7.69	0.186
150	12.023	11.482	40.98	14.885	15.76	14.89	7.58	0.183
151	11.797	11.256	41.26	14.782	15.76	14.78	7.53	0.181
152	11.858	11.317	41.53	14.443	15.76	14.44	7.35	0.178
153	12.123	11.582	41.80	14.156	15.76	14.16	7.21	0.174
154	12.574	12.033	42.08	14.003	15.76	14.00	7.13	0.172
155	12.815	12.274	42.35	13.958	15.76	13.96	7.11	0.170
156	13.212	12.671	42.62	13.676	15.76	13.68	6.96	0.168
157	13.420	12.879	42.90	13.343	15.76	13.34	6.79	0.165
158	14.544	14.003	43.17	13.125	15.76	13.13	6.68	0.161
159	15.426	14.885	43.44	12.933	15.76	12.93	6.58	0.159
160	16.307	15.766	43.72	12.879	15.76	12.88	6.56	0.157
161	18.098	17.557	43.99	12.743	15.76	12.74	6.49	0.156
162	16.732	16.191	44.26	12.671	15.76	12.67	6.45	0.155
163	17.285	16.744	44.54	12.549	15.76	12.55	6.39	0.154
164	16.874	16.333	44.81	12.373	15.76	12.37	6.30	0.152
165	18.757	18.216	45.08	12.274	15.76	12.27	6.25	0.150
166	19.774	19.233	45.36	12.044	15.76	12.04	6.13	0.148
167	20.174	19.633	45.63	12.033	15.76	12.03	6.13	0.147
168	22.455	21.914	45.90	11.947	15.76	11.95	6.08	0.146
169	25.575	25.034	46.17	11.937	15.76	11.94	6.08	0.146
170	26.611	26.070	46.45	11.785	15.76	11.78	6.00	0.145

171	27.783	27.242	46.72	11.777	15.76	11.78	6.00	0.144
172	28.068	27.527	46.99	11.639	15.76	11.64	5.93	0.143
173	29.938	29.397	47.27	11.582	15.76	11.58	5.90	0.141
174	31.772	31.231	47.54	11.512	15.76	11.51	5.86	0.141
175	33.925	33.384	47.81	11.482	15.76	11.48	5.85	0.140
176	37.358	36.817	48.09	11.366	15.76	11.37	5.79	0.139
177	39.342	38.801	48.36	11.317	15.76	11.32	5.76	0.138
178	39.597	39.056	48.63	11.256	15.76	11.26	5.73	0.138
179	43.405	42.864	48.91	11.159	15.76	11.16	5.68	0.137
180	43.987	43.446	49.18	10.977	15.76	10.98	5.59	0.135
181	46.236	45.695	49.45	10.779	15.76	10.78	5.49	0.133
182	45.543	45.002	49.73	10.611	15.76	10.61	5.40	0.130
183	51.848	51.307	50.00	10.423	15.76	10.42	5.31	0.128
184	53.216	52.675	50.27	10.280	15.76	10.28	5.23	0.126
185	55.092	54.551	50.55	10.139	15.76	10.14	5.16	0.124
186	54.145	53.604	50.82	10.086	15.76	10.09	5.14	0.123
187	52.163	51.622	51.09	9.981	15.76	9.98	5.08	0.122
188	56.592	56.051	51.37	9.813	15.76	9.81	5.00	0.121
189	55.265	54.724	51.64	9.642	15.76	9.64	4.91	0.119
190	61.042	60.501	51.91	9.597	15.76	9.60	4.89	0.117
191	57.660	57.119	52.19	9.529	15.76	9.53	4.85	0.117
192	59.169	58.628	52.46	9.527	15.76	9.53	4.85	0.116
193	64.004	63.463	52.73	9.480	15.76	9.48	4.83	0.116
194	61.353	60.812	53.01	9.436	15.76	9.44	4.80	0.115
195	62.015	61.474	53.28	9.286	15.76	9.29	4.73	0.114
196	68.165	67.624	53.55	9.157	15.76	9.16	4.66	0.112
197	69.243	68.702	53.83	9.087	15.76	9.09	4.63	0.111
198	72.291	71.750	54.10	9.037	15.76	9.04	4.60	0.110
199	72.958	72.417	54.37	9.003	15.76	9.00	4.58	0.110
200	70.532	69.991	54.64	8.954	15.76	8.95	4.56	0.109
201	74.578	74.037	54.92	8.929	15.76	8.93	4.55	0.109
202	78.048	77.507	55.19	8.787	15.76	8.79	4.47	0.108
203	77.966	77.425	55.46	8.753	15.76	8.75	4.46	0.107
204	78.082	77.541	55.74	8.668	15.76	8.67	4.41	0.106
205	77.186	76.645	56.01	8.662	15.76	8.66	4.41	0.106
206	80.192	79.651	56.28	8.552	15.76	8.55	4.35	0.105
207	82.010	81.469	56.56	8.525	15.76	8.52	4.34	0.104
208	83.183	82.642	56.83	8.495	15.76	8.50	4.33	0.104
209	84.970	84.429	57.10	8.494	15.76	8.49	4.32	0.104
210	85.279	84.738	57.38	8.487	15.76	8.49	4.32	0.103
211	82.826	82.285	57.65	8.477	15.76	8.48	4.32	0.103
212	81.765	81.224	57.92	8.469	15.76	8.47	4.31	0.103
213	89.566	89.025	58.20	8.310	15.76	8.31	4.23	0.102
214	85.871	85.330	58.47	8.187	15.76	8.19	4.17	0.101
215	84.228	83.687	58.74	8.163	15.76	8.16	4.16	0.100
216	83.938	83.397	59.02	8.116	15.76	8.12	4.13	0.099
217	80.352	79.811	59.29	8.063	15.76	8.06	4.10	0.099
218	80.597	80.056	59.56	8.001	15.76	8.00	4.07	0.098

219	80.696	80.155	59.84	7.950	15.76	7.95	4.05	0.097
220	77.664	77.123	60.11	7.892	15.76	7.89	4.02	0.097
221	78.616	78.075	60.38	7.839	15.76	7.84	3.99	0.096
222	81.064	80.523	60.66	7.813	15.76	7.81	3.98	0.095
223	82.981	82.440	60.93	7.807	15.76	7.81	3.97	0.095
224	82.822	82.281	61.20	7.762	15.76	7.76	3.95	0.095
225	84.736	84.195	61.48	7.758	15.76	7.76	3.95	0.095
226	85.120	84.579	61.75	7.709	15.76	7.71	3.93	0.094
227	86.371	85.830	62.02	7.682	15.76	7.68	3.91	0.094
228	85.276	84.735	62.30	7.664	15.76	7.66	3.90	0.094
229	82.831	82.290	62.57	7.547	15.76	7.55	3.84	0.093
230	85.617	85.076	62.84	7.544	15.76	7.54	3.84	0.092
231	91.021	90.480	63.11	7.465	15.76	7.46	3.80	0.091
232	90.134	89.593	63.39	7.423	15.76	7.42	3.78	0.091
233	88.404	87.863	63.66	7.417	15.76	7.42	3.78	0.090
234	88.101	87.560	63.93	7.330	15.76	7.33	3.73	0.090
235	84.475	83.934	64.21	7.297	15.76	7.30	3.72	0.089
236	87.555	87.014	64.48	7.239	15.76	7.24	3.69	0.089
237	82.110	81.569	64.75	7.234	15.76	7.23	3.68	0.088
238	86.037	85.496	65.03	7.144	15.76	7.14	3.64	0.088
239	83.466	82.925	65.30	7.125	15.76	7.13	3.63	0.087
240	84.673	84.132	65.57	7.085	15.76	7.09	3.61	0.087
241	83.156	82.615	65.85	7.062	15.76	7.06	3.60	0.086
242	81.587	81.046	66.12	7.056	15.76	7.06	3.59	0.086
243	79.365	78.824	66.39	6.997	15.76	7.00	3.56	0.086
244	77.463	76.922	66.67	6.996	15.76	7.00	3.56	0.085
245	77.643	77.102	66.94	6.898	15.76	6.90	3.51	0.085
246	73.608	73.067	67.21	6.892	15.76	6.89	3.51	0.084
247	73.421	72.880	67.49	6.846	15.76	6.85	3.49	0.084
248	74.490	73.949	67.76	6.827	15.76	6.83	3.48	0.083
249	73.372	72.831	68.03	6.785	15.76	6.78	3.45	0.083
250	72.461	71.920	68.31	6.779	15.76	6.78	3.45	0.083
251	68.649	68.108	68.58	6.737	15.76	6.74	3.43	0.082
252	68.098	67.557	68.85	6.641	15.76	6.64	3.38	0.082
253	68.213	67.672	69.13	6.613	15.76	6.61	3.37	0.081
254	65.704	65.163	69.40	6.611	15.76	6.61	3.37	0.081
255	63.439	62.898	69.67	6.596	15.76	6.60	3.36	0.080
256	63.111	62.570	69.95	6.522	15.76	6.52	3.32	0.080
257	61.718	61.177	70.22	6.440	15.76	6.44	3.28	0.079
258	61.316	60.775	70.49	6.417	15.76	6.42	3.27	0.078
259	58.281	57.740	70.77	6.389	15.76	6.39	3.25	0.078
260	57.332	56.791	71.04	6.380	15.76	6.38	3.25	0.078
261	58.928	58.387	71.31	6.355	15.76	6.35	3.24	0.078
262	53.934	53.393	71.58	6.350	15.76	6.35	3.23	0.077
263	52.932	52.391	71.86	6.321	15.76	6.32	3.22	0.077
264	53.482	52.941	72.13	6.296	15.76	6.30	3.21	0.077
265	50.936	50.395	72.40	6.281	15.76	6.28	3.20	0.077
266	49.103	48.562	72.68	6.270	15.76	6.27	3.19	0.076

267	48.552	48.011	72.95	6.254	15.76	6.25	3.18	0.076
268	47.560	47.019	73.22	6.207	15.76	6.21	3.16	0.076
269	47.072	46.531	73.50	6.184	15.76	6.18	3.15	0.075
270	45.966	45.425	73.77	6.173	15.76	6.17	3.14	0.075
271	44.692	44.151	74.04	6.133	15.76	6.13	3.12	0.075
272	43.324	42.783	74.32	6.127	15.76	6.13	3.12	0.075
273	43.962	43.421	74.59	6.107	15.76	6.11	3.11	0.075
274	41.869	41.328	74.86	6.022	15.76	6.02	3.07	0.074
275	38.698	38.157	75.14	6.021	15.76	6.02	3.07	0.073
276	37.746	37.205	75.41	6.009	15.76	6.01	3.06	0.073
277	37.371	36.830	75.68	5.999	15.76	6.00	3.05	0.073
278	35.026	34.485	75.96	5.983	15.76	5.98	3.05	0.073
279	34.529	33.988	76.23	5.970	15.76	5.97	3.04	0.073
280	34.553	34.012	76.50	5.910	15.76	5.91	3.01	0.072
281	32.213	31.672	76.78	5.881	15.76	5.88	2.99	0.072
282	30.840	30.299	77.05	5.843	15.76	5.84	2.98	0.071
283	29.694	29.153	77.32	5.838	15.76	5.84	2.97	0.071
284	30.367	29.826	77.60	5.796	15.76	5.80	2.95	0.071
285	29.051	28.510	77.87	5.796	15.76	5.80	2.95	0.071
286	28.693	28.152	78.14	5.783	15.76	5.78	2.94	0.071
287	27.070	26.529	78.42	5.779	15.76	5.78	2.94	0.070
288	25.783	25.242	78.69	5.767	15.76	5.77	2.94	0.070
289	24.973	24.432	78.96	5.757	15.76	5.76	2.93	0.070
290	24.195	23.654	79.23	5.744	15.76	5.74	2.92	0.070
291	24.133	23.592	79.51	5.740	15.76	5.74	2.92	0.070
292	25.227	24.686	79.78	5.696	15.76	5.70	2.90	0.070
293	23.709	23.168	80.05	5.683	15.76	5.68	2.89	0.069
294	23.265	22.724	80.33	5.660	15.76	5.66	2.88	0.069
295	22.092	21.551	80.60	5.644	15.76	5.64	2.87	0.069
296	21.188	20.647	80.87	5.610	15.76	5.61	2.86	0.069
297	20.279	19.738	81.15	5.570	15.76	5.57	2.84	0.068
298	19.569	19.028	81.42	5.529	15.76	5.53	2.82	0.068
299	19.281	18.740	81.69	5.493	15.76	5.49	2.80	0.067
300	18.715	18.174	81.97	5.478	15.76	5.48	2.79	0.067
301	18.193	17.652	82.24	5.475	15.76	5.47	2.79	0.067
302	17.790	17.249	82.51	5.462	15.76	5.46	2.78	0.067
303	17.435	16.894	82.79	5.447	15.76	5.45	2.77	0.066
304	17.011	16.470	83.06	5.427	15.76	5.43	2.76	0.066
305	16.678	16.137	83.33	5.425	15.76	5.42	2.76	0.066
306	16.335	15.794	83.61	5.419	15.76	5.42	2.76	0.066
307	16.036	15.495	83.88	5.418	15.76	5.42	2.76	0.066
308	15.652	15.111	84.15	5.383	15.76	5.38	2.74	0.066
309	15.323	14.782	84.43	5.368	15.76	5.37	2.73	0.066
310	14.984	14.443	84.70	5.365	15.76	5.36	2.73	0.065
311	14.697	14.156	84.97	5.353	15.76	5.35	2.73	0.065
312	14.499	13.958	85.25	5.345	15.76	5.35	2.72	0.065
313	14.217	13.676	85.52	5.340	15.76	5.34	2.72	0.065
314	13.884	13.343	85.79	5.296	15.76	5.30	2.70	0.065

315	13.666	13.125	86.07	5.291	15.76	5.29	2.69	0.065
316	13.474	12.933	86.34	5.288	15.76	5.29	2.69	0.064
317	13.284	12.743	86.61	5.249	15.76	5.25	2.67	0.064
318	13.090	12.549	86.89	5.231	15.76	5.23	2.66	0.064
319	12.914	12.373	87.16	5.210	15.76	5.21	2.65	0.064
320	12.585	12.044	87.43	5.209	15.76	5.21	2.65	0.063
321	12.478	11.937	87.70	5.203	15.76	5.20	2.65	0.063
322	12.326	11.785	87.98	5.192	15.76	5.19	2.64	0.063
323	12.180	11.639	88.25	5.187	15.76	5.19	2.64	0.063
324	11.907	11.366	88.52	5.185	15.76	5.18	2.64	0.063
325	11.700	11.159	88.80	5.175	15.76	5.18	2.63	0.063
326	11.518	10.977	89.07	5.152	15.76	5.15	2.62	0.063
327	11.320	10.779	89.34	5.145	15.76	5.15	2.62	0.063
328	11.152	10.611	89.62	5.141	15.76	5.14	2.62	0.063
329	10.964	10.423	89.89	5.135	15.76	5.14	2.61	0.063
330	10.821	10.280	90.16	5.132	15.76	5.13	2.61	0.063
331	10.680	10.139	90.44	5.124	15.76	5.12	2.61	0.062
332	10.522	9.981	90.71	5.121	15.76	5.12	2.61	0.062
333	10.354	9.813	90.98	5.120	15.76	5.12	2.61	0.062
334	10.183	9.642	91.26	5.100	15.76	5.10	2.60	0.062
335	10.070	9.529	91.53	5.093	15.76	5.09	2.59	0.062
336	10.068	9.527	91.80	5.091	15.76	5.09	2.59	0.062
337	9.977	9.436	92.08	5.089	15.76	5.09	2.59	0.062
338	9.698	9.157	92.35	5.085	15.76	5.08	2.59	0.062
339	9.544	9.003	92.62	5.082	15.76	5.08	2.59	0.062
340	9.470	8.929	92.90	5.078	15.76	5.08	2.59	0.062
341	9.328	8.787	93.17	5.067	15.76	5.07	2.58	0.062
342	9.203	8.662	93.44	5.058	15.76	5.06	2.58	0.062
343	9.066	8.525	93.72	5.057	15.76	5.06	2.57	0.062
344	9.028	8.487	93.99	5.052	15.76	5.05	2.57	0.062
345	9.035	8.494	94.26	5.045	15.76	5.05	2.57	0.062
346	8.851	8.310	94.54	5.044	15.76	5.04	2.57	0.061
347	8.728	8.187	94.81	5.042	15.76	5.04	2.57	0.061
348	8.657	8.116	95.08	5.041	15.76	5.04	2.57	0.061
349	8.604	8.063	95.36	5.039	15.76	5.04	2.57	0.061
350	8.491	7.950	95.63	5.030	15.76	5.03	2.56	0.061
351	8.380	7.839	95.90	5.029	15.76	5.03	2.56	0.061
352	8.348	7.807	96.17	5.022	15.76	5.02	2.56	0.061
353	8.250	7.709	96.45	5.015	15.76	5.02	2.55	0.061
354	8.205	7.664	96.72	5.013	15.76	5.01	2.55	0.061
355	8.085	7.544	96.99	5.013	15.76	5.01	2.55	0.061
356	8.006	7.465	97.27	5.012	15.76	5.01	2.55	0.061
357	7.871	7.330	97.54	5.010	15.76	5.01	2.55	0.061
358	7.838	7.297	97.81	5.009	15.76	5.01	2.55	0.061
359	7.780	7.239	98.09	5.004	15.76	5.00	2.55	0.061
360	7.666	7.125	98.36	4.986	15.76	4.99	2.54	0.061
361	7.626	7.085	98.63	4.960	15.76	4.96	2.53	0.061
362	7.685	7.144	98.91	4.939	15.76	4.94	2.51	0.060

363	7.603	7.062	99.18	4.924	15.76	4.92	2.51	0.060
364	7.537	6.996	99.45	4.915	15.76	4.92	2.50	0.060
365	7.439	6.898	99.73	4.879	15.76	4.88	2.48	0.060
366	7.387	6.846	100.00	4.869	15.76	4.87	2.48	0.059

A.3 Flood Hydrology

A.3.1 Analysis of Flood Discharge -UAF

Table 35: Regional Extreme (Maximum Instantaneous) Floods in 5 HSC

Year/Stations	Flood Discharge (m^3/s)				
	447	**505**	**620**	**630**	**647**
1977	935	17.3	780	2000	1370
1978	924	74.8	600	2690	1130
1979	935	3.53	275	2260	754
1980	941	10.5	1420	1720	1070
1981	895	17.3	520	1950	1080
1982	851	10	2100	1590	736
1983	740	34.4	980	1390	1240
1984	955	7.8	932	1550	1190
1985	2100	11.5	964	1660	1040
1986	884	6.43	660	1120	1700
1987	1090	5.48	446	2120	740
1988	953	7.4	836	3940	1170
1989	724	9	580	1570	1000
1990	1540	4.98	580	1410	990
1991	1160	6.1	310	1620	822
1992	1540	17.9	210	1210	939
1993	1980	7.8	1000	2370	1010
1994	1180	6.24	225	1130	1360
1995	1820	5.29	442	1420	1280
1996	2040	7.22	370	1960	1380
1997	1060	7.8	620	2270	1120
1998	1490	4.84	560	2960	2130
1999	1440	5.82	740	1750	1710
2000	1640	6.8	500	1810	940.00
2001	1430	5.4	400	2180	1570
2002	1520	10	370	1570	1440
2003	1490	8.6	285	2180	459.00
2004	1150	8	360	1310	1390
2005	1380	8.2	588	1180	1480
2006	1260	14.6	316	1140	1940

Table 36: Annual series of floods at IW-3 intake transposed from average UAF of 5 HSC

Year/Stations	UAF($m^3/s/km^2$)					Average UAF	5 HSC (m^3/s)
	447	**505**	**`620**	**630**	**647**		
Area(km^2)	4110	17	629	4920	2753		**Intake area =431.34 km^2**
1977	0.227	1.018	1.240	0.407	0.498	0.678	292.45
1978	0.225	4.400	0.954	0.547	0.410	1.307	563.76
1979	0.227	0.208	0.437	0.459	0.274	0.321	138.46
1980	0.229	0.618	2.258	0.350	0.389	0.768	331.27
1981	0.218	1.018	0.827	0.396	0.392	0.570	245.86
1982	0.207	0.588	3.339	0.323	0.267	0.945	407.62
1983	0.180	2.024	1.558	0.283	0.450	0.899	387.77
1984	0.232	0.459	1.482	0.315	0.432	0.584	251.90
1985	0.511	0.676	1.533	0.337	0.378	0.687	296.33
1986	0.215	0.378	1.049	0.228	0.618	0.498	214.81
1987	0.265	0.322	0.709	0.431	0.269	0.399	172.10
1988	0.232	0.435	1.329	0.801	0.425	0.644	277.78
1989	0.176	0.529	0.922	0.319	0.363	0.462	199.28
1990	0.375	0.293	0.922	0.287	0.360	0.447	192.81
1991	0.282	0.359	0.493	0.329	0.299	0.352	151.83
1992	0.375	1.053	0.334	0.246	0.341	0.470	202.73
1993	0.482	0.459	1.590	0.482	0.367	0.676	291.59
1994	0.287	0.367	0.358	0.230	0.494	0.347	149.67
1995	0.443	0.311	0.703	0.289	0.465	0.442	190.65
1996	0.496	0.425	0.588	0.398	0.501	0.482	207.91
1997	0.258	0.459	0.986	0.461	0.407	0.514	221.71
1998	0.363	0.285	0.890	0.602	0.774	0.583	251.47
1999	0.350	0.342	1.176	0.356	0.621	0.569	245.43
2000	0.399	0.400	0.795	0.368	0.341	0.461	198.85
2001	0.348	0.318	0.636	0.443	0.570	0.463	199.71
2002	0.370	0.588	0.588	0.319	0.523	0.478	206.18
2003	0.363	0.506	0.453	0.443	0.167	0.386	166.50
2004	0.280	0.471	0.572	0.266	0.505	0.419	180.73
2005	0.336	0.482	0.935	0.240	0.538	0.506	218.26
2006	0.307	0.859	0.502	0.232	0.705	0.521	224.73

A.3.2 Floods by Logarithmic (Excel) Analysis

Table 37: Transposed floods at intake from 5 HSC and fitting of Logarithmic (Excel) Formula

Year	Extreme Discharge (m3/s)	Extreme Discharge in descending order(m3/s)	Rank(m)	Return period T=(n+1)/m
1977	292.45	563.76	1	31.000
1978	563.76	407.62	2	15.500
1979	138.46	387.77	3	10.333
1980	331.27	331.27	4	7.750
1981	245.86	296.33	5	6.200
1982	407.62	292.45	6	5.167
1983	387.77	291.59	7	4.429
1984	251.90	277.78	8	3.875
1985	296.33	251.90	9	3.444
1986	214.81	251.47	10	3.100
1987	172.10	245.86	11	2.818
1988	277.78	245.43	12	2.583
1989	199.28	224.73	13	2.385
1990	192.81	221.71	14	2.214
1991	151.83	218.26	15	2.067

1992	202.73	214.81	16	1.938
1993	291.59	207.91	17	1.824
1994	149.67	206.18	18	1.722
1995	190.65	202.73	19	1.632
1996	207.91	199.71	20	1.550
1997	221.71	199.28	21	1.476
1998	251.47	198.85	22	1.409
1999	245.43	192.81	23	1.348
2000	198.85	190.65	24	1.292
2001	199.71	180.73	25	1.240
2002	206.18	172.10	26	1.192
2003	166.50	166.50	27	1.148
2004	180.73	151.83	28	1.107
2005	218.26	149.67	29	1.069
2006	224.73	138.46	30	1.033

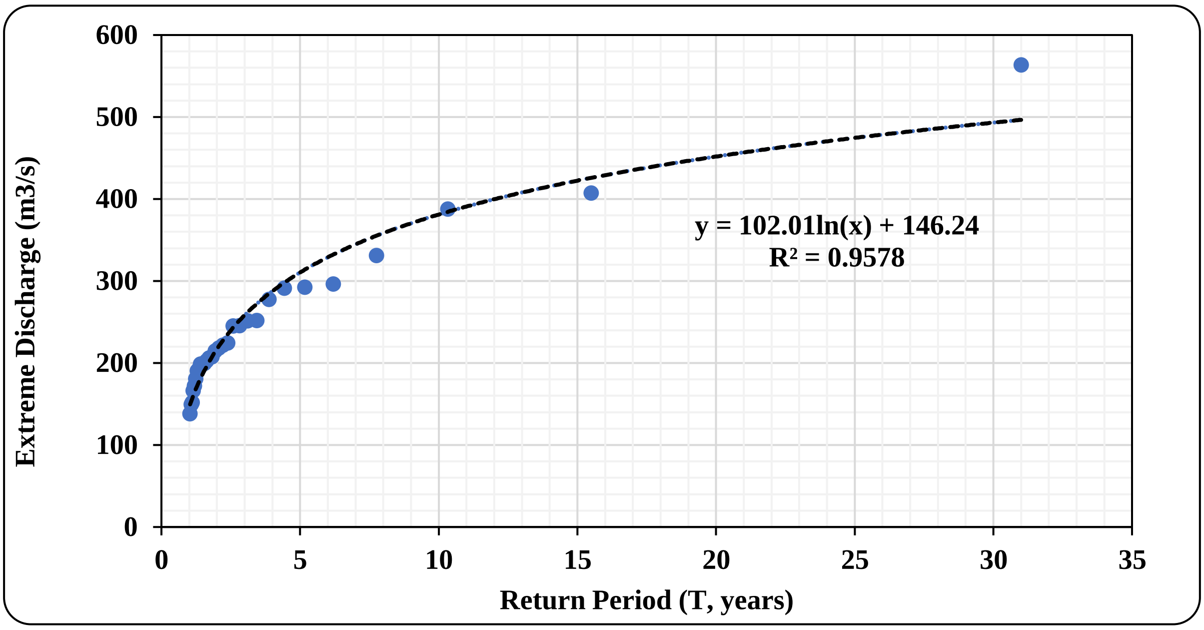


Figure 25: Transposed floods at intake from 5 HSC and fitting of Logarithmic (Excel) Formula

A.3.3 Floods by Gumbel Analysis

N=30 years			
X_{mean}=	242.67	Y_N=	0.5362
Stdeviation=	88.6416	S_N=	1.1124
Return Period	Y_T	K_T	**X_T**
10	2.250	1.540	**379.265**
50	3.901	3.025	**510.871**
100	4.600	3.653	**566.508**
200	5.295	4.278	**621.942**

Appendix-B (Design Calculation)

B. Design Calculation:

B.1 Design of Weir

(Source: Irrigation Engineering and Hydraulic Structures by: S.K. Garg)

Design discharge (Q_{100}) = 871.46 m^3/sec for 100 year return periods

Average R.L. of river bed level = 918 m

Lacey's perimeter Formula: $P=4.75\times\sqrt{Q}$ = 140.22 m for alluvial channel.

For boulder stage rivers this formula may not work properly, and designer has to judge considering the following points.

a. Average wetted width of river during flood

b. Formation of shoals upstream of weir

c. Energy dissipation devices downstream of weir

The value of effective length, L, used in Eq. 14 shall take into account the side contractions of the overflow due to crest piers and abutments. Accordingly, L shall be calculated using the relation (USBR, 1978)

$$L = L_e - 2(N.K_p + K_a)\, H_e$$

Description	Unit	Value	Remarks
Design Flood Discharge	m^3/s	871.47	Q_{100}: Inflow Design Flood
Crest Level of weir	m	923	
Approach Channel (Upstream Apron Elevation)	m	919	
Effective Head Over Crest during Flood Discharge	m	5.5	

Refer: Design Guidelines for Hydropower Structures by DoED

From economic point of view, Length of weir along with Under sluice is taken as 40m, same as existing structure. So, from DOED guidelines:

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

Considering the flood value and factor of safety we adopt L= 40m (including 10m for under sluice portion) as in the existing project.

Average Discharge intensity, q=Q/L= 871.46/40=21.78 m^3/s

Width of weir=2m (assumed)

R.L. of D/S HFL before weir construction = 926.5 m

R.L of U/S HFL = D/S HFL + afflux = 926.5 + 1.5 =928 m

R.L of weir-bay crest level =918 + 5 = 923 m

Scour Depth, $R=1.35\left(\frac{q^2}{f}\right)^{\frac{1}{3}} = 1.35\left(\frac{21.78^2}{3.5}\right)^{\frac{1}{3}} = 6.93$ m

Here, f=3.5 for coarse gravel (8-20mm)

Regime flow velocity, $v=\frac{q}{R}=\frac{21.78}{6.93}=3.14$ m/s

Approach velocity head, $h_a=\frac{v^2}{2\times g}=\frac{3.14^2}{2\times 9.81}=0.502$ m

U/S TEL=U/S HFL+h_a=928 + 0.502 =928.502 m

Discharge passing through the boulder weir is given by discharge formula for a narrow crested weir,
Effective Head Over Crest during Flood Discharge(H_0) = 928.502 − 923 = 5.502 m

$Q=1.9 \times [L-2(NK_p + K_a) H) \times H^{3/2}$

So, $Q = 1.9 \times (30 - 2 \times 0.02 \times 5.502) \times 5.502^{(3/2)} = 729.83\ m^3/s$

(Which is about 83% of Design flood)
Remaining 17% Flood pass through Under sluice

B.2 Design of Protection Works

Protection works are required on the upstream as well as on the downstream to obviate the possibility of scour hole travelling close to the u/s and d/s floor of the weir and to relieve any residual uplift pressure through the floor.

The arrangement consists of

(i) 500 mm gravel layer, and
(ii) 500 mm thick lean concrete
(iii) Row of RCC sheet pile

B.3 Checking for the stability of the weir

Table 38: Stability analysis of weir

SN	Contents	Symbol	Description	Forces (KN/m)		Lever arm (m)	Moments (KN)	
				V	H		M_O	M_R
1	Weight of Weir	W1	1/2×3.25×6.5×18	190.125		16.083		3057.844
		W2	2×6.5×18	234		14.000		3276.000
		W3	1/2×13×6.5×18	760.5		8.667		6591.000
2	Water Pressure	P1	1/2×3.25×4×9.81	63.765		17.167		1094.633
	(Assuming u/s WL up to crest)	P2	1/2×9.81×4×4		78.48	1.333	104.64	
	(Assuming d/s WL =1/2 of dam ht.)	P3	1/2×9.81×2×2		-19.62	0.667		13.080
3	Uplift Pressure	U1	9.81×2×18.25	-358.065		9.125	3267.343	
		U2	1/2×19.62×18.25	-179.033		12.167	2178.229	
Total sum=				711.2925	58.86		5550.212	14032.556

1) **Safety against sliding**

$$FOS=\mu\times\frac{\sum V}{\sum H}=0.65\times\frac{711.293}{58.86}=7.85>1\ (Ok)$$

2) **Safety against overturning**

$$FOS=\frac{\sum M_R}{\sum M_O}=\frac{14032.556}{5550.212}=2.53>1.5\ (Ok)$$

3) **Check for tension**

Distance of resultant from Toe:

$$X=\frac{\sum M}{\sum V}=\frac{\sum M_R-\sum M_O}{\sum V}=\frac{14032.556-5550.212}{711.293}=11.93$$

Eccentricity, $e=\frac{B}{2}-X=3.42$ m

$B/6=18.25/6=3.042<e$, Ok

4) **Safety against Principal Stress**

$$\sigma_{max}=\frac{\sum V}{B}\left(1+\frac{6e}{B}\right)$$
$$=\frac{711.293}{18.25}\left(1+\frac{6\times3.42}{18.25}\right)$$
$$=82.798\ kN/m^2<1800\ kN/m^2,\ Ok$$

B.4 Intake Design

Intake invert level = 1m above the under sluice crest level = 920 m amsl

(source: Design guidelines for headworks)

Design discharge for turbine $Q_d = 15.76\ m^3/s$

Increasing design discharge by 30% for intake discharge,

$Q_{intake} = 1.3 \times Q_d = 1.3 \times 15.76 = 20.488\ m^3/s$

Assume velocity through intake = 2 m/s

Cross section area required $= \dfrac{Q_{intake}}{\text{velocity}} = \dfrac{20.488}{2} = 10.244\ m^2$

Assume 3 intake openings, C/S area of each opening = 10.244/3 = 3.415 m^2

Assume height of intake = 2 m

Width of each opening = 3.415/2 = 1.707 m , So adopt B = 2m

Actual velocity through intake $= \dfrac{Q_{intake}}{3 \times \text{C/S area of each opening}} = \dfrac{20.488}{3 \times 2 \times 2} = 1.7\text{m/s} < 2\text{m/s Ok}$

Also, We have $Q = C \times A \times \sqrt{2gH}$

Here, C = 0.6 for sharp edge and roughly finished concrete

or, $\quad 20.488 = 0.6 \times 10.244 \times \sqrt{2 \times 9.81 \times H}$

On Solving, $\quad H = 0.566$ m

Entrance Loss through intake $= k \times \dfrac{v^2}{2g} = 0.5 \times \dfrac{1.7^2}{2 \times 9.81} = 0.0736$ m

Total width of intake

$= 3$ intake openings of $2m \times 2m$ + 2 piers of 1m each + 2 edge piers of 1m each

$= 3 \times 2 + 1 \times 2 + 1 \times 2$

$= 10$ m

Let, angle of inclination of trash rack with horizontal, $\alpha = 72°$ (3V: 1H)

Assume thickness of bar, t = 20 mm

Assume, Spacing of bars, a = 100 mm

Total submerged width of trash rack = width of intake = 10 m

Gross submerged area of trash rack $= \dfrac{10 \times 2}{\sin 72°} = 21.029\ m^2$

Percentage opening of trashrack $= \dfrac{a}{a + t} \times 100\% = \dfrac{100}{100 + 20} \times 100\% = 83.33\%$

Effective opening area of trashrack = 83.33% of 21.029 m^2 = 17.523 m^2

Approach Velocity, $V_o = \dfrac{Q_{intake}}{A_{effective}} = \dfrac{20.488}{17.523} = 1.169$ m/s in the range of (1 − 3) m/s, Ok

Head Loss through trash-rack is given by,

$$h_t = k \times \left(\frac{t}{a}\right)^{\frac{4}{3}} \times \frac{v^2}{2g} \times \sin\alpha$$

Here, k = 1.67 for rounded edge bars for trashrack

$$= 1.67 \times \left(\frac{20}{100}\right)^{\frac{4}{3}} \times \frac{1.169^2}{2 \times 9.81} \times \sin 72°$$

= 0.0129 m

Total Losses = Entrance loss + Head loss through thrashrack

$= 0.0736 + 0.0129 = 0.0865 \sim 0.1$ m

Now, Operating level of weir = Intake invert level + height of intake orifice + total losses

$= 920 + 2 + 0.1 = 922.1$m (at normal operation level)

Assuming FB as 0.9m, Crest level of diversion weir = 922.1 + 0.9 = 923 m

B.5 Design of Settling Basin

B.5.1 Geometry Calculation:

Discharge(Q) = $15.76 m^3/sec$

Particle size to be removed, $d_{limit} = 0.3$ mm

(for medium head scheme, $10 < h \leq 100m$, $d_{limit} = 0.2mm$ to $0.3mm$)

(Source: Baral, Sanjeeb: Fundamentals of Hydropower Engineering)

Specific gravity of particles = 2.65

Assume temperature of water = 25°C

Kinematic viscosity of water at 25° C = $0.91 \times 10^{-6}\ m^2/s$

Fall(Settling) Velocity, $V_f = 418\,(S-1) \times d^2 \times \frac{3t+70}{100}$; Where, t is in centigrade

$$\therefore V_f = 418\,(2.65-1)\,(0.3)^2 \frac{3 \times 25 + 70}{100} = 90 \text{ mm/sec}$$

Reynold′s number, $R_e = \frac{V_f\, d}{\vartheta} = \frac{90 \times 0.3}{0.91} = 29.67$

Reynold′s number lies between 1 to 1000, which indicates the flow is transistion,

$\therefore$ We have, $V_f = \sqrt{\frac{4}{3} \times \frac{g}{C_D} \times (S-1)D}$; Where, $C_D = \frac{24}{Re} + \frac{3}{\sqrt{Re}} + 0.34$

Now,

$$C_D = \frac{24}{29.67} + \frac{3}{\sqrt{29.67}} + 0.34 = 1.7$$

$$V_f = \sqrt{\frac{4}{3} \times \frac{9.81}{1.7} \times (2.65-1) \times \frac{0.3}{1000}} = 0.06171 \text{ m/sec} = 61.714 \text{ mm/sec}$$

Which is less than the initial taken value of 90 mm/sec $\therefore$ Repeat the procedure as,

$R_e = \frac{V_f\, d}{\vartheta}$; $C_D = \frac{24}{Re} + \frac{3}{\sqrt{Re}} + 0.34$; $V_f = \sqrt{\frac{4}{3} \times \frac{g}{C_D} \times (S-1)D}$

The detail calculation is shown in the table below:

Table 39: Iterative Calculation table for fall velocity

S. N	V_f (mm/sec)	Re	C_D	New V_f (mm/sec)
1	61.714	20.345	2.185	54.439
2	54.439	17.947	2.385	52.098
3	52.098	17.175	2.461	51.290
4	51.290	16.909	2.489	51.003
5	51.003	16.814	2.499	50.901
6	50.901	16.781	2.503	50.864
7	50.864	16.768	2.504	50.851
8	50.851	16.764	2.504	50.846
9	50.846	16.763	2.505	50.845
10	50.845	16.762	2.505	50.844
11	50.844	16.762	2.505	50.844

But this theoretical analysis is for truly spherical particles. Therefore, for actual shape sediment taking 65% of theoretical velocity = 50.844×0.65 mm/sec = 33.0486 mm/sec

Horizontal (Limiting) Flow velocity, $v = a \times \sqrt{d_{mm}} = 0.44 \times \sqrt{0.3} = 0.24$ m/s

Also, from particle size vs temperature table, $\omega = 33.26$ mm/s for (d = 0.3 mm , t = 25°C)

(Source: Zhurablov, 1975)

Increasing 25% discharge for flushing, Design discharge(Q_d) = $1.25 \times 15.76 = 19.70\ m^3/s$

Hence, taking horizontal component of velocity as 0.24m/sec, the length of the basin can be computed by using the following formula.

The length of settling basin, $L = \frac{HV}{\omega} = \frac{8 \times 0.24}{0.03305} = 58.102 \text{ m}$

Width of settling basin (B) is calculated from the computed horizontal component of velocity and given discharge,

$$Q = V \times A = 0.24 \times B \times H = 0.24 \times B \times 8$$

or, $19.70 = 0.24 \times B \times 8$

$$\therefore B = 10.263 \text{ m}$$

Checking using M.A. Velikanov's equation:

$$L = \frac{\lambda^2 v^2 \left(\sqrt{H} - 0.2\right)^2}{7.51\omega^2}$$

$$= \frac{1.2^2 \times 0.24^2 \times \left(\sqrt{8} - 0.2\right)^2}{7.51 \times 0.03305^2} = 69.854 \text{ m} ; \text{ So, Adopt Length(L)} = 70 \text{ m}$$

Here, for 95% removal efficiency, $\lambda = 1.2$ from Velikanov's relation curve, $W = f(\lambda)$

Considering the case of turbulent flow,

$$L = \frac{H^{\frac{3}{2}}V}{\omega\sqrt{H} - 0.132V} = \frac{8^{\frac{3}{2}} \times 0.24}{0.03305\sqrt{8} - 0.132 \times 0.24} = 87.273 \text{ m} ; \text{So, Adopt Length(L)} = 88 \text{ m}$$

Inlet transition: Set horizontal expansion ratio 1:10 and vertical expansion ratio 1:4

Outlet transition: Set horizontal expansion ratio 1:3 and vertical expansion ratio 1:2

Floor slope: 1:3

B.5.2 Sludge Depth Calculation:

Sediment concentration (C) = 2000 mg/l = 2 kg/m^3

Density of sediment = 2600 kg/m^3

Assume, Frequency of sediment flushing = 12 hrs.

$\therefore$ Sediment Load $= Q \times T \times C = 19.70 \times 12 \times 3600 \times 2 = 1702080$ kg

Assume, Packing Factor = 0.5

$\therefore$ Volume of sediment $= \frac{\text{Sediment Load}}{\text{Density of sediment} \times \text{PF}} = \frac{1702080}{2600 \times 0.5} = 1309.292 \text{ m}^3$

Also, We have Plan area $= 88 \times 10.26 = 902.880\ m^2$

So, Height of sediment storage $= \frac{1309.292}{902.880} = 1.450$ m

B.5.3 Checking for Efficiency:

1. Using Camp's Graph:

According to Camp's, efficiency depends upon two dimensionless parameters namely $\frac{\omega}{u_*}$ and $\frac{\omega A_s}{Q}$

Shear velocity, $u_* = \sqrt{gRS_e}$ or $u_* = \frac{0.042 \times v_m}{R^{\frac{1}{6}}}$

Here, $A = H \times B = 8 \times 10.3 = 82.4 \text{ m}^2$

$$P = 2H + B = 2 \times 8 + 10.3 = 26.3 \text{ m}$$
$$R = \frac{A}{P} = \frac{82.4}{26.3} = 3.133 \text{ m}$$
$$\therefore u_* = \frac{0.042 \times v_m}{R^{\frac{1}{6}}} = \frac{0.042 \times 0.24}{3.133^{\frac{1}{6}}} = 0.00833$$
$$\text{Now, } \frac{\omega}{u_*} = \frac{0.03305}{0.00833} = 3.96 \ \& \ \frac{\omega \times A_s}{Q} = \frac{0.03305 \times 88 \times 10.3}{19.70} = 1.521$$
$$\therefore \text{From Camp's Graph, Efficiency } (\eta) = 100\%$$

2. Using Vetter's equation:

$$\eta = 1 - e^{-\frac{\omega A_s}{Q}} = 1 - e^{-1.521} = 0.7815 = 78.15\%$$

3. Using Hazen's equation:

$$\eta = \left(1 + m\frac{\omega}{v_o}\right)^{-\frac{1}{m}} = \left(1 + 0.17 \times \frac{0.03305}{0.24}\right)^{-\frac{1}{0.17}} = 87.27\%$$

B.6 Design of Tunnel

Design Discharge	15.76 m^3/s
Turbine Efficiency	90%
Generator Efficiency	97%
Transformer Efficiency	98%
Length of Tunnel	2934 m
Manning's Coefficient	0.02
Life of Project	40 years
Interest Rate	10%

SN	Description	Quantity	Unit	Rate
1	Tunnel Excavation	1	m^3	
2	RCC	1	m^3	
3	Steel Fiber with Plain Shotcrete	1	m^3	
4	PCC	1	m^3	
5	Wire Mesh	1	m^3	
6	Wire Mesh Shotcreting	0.1	m	
7	Bolt		m	
8	Invert RCC	0.3	m	
9	Level PCC	0.05	m	
10	Reinforcement	1	m	
Here D-Shaped Tunnel is proposed for project				

Inside Area = Area of semicircular portion + Area of rectangular portion
Wetted Perimeter = $\pi D/2 + 2D$
Hydraulic Radius = Inside Radius/ Wetted Perimeter
Tunnel Area = Total area of excavation of tunnel area including additional area for shotcrete and RCC
$$= \frac{\pi(D + 2 \times FS)^2}{8} + \frac{(D + 2 \times FS)(D + \text{Invert lvl} + \text{Level PCC})}{2}$$
Where FS = Fibre Shotcrete
Excavation cost = Tunnel area $\times$ Tunnel Excavation Rate
Cost of bolting + Shotcrete = Perimeter of tunnel $\times$ Length $\times$ Rate
Cost of RCC = D $\times$ Length of Tunnel $\times$ Corresponding Rate

Excavation Cost	5000	NPR/ m^3
Steel Fiber Cost	50,000	NPR/ m^3
Shotcrete Thickness	5	cm
Concrete Thickness at Invert	30	cm
Cost of Concrete (M25)	20000	NPR/ m^3
Concrete Lining	30	cm
Rock Bolt Length	2*D/3	
Bolt Cost	1500	NPR/m
Reinforcement	50	kg/ m^3 of concrete
Cost of Reinforcement	140	NPR/kg
Cost of Formwork	619	NPR per m^2

B.6.1 Tunnel Cost Optimization

Table 40: Tunnel Cost Optimization

		Length of Tunnel	m	2934										
		Design Discharge	m^3/s	15.760	15.760	15.760	15.760	15.760	15.760	15.760	15.760	15.760	15.760	15.760
		Tunnel Diameter	m	2.800	2.900	3.000	3.100	3.200	3.300	3.400	3.500	3.600	3.700	3.800
		CSA	m^2	6.999	7.508	8.034	8.579	9.141	9.721	10.319	10.935	11.569	12.221	12.890
		Wetted Perimeter	m	9.998	10.355	10.712	11.069	11.426	11.783	12.141	12.498	12.855	13.212	13.569
		Hydraulic Radius	m	0.700	0.725	0.750	0.775	0.800	0.825	0.850	0.875	0.900	0.925	0.950
		Flow velocity	m/s	2.252	2.099	1.962	1.837	1.724	1.621	1.527	1.441	1.362	1.290	1.223
		Head loss due to friction	$m*Q^2$	5.416	4.545	3.836	3.256	2.778	2.382	2.052	1.775	1.542	1.344	1.176
		Bend loss	$m*Q^2$	0.026	0.022	0.020	0.017	0.015	0.013	0.012	0.011	0.009	0.008	0.008
		Entry loss	$m*Q^2$	0.041	0.036	0.031	0.028	0.024	0.021	0.019	0.017	0.015	0.014	0.012
		Exit loss	$m*Q^2$	0.258	0.225	0.196	0.172	0.152	0.134	0.119	0.106	0.095	0.085	0.076
		Total head loss	$m*Q^2$	5.742	4.828	4.083	3.473	2.969	2.551	2.201	1.908	1.661	1.451	1.272
		Constant	$m*Q^2$	0.324	0.268	0.224	0.188	0.159	0.135	0.115	0.098	0.085	0.073	0.063
		Maximum Capacity Loss	MW	1.266	1.050	0.877	0.736	0.621	0.527	0.450	0.385	0.332	0.286	0.248
Month	Days	Available Flow(m^3/s)	Actual Discharge (m^3/s)	**Energy (GWh)**										
Jan	31	6.259	6.259	0.035	0.030	0.025	0.021	0.018	0.016	0.014	0.012	0.010	0.009	0.008
Feb	28	5.267	5.267	0.019	0.016	0.014	0.012	0.010	0.008	0.007	0.006	0.006	0.005	0.004
Mar	31	5.008	5.008	0.018	0.015	0.013	0.011	0.009	0.008	0.007	0.006	0.005	0.005	0.004
Apr	30	5.655	5.655	0.025	0.021	0.018	0.015	0.013	0.011	0.010	0.008	0.007	0.006	0.006
May	31	8.933	8.933	0.103	0.086	0.073	0.062	0.053	0.046	0.039	0.034	0.030	0.026	0.023
Jun	30	24.634	15.760	0.547	0.460	0.389	0.331	0.283	0.243	0.210	0.182	0.158	0.138	0.121

Jul	31	68.890	15.760	0.565	0.475	0.402	0.342	0.292	0.251	0.217	0.188	0.163	0.143	0.125
Aug	31	83.210	15.760	0.565	0.475	0.402	0.342	0.292	0.251	0.217	0.188	0.163	0.143	0.125
Sep	30	57.977	15.760	0.547	0.460	0.389	0.331	0.283	0.243	0.210	0.182	0.158	0.138	0.121
Oct	31	25.411	15.760	0.565	0.475	0.402	0.342	0.292	0.251	0.217	0.188	0.163	0.143	0.125
Nov	30	12.125	12.125	0.249	0.209	0.177	0.151	0.129	0.111	0.095	0.083	0.072	0.063	0.055
Dec	31	7.941	7.941	0.072	0.061	0.051	0.044	0.037	0.032	0.028	0.024	0.021	0.018	0.016
Wet Season Energy Loss (GWh)				3.213	2.701	2.285	1.943	1.661	1.427	1.232	1.068	0.929	0.812	0.712
Dry Season Energy Loss (GWh)				0.098	0.082	0.070	0.059	0.051	0.043	0.037	0.033	0.028	0.025	0.022
Total Energy Loss (GWh)				3.310	2.783	2.354	2.002	1.712	1.471	1.269	1.100	0.957	0.837	0.734
Total Energy Lost Cost (In Million NRs)				16.242	13.656	11.550	9.823	8.398	7.215	6.227	5.398	4.698	4.105	3.599

B.6.2 Calculation of Annual Cost

No. of Bolts Longitudinal = 1468 for all diameters
Bolting cost = 2.201 million NRs. for all diameters

Table 41: Calculation of Annual Cost of Tunnel

Diameter (m)	**Inside Area (m2)**	**Perimeter (m)**	**Area (m2)**	**Excavation Cost (In million Nrs)**	**No of Bolts in Perimeter**	**Wiremesh Cost (In million Nrs)**	**Concrete for Inv. RCC (In million Nrs)**	**Shortcreting Cost (In million Nrs)**	**Rebar Cost (In million Nrs)**	**Total Cost (In million Nrs)**	**Annual Cost (In million Nrs)**
2.4	5.142	8.570	7.267	106.604	5	13.578	42.250	1.232	0.176	166.040	16.969
2.5	5.579	8.927	7.660	112.366	5	14.733	44.010	1.284	0.183	174.776	17.862
2.6	6.035	9.284	8.060	118.243	6	15.935	45.770	1.335	0.191	183.674	18.772
2.7	6.508	9.641	8.469	124.235	6	17.184	47.531	1.386	0.198	192.735	19.698
2.8	6.999	9.998	8.885	130.343	6	18.481	49.291	1.438	0.205	201.958	20.640
2.9	7.508	10.355	9.309	136.566	6	19.824	51.052	1.489	0.213	211.344	21.599
3.0	8.034	10.712	9.741	142.904	6	21.215	52.812	1.540	0.220	220.892	22.575
3.1	8.579	11.069	10.181	149.357	7	22.653	54.572	1.592	0.227	230.602	23.568
3.2	9.141	11.426	10.629	155.925	7	24.138	56.333	1.643	0.235	240.475	24.576
3.3	9.721	11.783	11.084	162.609	7	25.670	58.093	1.694	0.242	250.509	25.602
3.4	10.319	12.141	11.548	169.408	7	27.250	59.854	1.746	0.249	260.707	26.644

3.5	10.935	12.498	12.019	176.322	7	28.876	61.614	1.797	0.257	271.066	27.703
3.6	11.569	12.855	12.498	183.351	7	30.550	63.374	1.848	0.264	281.588	28.778
3.7	12.221	13.212	12.985	190.496	8	32.270	65.135	1.900	0.271	292.273	29.870
3.8	12.890	13.569	13.480	197.756	8	34.038	66.895	1.951	0.279	303.120	30.979
3.9	13.578	13.926	13.983	205.131	8	35.853	68.656	2.002	0.286	314.129	32.104
4.0	14.283	14.283	14.494	212.621	8	37.716	70.416	2.054	0.293	325.300	33.246
4.1	15.006	14.640	15.012	220.226	8	39.625	72.176	2.105	0.301	336.634	34.404
4.2	15.747	14.997	15.538	227.947	8	41.582	73.937	2.156	0.308	348.130	35.579
4.3	16.506	15.354	16.072	235.783	9	43.585	75.697	2.208	0.315	359.789	36.770
4.4	17.282	15.711	16.614	243.734	9	45.636	77.458	2.259	0.323	371.610	37.978
4.5	18.077	16.068	17.164	251.800	9	47.734	79.218	2.311	0.330	383.593	39.203
4.6	18.889	16.425	17.722	259.981	9	49.879	80.978	2.362	0.337	395.738	40.444
4.7	19.719	16.783	18.288	268.278	9	52.071	82.739	2.413	0.345	408.046	41.702
4.8	20.568	17.140	18.861	276.690	10	54.311	84.499	2.465	0.352	420.517	42.977
4.9	21.433	17.497	19.442	285.217	10	56.597	86.260	2.516	0.359	433.149	44.268

B.6.3 Tunnel Optimization Chart Value

Table 42: Tunnel Optimization chart value

Tunnel Diameter(m)	2.80	2.90	3.00	3.10	3.20	**3.30**	3.40	3.50	3.60	3.70	3.80
Cost of Energy Lost (Million NRs.)	16.24	13.66	11.55	9.82	8.40	**7.21**	6.23	5.40	4.70	4.10	3.60
Cost of Tunnel (Million NRs.)	20.64	21.60	22.58	23.57	24.58	**25.60**	26.64	27.70	28.78	29.87	30.98
Total Cost (Million NRs.)	36.88	35.26	34.13	33.39	32.97	**32.82**	32.87	33.10	33.48	33.97	34.58

B.7 Design of Surge Tank

Design Discharge, Q = 15.760 m^3/s

Considering 100% shotcrete			
Gross Head of Project	H_g	63.000	m
Net Head	Hn	60.000	m
Velocity of flow in headrace tunnel	v_o	1.621	m/s
Head loss in Headrace Tunnel	H_f	2.551	m
Total Length of pressure tunnel	L	2934.000	m
Cross Sectional Area of pressure tunnel	At	9.721	m^2
Diameter of pressure tunnel	Dt	3.300	m
Wetted Perimeter of pressure tunnel	P	11.783	m
Hydraulic Radius of pressure tunnel	R	0.825	m
Preliminary Area of Surge Shaft	Ast	40.127	m^2
Diameter of surge shaft	Dst	7.148	m
Using Factor of Safety for Ast		1.500	
Area of Surge Shaft	Astm	60.191	m^2
Diameter of surge shaft	Dstm	8.754	m
Adopt Diameter	Dst	9.000	m
Provided Area of Surge Tank	$A_{st,provided}$	63.617	m^2

Check for A_{st} min. by Thoma Criteria

$$A_{st} = \frac{A_t L_t V_t^2}{2gh_f\left(H_g - h_f\right)} = 24.772\ m^2 < Ast, \text{provided (OK!!)}$$

Time period of oscillation	T	277.971
Time period of half cycle	T/2	138.985
Maximum surge height without friction	z_{max}	10.959
	Po	0.233
Upsurge Head	$z_{upsurge}$	9.324
Downsurge Head	$z_{downsurge}$	-3.857

Since $Z_{downsurge}$ is negative, the water level in the surge tank will be below the reservoir level
Since the upsurge and downsurge height is small the given diameter of the surge tank is adopted in the further calculation

Normal Water Level at intake	NWL	922.100	m asl
Due to head loss, static level at surge tank		919.549	m asl

Downsurge level considering maximum downsurge at surge shaft	918.243	m
Upsurge Level considering maximum upsurge	931.424	m

Provide Freeboard:		
Above Upsurge	2.000	m

So, Final Diameter of surge tank=9m and Height =16 m

B.8 Penstock Pipe Diameter Optimization

Design parameters

Design discharge	5.25	m3/s
Overall efficiency of powerplant	86.5%	
Total gross head, Hg	63.00	m

Pipe parameters

Density of pipe	7850	Kg/m3
Cost of pipe/kg	180	Rs.
Joint Efficiency	90%	
Dynamic Head	20%	of static head

Other data

Minimum thickness for corrosion	1.5	mm
Life of Project	40	yrs
Interest rate	10%	
Unit Price of Electricity for dry season (Dec-Mar)	8.4	Rs/kwh
Unit Price of Electricity for wet season (Apr-Nov)	4.8	Rs/kwh

Design Discharge (m3/s)	5.250	5.250	5.250	5.250	5.250	5.250	5.250	5.250	5.250	5.250	5.250	5.250
Penstock Diameter (m)	**1.500**	**1.600**	**1.700**	**1.800**	**1.900**	**2.000**	**2.100**	**2.200**	**2.300**	**2.400**	**2.500**	**2.600**
Cross Sectional Area (m2)	1.767	2.011	2.270	2.545	2.835	3.142	3.464	3.801	4.155	4.524	4.909	5.309
Velocity	2.971	2.611	2.313	2.063	1.852	1.671	1.516	1.381	1.264	1.161	1.070	0.989

Reynold's number (x10^6)	4.447	4.169	3.924	3.706	3.511	3.336	3.177	3.032	2.901	2.780	2.668	2.566
Relative. Roughness Height	0.00017	0.00016	0.00015	0.00014	0.00014	0.00013	0.00012	0.00012	0.00011	0.00011	0.00010	0.00010
Friction Factor(From Moody Chart)	0.015	0.015	0.015	0.015	0.015	0.015	0.015	0.015	0.015	0.015	0.015	0.015
Static head(m)	60	60	60	60	60	60	60	60	60	60	60	60
Allowable design stress of pipe material (kg/cm2)	1400	1400	1400	1400	1400	1400	1400	1400	1400	1400	1400	140000000
Dynamic Head(m)	12	12	12	12	12	12	12	12	12	12	12	12
Net design head(m)	72	72	72	72	72	72	72	72	72	72	72	72
Pressure (P), kg/cm2	7.200	7.200	7.200	7.200	7.200	7.200	7.200	7.200	7.200	7.200	7.200	7.200
Thickness (m)	0.006	0.006	0.006	0.007	0.007	0.007	0.008	0.008	0.008	0.008	0.009	0.009
Volume/m	0.027	0.031	0.034	0.038	0.042	0.046	0.050	0.054	0.059	0.063	0.068	0.073
Density of Steel (kg/m3)	7850	7850	7850	7850	7850	7850	7850	7850	7850	7850	7850	7850
Kg/m of steel	215.402	241.103	268.223	296.761	326.716	358.090	390.883	425.093	460.721	497.768	536.232	576.115
Cost of steel per kg (NRs)	180	180	180	180	180	180	180	180	180	180	180	180
Cost in Million NRs	11.632	13.020	14.484	16.025	17.643	19.337	21.108	22.955	24.879	26.879	28.957	31.110
AWF	0.102	0.102	0.102	0.102	0.102	0.102	0.102	0.102	0.102	0.102	0.102	0.102
Annual Cost of Material (Million NRs)	1.189	1.331	1.481	1.639	1.804	1.977	2.158	2.347	2.544	2.749	2.961	3.181

		Penstock Diameter (m)	1.500	1.600	1.700	1.800	1.900	2.000	2.100	2.200	2.300	2.400	2.500	2.600
Month	**Days**	**Plant Discharge (m3/s)**	**Energy (MWh)**											
Jan	31	6.26	4.187	3.099	2.339	1.795	1.398	1.104	0.882	0.713	0.582	0.479	0.398	0.333

Feb	28	5.27	2.253	1.668	1.259	0.966	0.752	0.594	0.475	0.384	0.313	0.258	0.214	0.179
Mar	31	5.01	2.145	1.588	1.198	0.919	0.716	0.565	0.452	0.365	0.298	0.245	0.204	0.171
Apr	30	5.65	2.988	2.212	1.669	1.281	0.998	0.788	0.630	0.509	0.415	0.342	0.284	0.238
May	31	8.93	12.172	9.011	6.799	5.218	4.065	3.209	2.565	2.073	1.691	1.393	1.157	0.968
Jun	30	15.76	64.694	47.893	36.138	27.733	21.604	17.058	13.633	11.015	8.990	7.404	6.149	5.146
Jul	31	15.76	66.851	49.489	37.343	28.657	22.324	17.627	14.087	11.383	9.289	7.650	6.354	5.317
Aug	31	15.76	66.851	49.489	37.343	28.657	22.324	17.627	14.087	11.383	9.289	7.650	6.354	5.317
Sep	30	15.76	64.694	47.893	36.138	27.733	21.604	17.058	13.633	11.015	8.990	7.404	6.149	5.146
Oct	31	15.76	66.851	49.489	37.343	28.657	22.324	17.627	14.087	11.383	9.289	7.650	6.354	5.317
Nov	30	12.12	29.458	21.807	16.455	12.628	9.837	7.767	6.208	5.016	4.093	3.371	2.800	2.343
Dec	31	7.94	8.551	6.330	4.776	3.665	2.855	2.255	1.802	1.456	1.188	0.979	0.813	0.680
		Total energy loss (MWh)	391.695	289.968	218.799	167.908	130.804	103.279	82.540	66.693	54.429	44.826	37.227	31.154
		Total annual energy loss cost in Million (NRs)	3.229	2.390	1.803	1.384	1.078	0.851	0.680	0.550	0.449	0.369	0.307	0.257
		Total Annual Cost (Million NRs)	4.418	3.721	3.285	3.023	2.882	2.829	2.839	2.897	2.993	3.118	3.268	3.438

Table 43:Tabular representation of Cost Vs Diameter of Penstock

Penstock Diameter (m)	1.5	1.6	1.7	1.8	1.9	**2.0**	2.1	2.2	2.3	2.4	2.5	2.6
Annual Equivalent Cost of Pipe (Million NRs)	1.189	1.331	1.481	1.639	1.804	**1.977**	2.158	2.347	2.544	2.749	2.961	3.181
Total annual energy loss cost in Million (NRs)	3.229	2.390	1.803	1.384	1.078	**0.851**	0.680	0.550	0.449	0.369	0.307	0.257
Total Annual Cost (Million NRs)	4.418	3.721	3.285	3.023	2.882	**2.829**	2.839	2.897	2.993	3.118	3.268	3.438

B.9 Design of Francis Turbine:

Design discharge Q = 5.25 m^3/sec
Effective head H = 60 m
Power P = 2657.53 KW = 3562.37 HP

Specific speed $N_S = \frac{2400}{\sqrt{H}} = \frac{2400}{\sqrt{60}} = 309.84$ RPM

Synchronous speed $N = \frac{N_s \times H^{\frac{5}{4}}}{\sqrt{P}} = 866.87$ RPM

Number of poles $P = \frac{120 \times f}{N} = \frac{120 \times 50}{866.87} = 6.92$, $\therefore$ Adopt P = 8

Corrected synchronous speed $N = \frac{120 \times 50}{8} = 750$ RPM

Corrected specific speed $N_S' = \frac{750 \times \sqrt{3562.37}}{60^{\frac{5}{4}}} = 268.066$ RPM

Calculation of diameter of Francis Turbine:

$$\phi = 0.0197 N_S^{\frac{2}{3}} + 0.0275 = 0.0197 \times 268.066^{\frac{2}{3}} + 0.0275 = 0.846$$

Diameter $D = \frac{84.6 \times \phi \times \sqrt{H}}{N} = \frac{84.6 \times 0.846 \times \sqrt{60}}{750} = 0.739$ m

Setting of turbine:

$$H_S = H_A - H_V - \sigma \times H$$

Where, 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 = Thomas Cavitation Number
H = Net Effective Head in m

$$\sigma = 0.0318 \times \left(\frac{N_S}{100}\right)^2 = 0.0318 \times \left(\frac{268.066}{100}\right)^2 = 0.228 \text{ m}$$

H_S = 10.3 − 0.2 − 0.228X60 = −3.58 m

Hence, to avoid cavitation, the turbine must be set 3.58m below Tail Water Level

B.10 Dimensioning of Powerhouse

Machine hall:

center to center spacing between two turbine $= 5D + 2.5 = 5 \times 0.74 + 2.5 = 6.2$ m

Total length $= 2 \times (5D + 2.5) + \left(\frac{6}{2}\right) + \left(\frac{6}{2}\right) = 2 \times 6.2 + 3 + 3 = 18.4$ m

Width of hall , B $= 5D + 2.5 = 5 \times 0.74 + 2.5 = 6.2$ m , Adpot B = 8m

Height of superstructure $= 4.5\ (D + 1) = 4.5 \times (0.74 + 1) = 7.83$ m ~ 8m

Height of intermediate structure = (2 to 2.5) m = 2.5m (approximately)

Height of substructure = 4 m

Total height $= 8 + 2.5 + 4 = 14.5$ m

Erection and Control Bay:

Size of loading bay $= 6\ \text{m} \times 8\ \text{m}$; Size of office room $= 6\text{m} \times 8\text{m}$

Size of control bay $= 7.6\text{m} \times 8\text{m}$

Total Length of Powerhouse $= 18.6 + 6 + 6 + 7.6 = 38.2$, Adpot, L = 40m (extra for piers)

Total Width, B = 8m and Total Height = 15m

Appendix-C (Cost and Benefit Analysis)

Local Rates

Table 44: District rates for resources

S.N.	DESCRIPTION	RATE (Rs)	UNIT
1	Skilled Labor	1,090.00	MD
2	Unskilled Labour	790	MD
3	Coarse sand	1,870.00	m^3
4	Stone aggregate	1,897.50	m^3
5	Coarse aggregate 40mm down	1,650.00	m^3
6	Coarse aggregate 20mm down	1,815.00	m^3
7	Coarse aggregate 10mm down	1,650.00	m^3
8	Cement	17,000.00	MT
9	Reinforcement Bar	84,000.00	MT
10	Binding Wire	120	kg
11	Reinforcement Bar	91.5	kg
12	Diesel	101	ltr
13	Excavator	2,100.00	hr
14	Mixer	200	hr
15	Generator	150	hr
16	Plywood	946.1	m^3
17	Strut, bellies, etc	51,504.60	m^3
18	Nail, spike, etc	112.8	kg
19	Oiling of formwork	50	L.S.

Rate Analysis

Table 45: Schedule of Rates or Rate analysis of different works

S.N.	SOURCES	LEVEL/TYPE	UNIT	QUANTITY	RATE(NRs)	COST (NRs)	TOTAL COST (NRs)
1	**Earthwork in excavation for ordinary rock using mechanical means**						
i	Labour	Skilled		0.01	1,090.00	10.9	34.60
		Unskilled	Nos.	0.03	790	23.7	
ii	Materials						
	1. Diesel		ltr	0.75	101	75.75	75.75
iii	Equipment						
	1. Excavator		hr	0.07	2,100.00	147	147.00
iv	Sub total						257.35
v	Contractor's overhead and profit	Add 15% of Sub total					38.60
vi	Rate Per Unit						553.30
2	**Earthwork in excavation in medium rock using mechanical means**						
i	Labour	Skilled		0.02	1,090.00	21.8	61.3
		Unskilled	Nos.	0.05	790	39.5	
ii	Material						
	1. Diesel		ltr	1.5	101	151.5	151.5
iii	Equipment						
	1. Excavator		hr	0.13	2,100.00	273	273
iv	Sub total						485.8
v	Contractor's overhead and profit	Add 15% of Sub total					72.87
vi	Rate Per Unit						1044.47
3	**Earthwork in excavation in hard rock using mechanical means**						
i	Labour	Skilled		1	1,090.00	1090	8,990.00
		Unskilled	Nos.	10	790	7900	
ii	Material						
	1. Diesel		ltr	72	101	7272	7,272.00
	Equipment						

	1. Excavator		hr	6	2,100.00	12600	12,600.00
iv	Sub total						28,862.00
v	Contractor's overhead and profit	Add 15% of Sub total					4,329.30
vi	Rate per unit						62,053.30
4	**Providing and Placing of RCC M25 machine mixed for foundation (15 m^3)**						
i	Labour						
ii	Labour Materials	Skilled	Nos.	3	1,090.00	3,270.00	26,970.00
		Unskilled	Nos.	30	790	23,700.00	
	1.Cement		MT	6.1	17,000.00	103,700.00	103,700.00
	2.Coarse Sand		m3	6.75	1,870.00	12,622.50	12,622.50
	3.Coarse Aggregate 20 mm down		m3	8.1	1,815.00	14,701.50	14,701.50
	4.Coarse aggregate 10 mm down		m3	5.4	1,650.00	8,910.00	8,910.00
	5. Amixture@ 0.8% of cement		kg	48.8	322.76	15,750.69	15,750.69
	6. Diesel		ltr	6	101	606.00	606.00
iii	Equipment						
	1. Mixture		hr	6	200	1,200.00	1,200.00
	2. Generator		hr	6	150	900.00	900.00
iv	Sub total						185,360.69
v	Contractor's overhead and profit	Add 15% of Sub total					27,804.10
vi	Rate Per Unit						398,525.48
5	**Providing and Placing of RCC M20 machine mixed for foundation (15 m3)**						
i	Labour	Skilled	Nos.	3	1,090.00	3,270.00	26,970.00
		Unskilled	Nos.	30	790	23,700.00	
ii	Material						
	1.Cement		MT	6.05	17,000.00	102,850.00	102,850.00

	2.Coarse Sand		m3	6.75	1,870.00	12,622.50	12,622.50
	3.Coarse Aggregate 20 mm down		m3	8.1	1,815.00	14,701.50	14,701.50
	4.Coarse aggregate 10 mm down		m3	5.4	1,650.00	8,910.00	8,910.00
	5. Admixture@ 0.8% of cement		kg	24.2	322.76	7,810.79	7,810.79
	6. Diesel		ltr	6	101	606.00	606.00
iii	Equipment						
	1. Mixture		hr	6	200	1,200.00	1,200.00
	2. Generator		hr	6	150	900.00	900.00
iv	Sub total						176,570.79
v	Contractor's overhead and profit	Add 15% of Sub total					26,485.62
vi	Rate Per Unit						379,627.20
6	**Providing and Placing of RCC M15 machine mixed for foundation (15 m^3)**						
i	Labour	Skilled	Nos.	3	1,090.00	3,270.00	26,970.00
		Unskilled	Nos.	30	790	23,700.00	
ii	Material						
	1.Cement		MT	5.21	17,000.00	88,570.00	88,570.00
	2.Coarse Sand		m3	6.75	1,870.00	12,622.50	12,622.50
	3.Coarse Aggregate 40 mm down		m3	5.4	1,650.00	8,910.00	8,910.00
	4.Coarse aggregate 20 mm down		m3	5.4	1,815.00	9,801.00	9,801.00
	5. Coarse Aggregate 10 mm down		m3	2.7	1,650.00	4,455.00	4,455.00
	6. Diesel		ltr	6	101	606.00	606.00
iii	Equipment						
	1. Mixture		hr	6	200	1,200.00	1,200.00
	2. Generator		hr	6	150	900.00	900.00
iv	Sub total						154,034.50

v	Contractor's overhead and profit	Add 15% of Sub total					23,105.18
vi	Rate Per Unit						331,174.18
7	**Providing and Laying HYSD Reinforcement steel for superstructures (1 MT)**						
i	Labour	Skilled	Nos.	4	1,090.00	4,360.00	13,840.00
		Unskilled	Nos.	12	790	9,480.00	
ii	Material						
	1.Reinforcement Steel Bar		MT	1.1	84,000.00	92,400.00	92,400.00
	2. Binding wires		kg	8	120	960.00	960.00
iii	Equipment	Add 3%					284.40
iv	Sub total						107,484.40
v	Contractor's overhead and profit	Add 15% of Sub total				16,122.66	16,122.66
vi	Rate Per Unit						231,091.46
8	**Boulder Riprap (up to 1.2m Diameter Boulder) by mechanical means(100m^3)**						
i	Labour	Skilled	Nos.	2	1,090.00	2,180.00	5,340.00
		Unskilled	Nos.	4	790	3,160.00	
iii	Material						
	1. Boulder		m3	-			available on river
	2. Diesel		ltr	90	101	9,090.00	9,090.00
iii	Equipment						
	1. Excavator		hr	6	2,100.00	12,600.00	12,600.00
iv	Sub total						27,030.00
v	Contractor's overhead and profit	Add 15% of Sub total				4,054.50	4,054.50
vi	Rate Per Unit						58,114.50
9	**Stone Pitching (per m^3)**						
i	Labour	Skilled	Nos.	3	1,090.00	3,270.00	4,060.00
		Unskilled	Nos.	1	790	790.00	
ii	Material						
	1. Stone		m3	1.1	1,897.50	2,087.25	2,087.25

	2. Sand		m3	0.71	1,870.00	1,327.70	1,327.70
iii	Equipment	Add 3%					23.7
iv	Sub total						7,498.65
v	Contractor's overhead and profit	Add 15% of Sub total					1,124.80
vi	Rate Per Unit						16,122.10
10	**Earthfill**						
i	Labour	Skilled		0.02	1,090.00	21.8	235.100
		Unskilled	Nos.	0.27	790	213.3	
ii	Equipment	Add 3%					6.399
iii	Sub total						241.499
iv	Contractor's overhead and profit	Add 15% of Sub total					36.225
v	Rate Per Unit						519.223

Rate Analysis Summary

Table 46: Rate analysis Summary

S.N.	Description of work	Rate (Rs.)	Unit
1	Excavation of ordinary rock	553.30	per m^3
2	Excavation of medium rock	1044.47	per m^3
3	Excavation of Hard rock	2074.44	per m^3
4	Concrete M25	26,568.37	per m^3
5	Concrete M20	25,308.48	per m^3
6	Concrete M15	22,078.28	per m^3
7	Reinforcement	231,091.46	per tonne
8	Brickwork	15,000.00	per m^3
9	Boulder Riprap	58,114.50	per m^3
10	Stone pitching	16,122.10	per m^3
11	Earthfill	519.22285	per m^2

Quantity Calculation sheet

Table 47: Quantity Calculation Sheet for different civil works

S.N.	PARTICULAR	NOS	LENGTH (m)	BREADTH (m)	HEIGHT (m)	QUANTITY	UNIT	REMARKS
1	**Headworks**							
1.1	**EW in excavation**							
	Weir Section	1	30.00	13.00	2.50	975.00	m^3	
	Undersluice section	1	10.00	2.00	2.50	50.00	m^3	
	U/S Protection works	1	40.00	70.00	1.00	2800.00	m^3	Breadth approx.
	D/S Protection works	1	30.00	55.00	1.00	1650.00	m^3	Breadth approx.
	Divide wall	1	25.00	1.50	1.50	56.25	m^3	
	Intake and trash rack	1	10.00	2.00	1.00	20.00	m^3	
	Settling basin	1	116.00	10.30	4.80	5735.04	m^3	average height
	PH below turbine RL	1	18.40	10.00	5.00	920.00	m^3	
	PH footing	6	2.00	2.00	5.00	120.00	m^3	
					Total=	12326.29	m^3	
1.2	**Stone Masonry**							
	Weir section	1				65.82	m^3	Volume=area*height(from dwg)
	Divide wall	1	25.00	1.50	5.50	206.25	m^3	
					Total=	272.07	m^3	
1.3	**M15 PCC**	1						average breadth
	U/S Protection works	1	37.00	70.00	0.50	1295.00	m^3	Breadth approx.
	D/S Protection works	1	28.50	55.00	0.50	783.75	m^3	Breadth approx.

					Total=	2078.75	m^3	
1.4	**Stone Pitching**							
	U/S Protection works	1	37.00	70.00	0.50	1295.00	m^3	Breadth approx.
	D/S Protection works	1	28.50	55.00	0.50	783.75	m^3	Breadth approx.
	below Settling basin	1	116.00	8.10	0.45	422.82		average height,breadth
					Total=	2501.57	m^3	
1.5	**M20 PCC in RCC**							
	Weir U/S and D/S	1	155.00	4.50	2.00	1395.00	m^3	Length approx.
	Undersluice							
	Base	1	10.00	2.00	2.00	40.00	m^3	
	Piers	3	1.00	1.00	4.00	12.00	m^3	
	Intake							
	Piers	4	1.00	1.00	6.00	24.00	m^3	
	Below orifice	1	10.00	Area=9.4 m^2		94.00	m^3	Area from dwg
	Settling Basin							
	Wall	1	Area=132.105 m^2		6.00	792.63	m^3	Area from dwg, average height
	Base	1	Area=67.483 m^2	10.41		702.50	m^3	Area from dwg, average width
	Powerhouse							
	Base	1	42	10.00	1.5	630.00	m^3	
	Piers	8	1	1.00	15.0	120.00	m^3	
					Total=	3810.13	m^3	
1.6	**Reinforcement**	1% of PCC				299095.05	kg	density=7850 kg/m^3
1.7	**BW for wall in PH**	1	101	1.00	13.5	1363.50	m^3	

Detailed Estimate:

Table 48: Detailed estimate or Abstract of cost of different civil works

S.N.	PARTICULAR	UNIT	QUANTITY	RATE	AMOUNT	REMARKS
1	**Headworks**					
	EW in excavation					
	Weir Section	m^3	975.000	2074.440	2,022,579.00	
	Undersluice section	m^3	50.00	2074.440	103,722.00	
	U/S Protection works	m^3	2800.00	2074.440	5,808,432.00	
	D/S Protection works	m^3	1650.000	2074.440	3,422,826.00	
	Divide wall	m^3	56.25	2074.440	116,687.25	
	Intake and trash rack	m^3	20.000	2074.440	41,488.80	
	Settling basin	m^3	5735.040	2074.440	11,896,996.38	
	PH below turbine RL	m^3	920.000	2074.440	1,908,484.80	
	PH footing	m^3	120	2074.440	248,932.80	
1.2	**Stone Masonry**					
	Weir section	m^3	65.820	16,122.10	1,061,156.62	
	Divide wall	m^3	206.250	16,122.10	3,325,183.13	
1.3	**M15 PCC**					
	U/S Protection works	m^3	1295.000	22078.279	28,591,370.87	
	D/S Protection works	m^3	783.750	22078.279	17,303,850.91	
1.4	**Stone Pitching**					
	U/S Protection works	m^3	1295	16,122.10		
	D/S Protection works	m^3	783.750	16,122.10		
	below Settling basin	m^3	422.820	16,122.10	6,816,746.32	
1.5	**M20 PCC in RCC**					

	Weir U/S and D/S	m^3	1395	25,308.48	35,305,329.60	
	Undersluice					
	Base	m^3	40.000	25,308.48	1,012,339.20	
	Piers	m^3	12.000	25,308.48	303,701.76	
	Intake					
	Piers	m^3	24.000	25,308.48	607,403.52	
	Below orifice	m^3	94.000	25,308.48	2,378,997.12	
	Settling Basin					
	Wall	m^3	792.630	25,308.48	20,060,260.50	
	Base	m^3	702.498	25,308.48	17,779,157.34	
	Powerhouse					
	Base	m^3	630	25,308.48	15,944,342.40	
	Piers	m^3	120	25,308.48	3,037,017.60	
1.6	**Reinforcement**	Tone	299095.0504	231,091.46	69,118,311.87	**1% of PCC**
1.7	**BW for wall in PH**	m^3	1363.5	15000	20,452.50	
1.8	**Tunnel**				250509473.6	**From Optimization**
1.9	**Penstock**				2828656.741	
	Sub Total				501,573,900.63	
2	**Other Civil Component**		**5% of calculated Civil Cost**		**25,078,695.03**	**approximate**
Total civil cost					**526,652,595.66**	

Table 49: Abstract of cost of all items

S.N.	PARTICULAR	UNIT	QUANTITY	RATE	AMOUNT (in NRS)
1	Total Civil Cost				526,652,595.66
2	Electromechanical Cost	KW	8500	35000.000	297,500,000.00
3	Hydro Mechanical Cost				1,041,513.46
4	Transmission Line (7 % of Civil Works)				36,865,681.70
5	Total Based Cost (TBC)				862,059,790.81
6	Project Development Cost (2% of TBC)				17,241,195.82
7	Land Purchase (2% of TBC)				17,241,195.82
8	Site Office and Infrastructure Development Cost (5% of TBC)				43,102,989.54
9	Office Equipment and Vehicle (2% of TBC)				17,241,195.82
10	Environment Mitigation (0.5% of TBC)				4,310,298.95
11	Project Engineering and Supervision (5% of TBC)				43,102,989.54
12	Subtotal				1,004,299,656.30
13	VAT=13% of total project cost				130,558,955.32
14	Contingencies=5% of total project cost				56,742,930.58
15	Total Amount				1,191,601,542.20
Total Amount in Million NRs.					**1,191.60**

Indrawati III HPP: Power and Energy Calculation

Design Discharge, Q_{40}	15.75	m3/s
Gross head available, Hgr	63.0	m
Type of turbine	Francis	
Assumptions		
Turbine Efficiency, ηt	90%	
Generator Efficiency, ηg	97%	
Transformer Efficiency, ηtr	98%	
Combined efficiency, ηu	85.54%	

Months	No of days	River Flow	Discharge =river flow-10%of minimum flow	Design Discharge	Actual Discharge	Head Loss (full supply), hl, m	Net head, Hn, m	Power generation , Nd, MW	Total Energy, Et,GWh	Net Energy (after 4% outage), En, GWh	Dry Season, E4m, GWh	Wet Season, E8m, GWh
Jan	31	6.815	6.26	15.75	6.26	2.872	60.129	3.138	2.335	2.241	2.241	
Feb	28	5.823	5.27	15.75	5.27	2.872	60.129	2.641	1.774	1.703	1.703	
Mar	31	5.564	5.01	15.75	5.01	2.872	60.129	2.511	1.868	1.793	1.793	
Apr	30	6.211	5.65	15.75	5.65	2.872	60.129	2.835	2.041	1.960	1.960	
May	31	9.489	8.93	15.75	8.93	2.872	60.129	4.479	3.332	3.199	3.199	
Jun	30	25.190	24.63	15.75	15.75	2.872	60.129	7.897	5.686	5.458		5.458
Jul	31	69.446	68.89	15.75	15.75	2.872	60.129	7.897	5.875	5.640		5.640
Aug	31	83.766	83.21	15.75	15.75	2.872	60.129	7.897	5.875	5.640		5.640
Sep	30	58.533	57.98	15.75	15.75	2.872	60.129	7.897	5.686	5.458		5.458
Oct	31	25.967	25.41	15.75	15.75	2.872	60.129	7.897	5.875	5.640		5.640
Nov	30	12.681	12.12	15.75	12.12	2.872	60.129	6.079	4.377	4.202		4.202
Dec	31	8.497	7.94	15.75	7.94	2.872	60.129	3.981	2.962	2.844	2.844	
								Total Energy		45.779	13.740	32.039

Energy proportion		30.013	69.986

Gross Dry Energy	13.740	GWh
Gross Wet Energy	32.039	GWh

After Internal Consumption @0.5% and Transmission Loss @3%

				Rate(Rs/KWh)	Amount
Dry energy		13259124.89	KWh	8.4	111,376,649.10
Wet Energy		30917500.9	KWh	4.8	148,404,004.35
Total		44176626	KWh		259,780,653.44

Financial Analysis

B/C Ratio Calculation

S.N.	Description	Amount in (NRs)	Remarks
1	Total Initial Cost	1,191,601,542.198	5% extra for other component
2	Property Tax	1,191,602	0.1% of initial cost
3	Insurance premium	2,383,203	0.2% of initial cost
4	Spare parts	3,574,805	0.3% of initial cost
5	O&M labour	3,574,805	0.3% of initial cost
6	Total Annual cost	10,724,414	2 TO 5 Costs
7	Annual income of project	259,780,653.44	
8	Net annual income	249,056,239.56	

Discounted Payback Period (PW)

Year	Cash flow (NRs)	Present cash flow (NRs)	Present cash flow (Million NRs)
1	-1,191,601,542	-942,545,303	-942.545303
2	-942,545,303	-693,489,063	-3303.148194
3	-3,303,148,193.55193	-3,054,091,954	-2717.651492
4	-2,717,651,492.42928	-2,468,595,253	-2185.381764
5	-2,185,381,764.13597	-1,936,325,525	-1701.500193
6	-1,701,500,192.96023	-1,452,443,953	-1261.607856
7	-1,261,607,855.52774	-1,012,551,616	-861.7057306
8	-861,705,730.58911	-612,649,491	-498.1583443
9	-498,158,344.28127	-249,102,105	-167.6607204
10	-167,660,720.36505	81,395,519	132.791665
11	132,791,665.01334	381,847,905	405.9301972
12	405,930,197.17550	654,986,437	654.2379537
13	654,237,953.68656	903,294,193	879.9722778
14	879,972,277.78752	1,129,028,517	1085.1853
15	1,085,185,299.69749	1,334,241,539	1271.742592
16	1,271,742,592.34292	1,520,798,832	1441.340131
17	1,441,340,131.11148	1,690,396,371	1595.519712
18	1,595,519,711.81018	1,844,575,951	1735.682967
19	1,735,682,966.99082	1,984,739,207	1863.104108
20	1,863,104,108.06412	2,112,160,348	1978.941509
21	1,978,941,509.03985	2,227,997,749	2084.248237
22	2,084,248,237.19961	2,333,304,477	2179.981626
23	2,179,981,626.43575	2,429,037,866	2267.01198
24	2,267,011,980.28679	2,516,068,220	2346.130484
25	2,346,130,483.78773	2,595,186,723	2418.056396
26	2,418,056,396.06131	2,667,112,636	2483.443589
27	2,483,443,589.03730	2,732,499,829	2542.886492
28	2,542,886,491.74274	2,791,942,731	2596.925494
29	2,596,925,494.20223	2,845,981,734	2646.05186
30	2,646,051,860.07450	2,895,108,100	2690.712193
31	2,690,712,192.68565	2,939,768,432	2731.312495

32	2,731,312,495.05942	2,980,368,735	2768.221861
33	2,768,221,860.85376	3,017,278,100	2801.77583
34	2,801,775,829.75771	3,050,832,069	2832.279438
35	2,832,279,437.85220	3,081,335,677	2860.009991
36	2,860,009,990.66538	3,109,066,230	2885.219584
37	2,885,219,584.13191	3,134,275,824	2908.137396
38	2,908,137,396.37420	3,157,193,636	2928.971771
39	2,928,971,771.13993	3,178,028,011	2947.912112
40	2,947,912,111.83604	3,196,968,351	2965.130603
	Net Present Worth	**3,196,968,351**	**3196968.35140**

Hence, Discounted Payback Period (PBP) = 10 years

B/C Ratio

Rate of interest	0.1	
Project life	40 years	
Description	**Amount (NRs)**	**Annual worth (NRs)**
Initial cost	1,191,601,542	121,852,476
Net annual income	249,056,239.56	249,056,239.56
Present (O and M)	3,574,805	3,574,805
Salvage value	119,160,154	269,232
Modified B/C ratio		**2.02**
Salvage value is taken as 10%		

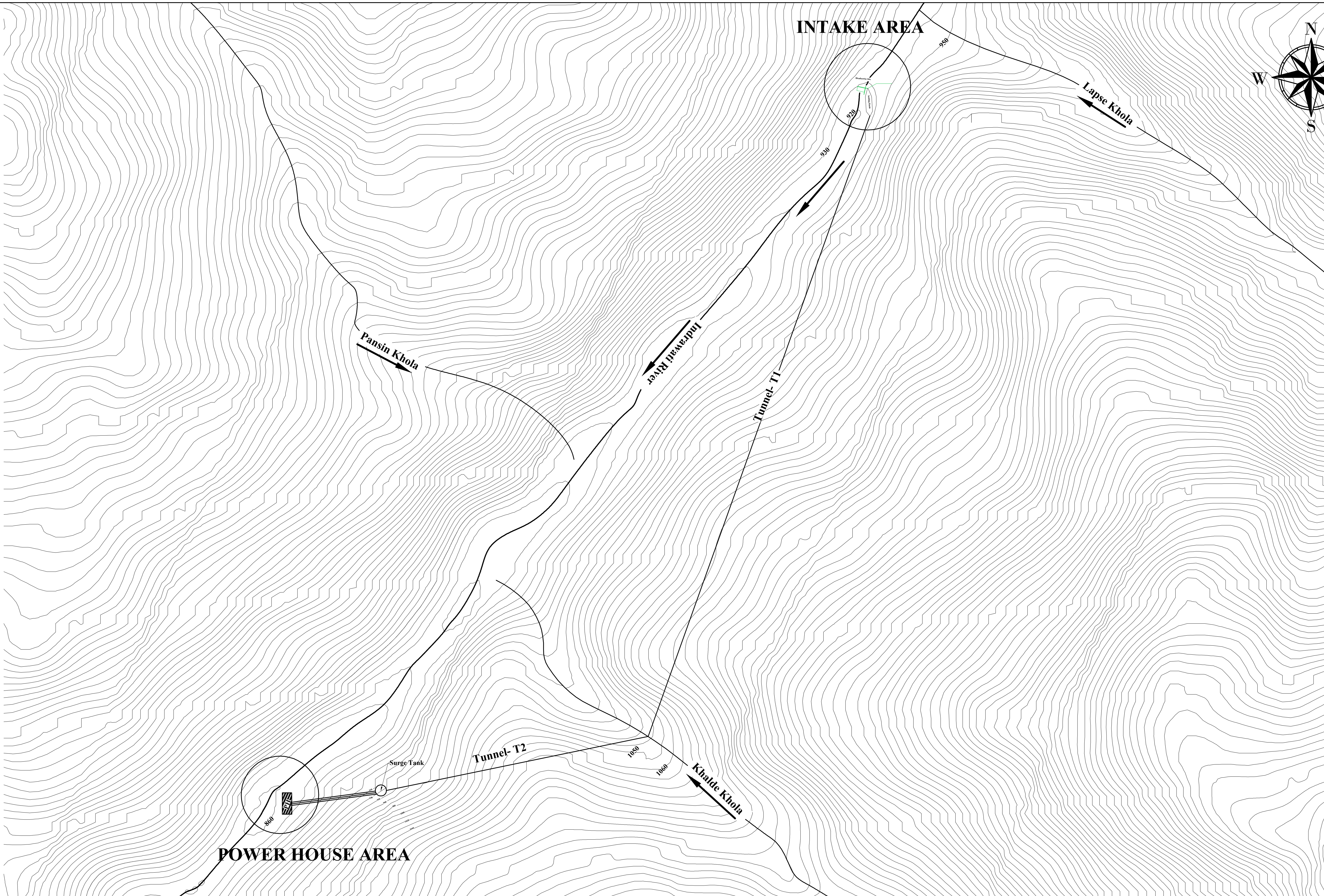
IRR Calculation

Initial cost (Million NRs)	I	1,191,601,542
Net annual income (Million NRs)	A	249,056,239.56
$I = A\times((1+x)^{40}-1)/(x\times(1+x)^{40}))$		
On Solving this, we get		
IRR (x) =	**20.59%**	

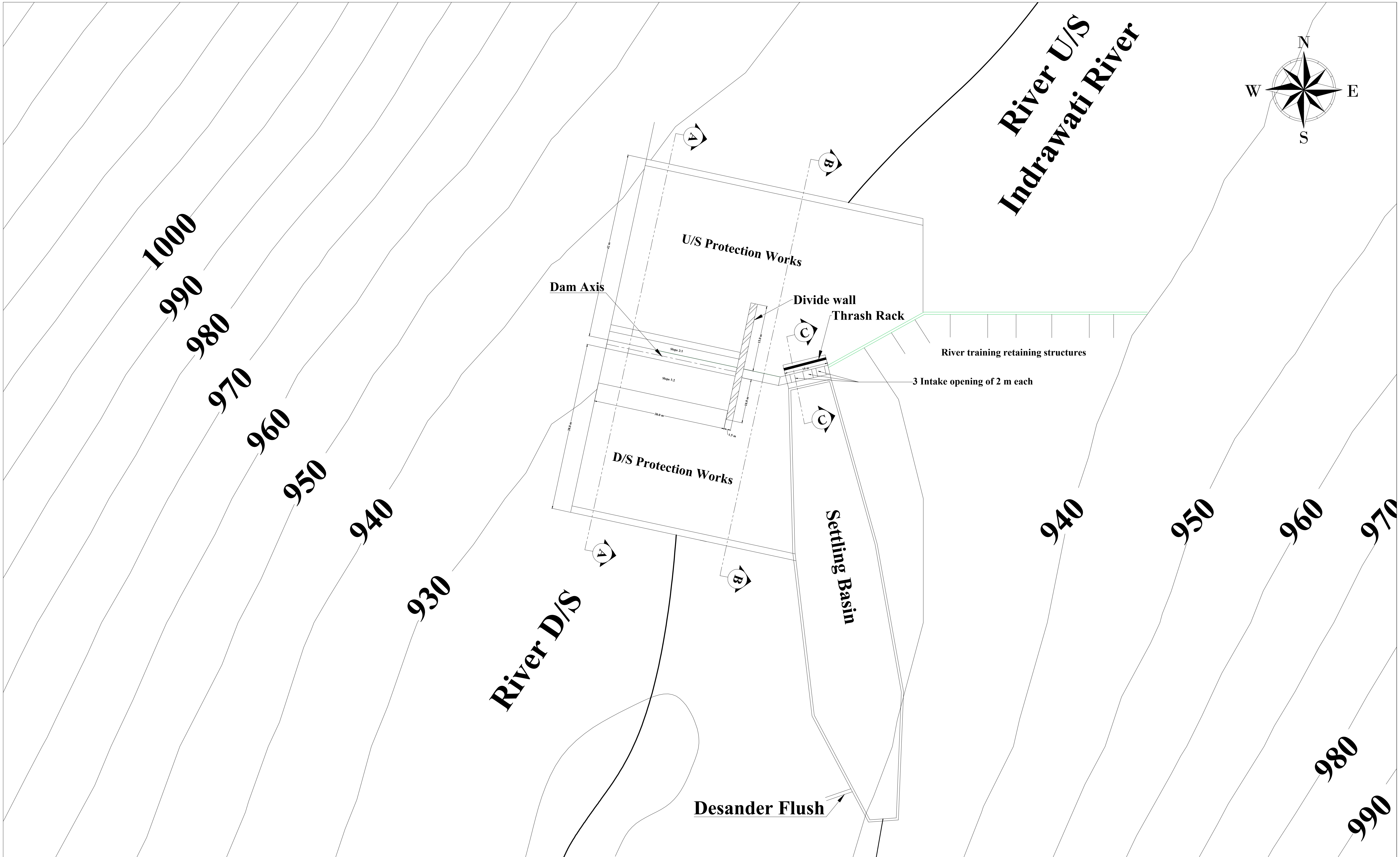
Summary of Financial Analysis

Discounted Payback Period	10 years
B/C ratio	2.02
IRR	20.59%

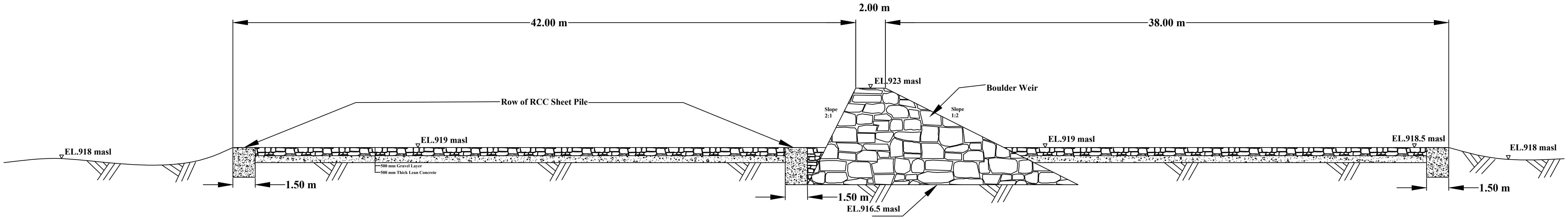
Appendix-D (Drawings)



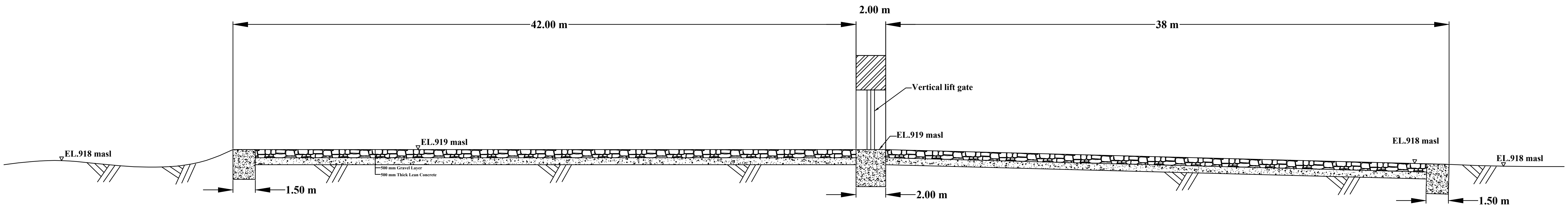
Tribhuvan University Institute of Engineering Department of Civil Engineering Pulchowk Campus	PRE-FEASIBILITY STUDY Indrawati-III Hydro Electric Project Sindhupalchowk, Nepal	Prepared by: Daman Raj Neupane (075BCE001) Anmol Yadav (075BCE021) Ashutosh Lamsal (075BCE029) Ashutosh Yadav(075BCE030) Basudev Chaudhary(075BCE032) Buddha Gubhaju(075BCE047)	Drawing No: 01	Date: 2079-11-02
	Title: Layout of Indrawati-III Hydropower Project		Checked by: Dr. Bhola Nath Sharma Ghimire Department of Civil Engineering Pulchowk Campus	



Tribhuvan University Institute of Engineering Department of Civil Engineering Pulchowk Campus	PRE-FEASIBILITY STUDY Indrawati-III Hydro Electric Project Sindhupalchowk, Nepal Title: Headworks Plan of Indrawati HEP	Prepared by: Daman Raj Neupane (075BCE001) Anmol Yadav (075BCE021) Ashutosh Lamsal (075BCE029) Ashutosh Yadav(075BCE030) Basudev Chaudhary(075BCE032) Buddha Gubhaju(075BCE047)	Drawing No: 02 Date: 2079-11-02 Checked by: Dr. Bhola Nath Sharma Ghimire Department of Civil Engineering Pulchowk Campus

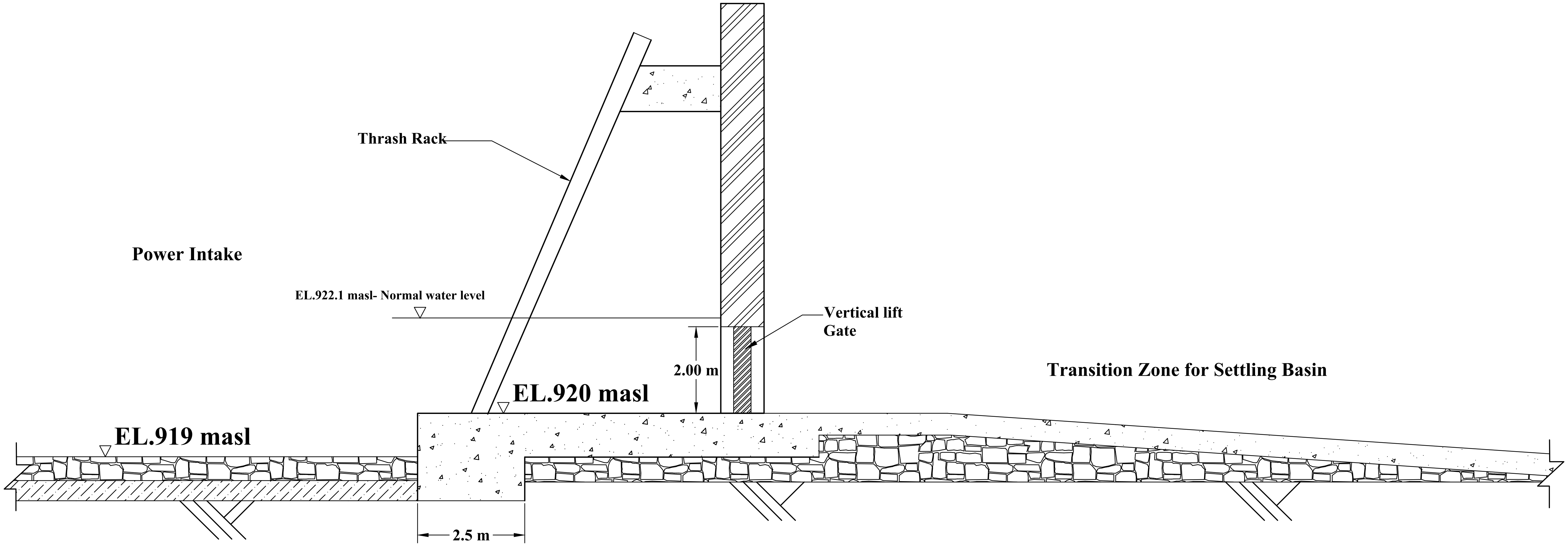


Section at A-A or Weir Section



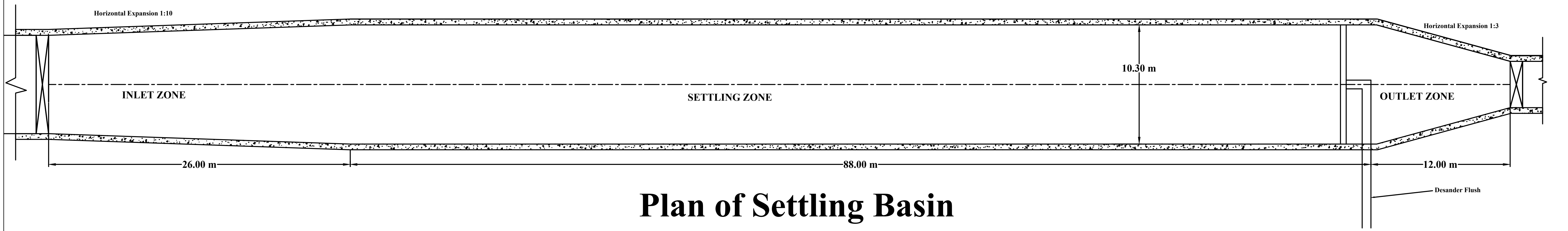
Section at B-B or Undersluice Section

Tribhuvan University Institute of Engineering Department of Civil Engineering Pulchowk Campus	PRE-FEASIBILITY STUDY Indrawati-III Hydro Electric Project Sindhupalchowk, Nepal Title: Section of Weir and Undersluice	Prepared by: Daman Raj Neupane (075BCE001) Anmol Yadav (075BCE021) Ashutosh Lamsal (075BCE029) Ashutosh Yadav(075BCE030) Basudev Chaudhary(075BCE032) Buddha Gubhaju(075BCE047)	Drawing No: 03 Date: 2079-11-02 Checked by: Dr. Bhola Nath Sharma Ghimire Department of Civil Engineering Pulchowk Campus

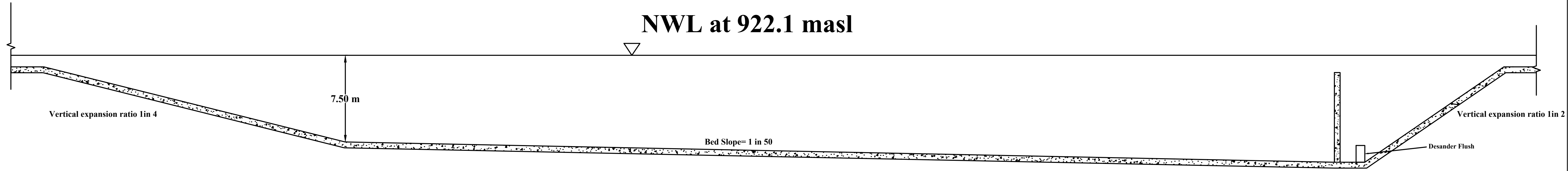


Section at C-C or Intake Section

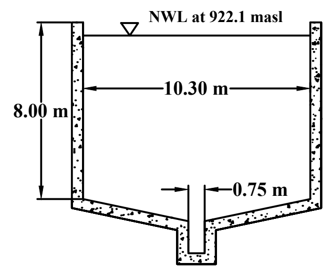
Tribhuvan University Institute of Engineering Department of Civil Engineering Pulchowk Campus	PRE-FEASIBILITY STUDY Indrawati-III Hydro Electric Project Sindhupalchowk, Nepal Title: Intake Section	Prepared by: Daman Raj Neupane (075BCE001) Anmol Yadav (075BCE021) Ashutosh Lamsal (075BCE029) Ashutosh Yadav(075BCE030) Basudev Chaudhary(075BCE032) Buddha Gubhaju(075BCE047)	Drawing No: 04 Checked by: Dr. Bhola Nath Sharma Ghimire Department of Civil Engineering Pulchowk Campus	Date: 2079-11-02



Plan of Settling Basin



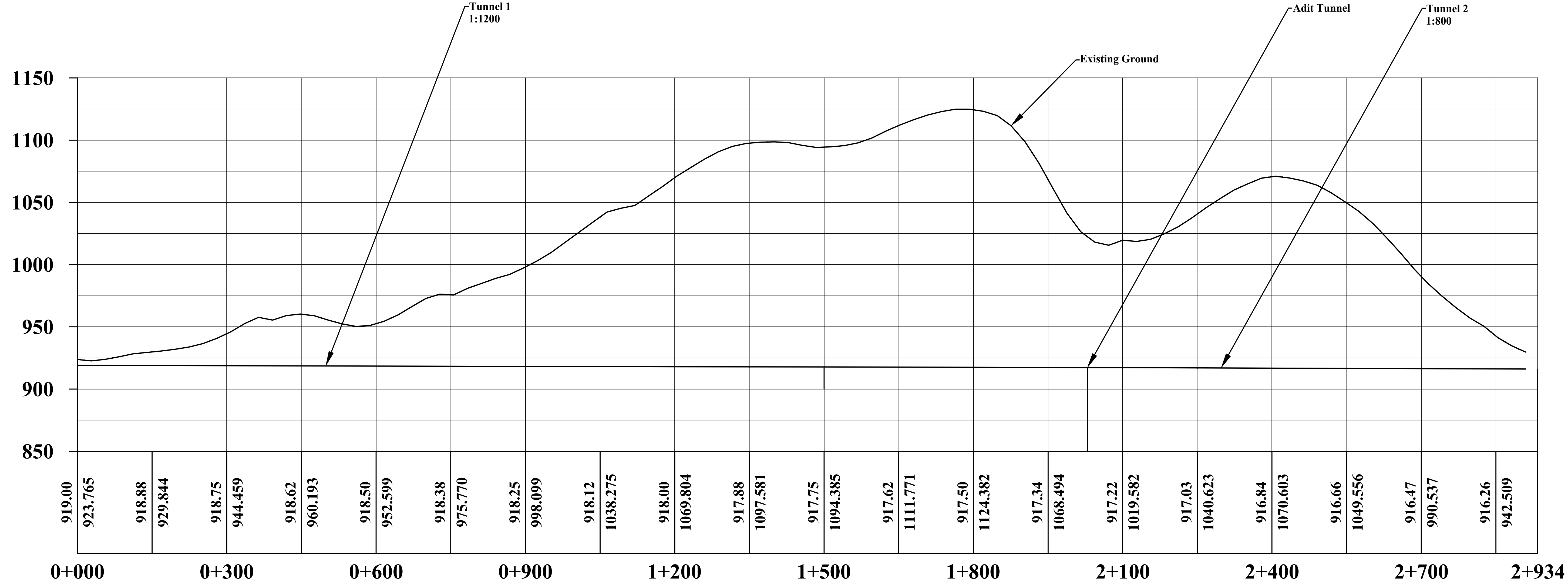
Longitudinal Section of Settling Basin



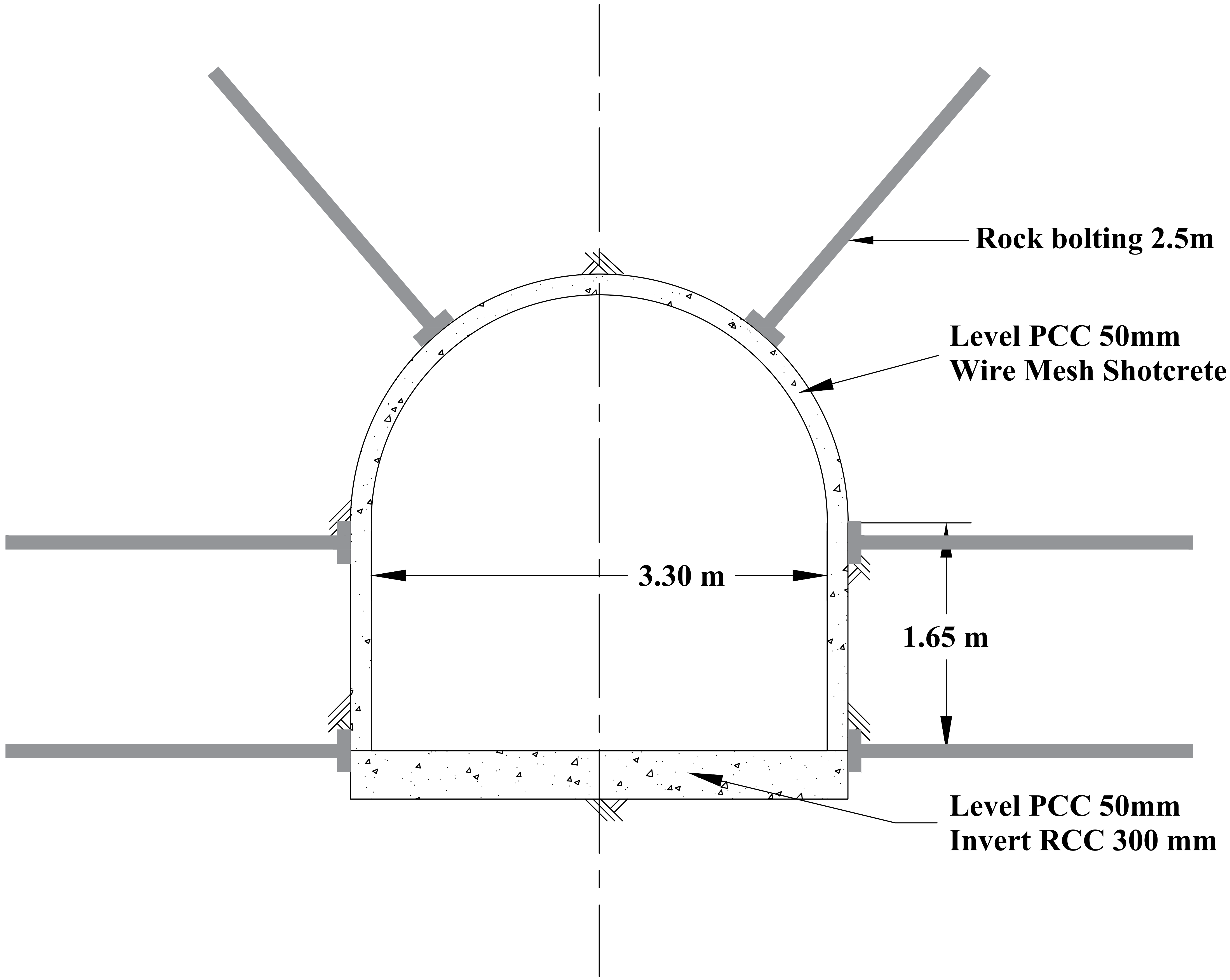
Cross Section of Settling Basin

Tribhuvan University **Institute of Engineering** **Department of Civil Engineering** **Pulchowk Campus**	**PRE-FEASIBILITY STUDY** **Indrawati-III Hydro Electric Project** **Sindhupalchowk, Nepal** **Title: Drawings of Settling Basin**	**Prepared by:** **Daman Raj Neupane (075BCE001)** **Anmol Yadav (075BCE021)** **Ashutosh Lamsal (075BCE029)** **Ashutosh Yadav(075BCE030)** **Basudev Chaudhary(075BCE032)** **Buddha Gubhaju(075BCE047)**	**Drawing No: 05** **Date: 2079-11-02** **Checked by: Dr. Bhola Nath Sharma Ghimire** **Department of Civil Engineering** **Pulchowk Campus**

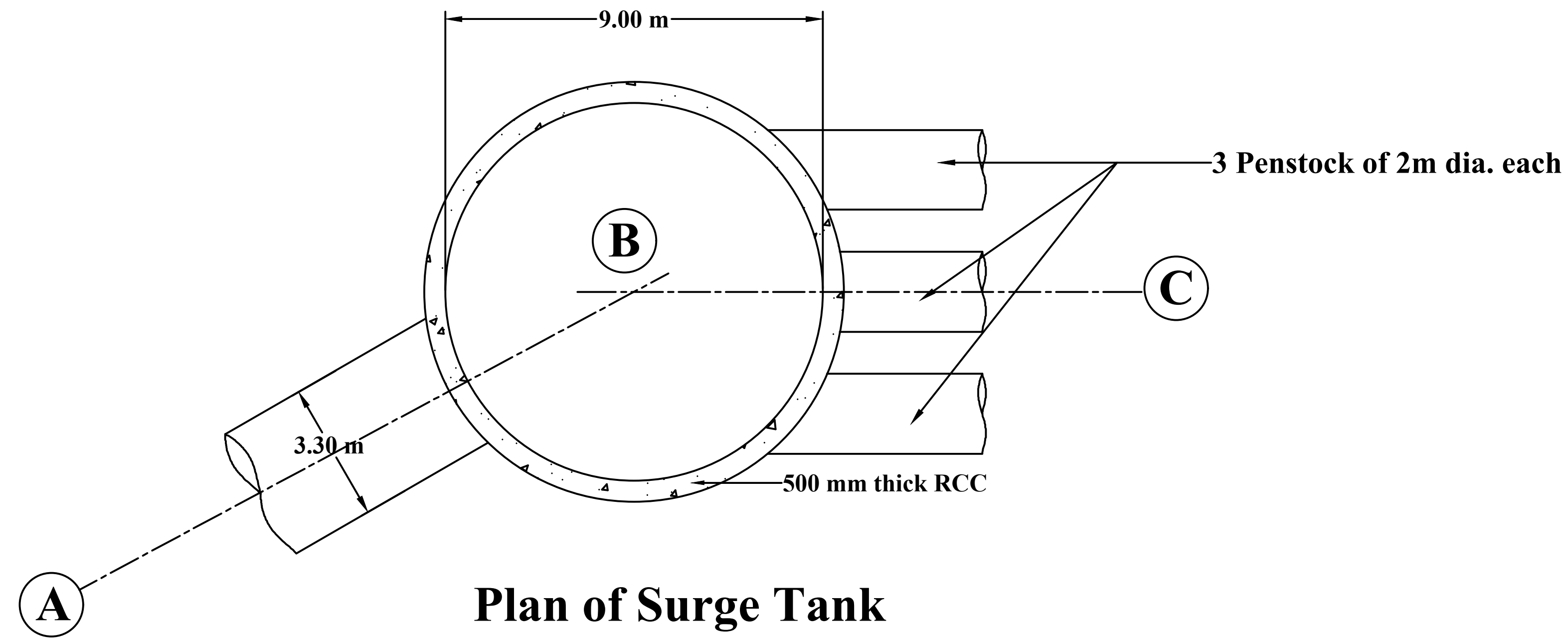
Longitudinal Profile of Head Race Tunnel



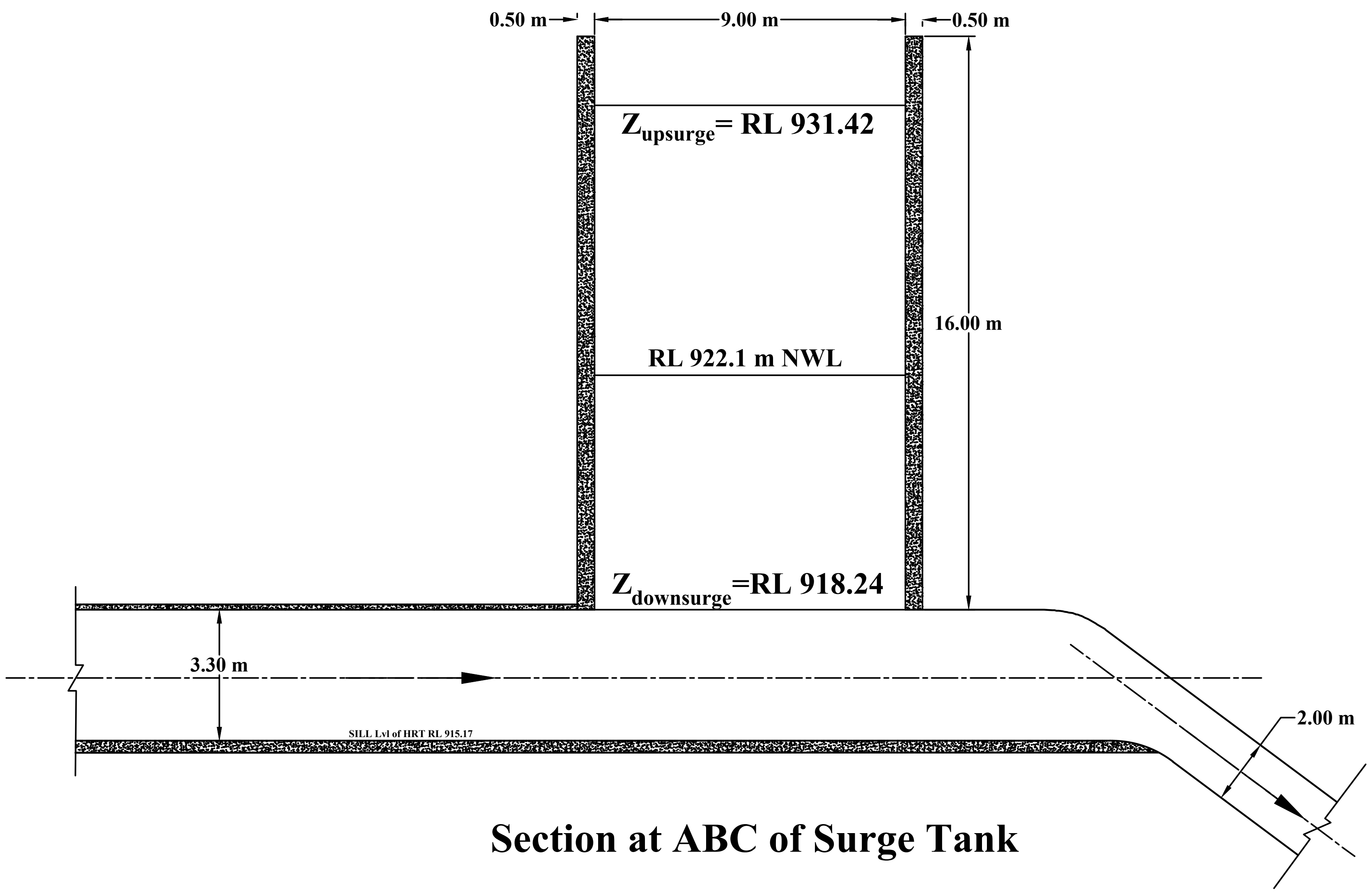
Tribhuvan University Institute of Engineering Department of Civil Engineering Pulchowk Campus	PRE-FEASIBILITY STUDY Indrawati-III Hydro Electric Project Sindhupalchowk, Nepal Title: Longitudinal Profile of Tunnel	Prepared by: Daman Raj Neupane (075BCE001) Anmol Yadav (075BCE021) Ashutosh Lamsal (075BCE029) Ashutosh Yadav(075BCE030) Basudev Chaudhary(075BCE032) Buddha Gubhaju(075BCE047)	Drawing No: 07 Checked by: Dr. Bhola Nath Sharma Ghimire Department of Civil Engineering Pulchowk Campus	Date: 2079-11-02



Tribhuvan University Institute of Engineering Department of Civil Engineering Pulchowk Campus	PRE-FEASIBILITY STUDY Indrawati-III Hydro Electric Project Sindhupalchowk, Nepal Title: Tunnel Cross Section	Prepared by: Daman Raj Neupane (075BCE001) Anmol Yadav (075BCE021) Ashutosh Lamsal (075BCE029) Ashutosh Yadav(075BCE030) Basudev Chaudhary(075BCE032) Buddha Gubhaju(075BCE047)	Drawing No: 06 / Date: 2079-11-02 Checked by: Dr. Bhola Nath Sharma Ghimire Department of Civil Engineering Pulchowk Campus

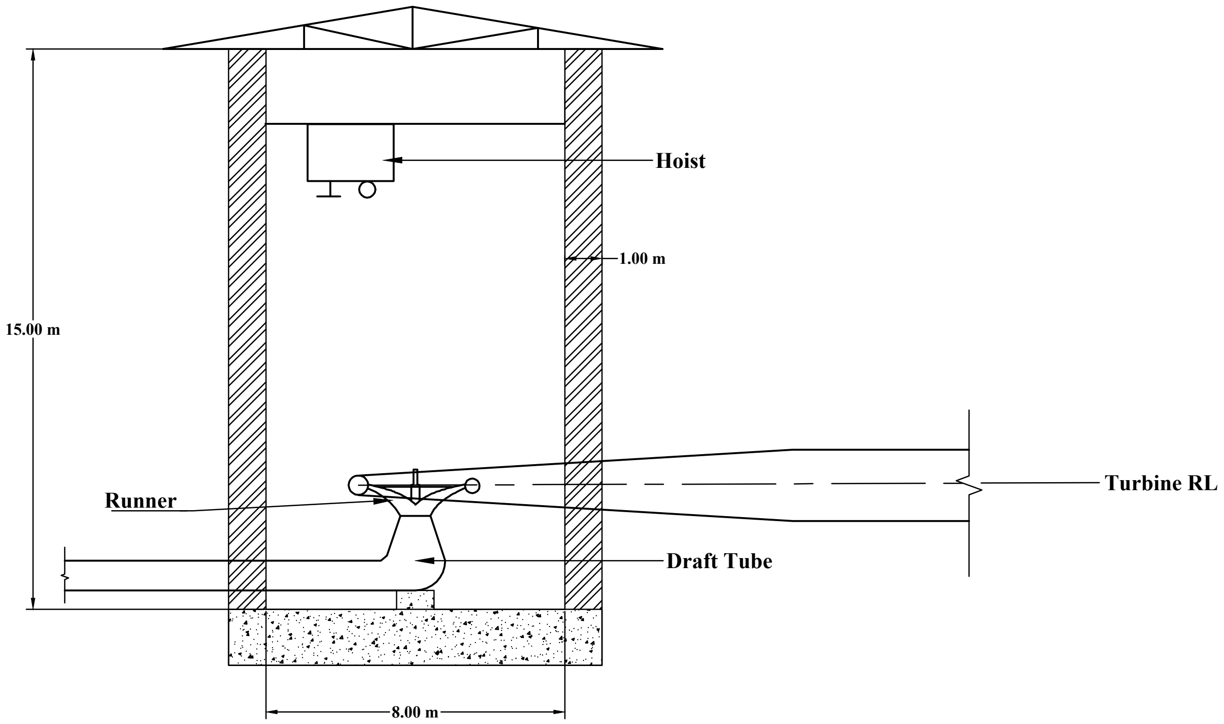


Plan of Surge Tank

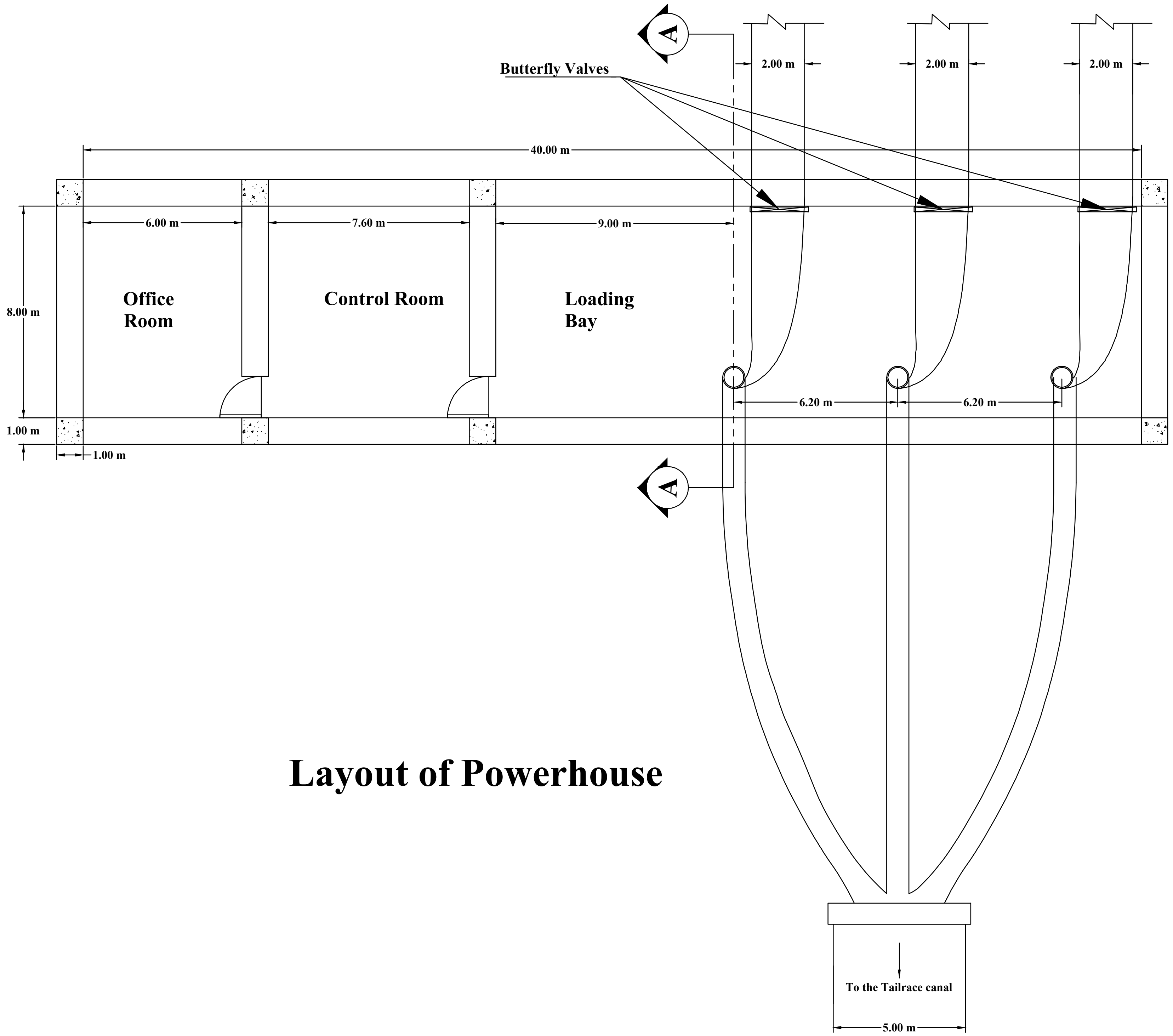


Section at ABC of Surge Tank

Tribhuvan University **Institute of Engineering** **Department of Civil Engineering** **Pulchowk Campus**	**PRE-FEASIBILITY STUDY** **Indrawati-III Hydro Electric Project** **Sindhupalchowk, Nepal** **Title: Drawing of Surge Tank**	**Prepared by:** **Daman Raj Neupane (075BCE001)** **Anmol Yadav (075BCE021)** **Ashutosh Lamsal (075BCE029)** **Ashutosh Yadav(075BCE030)** **Basudev Chaudhary(075BCE032)** **Buddha Gubhaju(075BCE047)**	**Drawing No: 08** **Date: 2079-11-02** **Checked by: Dr. Bhola Nath Sharma Ghimire** **Department of Civil Engineering** **Pulchowk Campus**



Section at A-A of Powerhouse



Layout of Powerhouse

Tribhuvan University Institute of Engineering Department of Civil Engineering Pulchowk Campus	PRE-FEASIBILITY STUDY Indrawati-III Hydro Electric Project Sindhupalchowk, Nepal Title: Layout and section of Powerhouse	Prepared by: Daman Raj Neupane (075BCE001) Anmol Yadav (075BCE021) Ashutosh Lamsal (075BCE029) Ashutosh Yadav(075BCE030) Basudev Chaudhary(075BCE032) Buddha Gubhaju(075BCE047)	Drawing No: 09 Date: 2079-11-02 Checked by: Dr. Bhola Nath Sharma Ghimire Department of Civil Engineering Pulchowk Campus