KATHMANDU UNIVERSITY

SCHOOL OF ENGINEERING

DEPARTMENT OF CIVIL ENGINEERING

FINAL YEAR REPORT ON

"POTENTIAL STUDY OF SETI KHOLA HYDROPOWER PROJECT"

A Final Year Project Report submitted in partial fulfilment of the requirements for the degree of Bachelor of Engineering in Civil Engineering (Specialization in Hydropower)

SUBMITTED BY:

SULAV ADHIKARI (028042-20)

AYUSH MALLA (028063-20)

ANUPA MANDAL (028064-20)

SITAL NEUPANE (028068-20)

DHARAMBIR YADAV (028097-20)

PROJECT SUPERVISOR:

DR. UMESH JUNG THAPA

SUBMITTED TO:

DEPARTMENT OF CIVIL ENGINEERING

KATHMANDU UNIVERSITY

JUNE, 2025

DEDICATED TO OUR PARENTS AND TEACHERS

DECLARATION

We affirm that the project titled "**POTENTIAL STUDY OF SETI KHOLA HYDROPOWER PROJECT**" is a genuine record of our own work completed as a requirement for the Bachelor of Engineering in Civil Engineering (Specialization in Hydropower). This project has not been previously published or submitted to fulfil any other bachelor's degree requirements. Any use of data, literature, or work from other sources has been properly acknowledged and cited in the references. This report is produced solely for academic purposes to satisfy the requirements for our bachelor's degree and does not guarantee the implementation of the findings and results in actual construction projects.

Group 9 Members	**Signature**
Sulav Adhikari	______________________
Ayush Malla	______________________
Anupa Mandal	______________________
Sital Neupane	______________________
Dharambir Yadav	______________________

Date: …………………………….

CERTIFICATION

FINAL YEAR PROJECT REPORT

ON

POTENTIAL STUDY OF SETI KHOLA HYDROPOWER PROJECT

Submitted By:

Sulav Adhikari

Ayush Malla

Anupa Mandal

Sital Neupane

Dharambir Yadav

Approved By:

1. **Project Supervisor**

______________________	______________________	______________________
(Name)	(Signature)	(Date)

2. **External Supervisor**

______________________	______________________	______________________
(Name)	(Signature)	(Date)

3. **Head of the Department**

______________________	______________________	______________________
(Name)	(Signature)	(Date)

ACKNOWLEDGEMENT

First and foremost, we extend our sincere appreciation to Kathmandu University School of Engineering and the Department of Civil Engineering for providing us with the academic platform, resources, and guidance necessary to undertake this research. Special thanks to **Dr. Shyam Sundar Khadka**, Associate Dean of KUSOE and **Asst. Prof. Manish Prakash**, Department In-charge, for their encouragement and support in shaping this project.

Our sincere thanks also go to **Asst. Prof. Dr. Suman Shrestha**, our project coordinator, for his invaluable advice, structured supervision, and unwavering support throughout the project duration. We are profoundly grateful to our project supervisor, **Dr. Umesh Jung Thapa**, whose expert guidance, insightful feedback, and continuous motivation were instrumental in refining our research. His dedication and patience helped us navigate technical challenges and strengthen our analytical approach.We are equally indebted to all the faculty members of the Department of Civil Engineering for their constructive suggestions and encouragement.

We acknowledge the generous cooperation of **Seti Khola Hydropower Pvt. Ltd.** for providing essential project data, technical reports, and field insights, which significantly enriched our study. Their willingness to share expertise was crucial in ensuring the practical relevance of our findings. We also extend our heartfelt thanks to our seniors, colleagues, and friends for their moral support, brainstorming sessions, and constructive critiques that helped refine our work. Their camaraderie made this academic journey both productive and enjoyable.

Thank You,

Group 9

CIEG (IV/II)

ABSTRACT

This project, titled "Potential Study of Seti Khola Hydropower Project," conducted as part of a Bachelor’s degree in Civil Engineering with a specialization in Hydropower Engineering at Kathmandu University, explores the viability of the Seti Khola Hydropower Project, a Run-of- River (RoR) hydropower scheme located in Pokhara Metropolitan City of the Kaski district of Nepal, aiming to harness the potential of the Seti River. The primary objectives include hydrological analysis, hydraulic component design, stability analysis, structural assessment of the powerhouse, and economic feasibility evaluation.

The project methodology integrates theoretical knowledge with practical techniques. Desk study outcomes guided alternative analysis to explore project configurations, hydrological analysis to assess design parameters and flood risks utilizing computational tools, hydraulic design and stability analysis to design and ensure the project integrity, powerhouse structural analysis to ensure powerhouse integrity under diverse conditions and economic analysis involved cost estimation, revenue projections, and financial indicator assessment.

The project revealed significant findings regarding the Seti Khola Hydropower Project's potential. With a gross head of 68 meters and an installed capacity of 23.5 MW, the project demonstrates substantial power generation potential. Hydrological assessments informed hydraulic design for critical structures such as the weir, intake, desander, headrace tunnel, surge tank, penstock, and powerhouse. Stability analysis of headworks, headrace tunnel, and powerhouse structure ensured component integrity. Economic evaluations yielded promising financial indicators, including an Internal Rate of Return (IRR) of 13% and a Payback Period of 8 years.

EXECUTIVE SUMMARY

Seti Khola Hydropower Project is a type of RoR scheme located mostly in Lamgadi Area, Pokhara Metropolitan City, Kaski District of Nepal. The project area is located between latitude 28° 05' 00" N to 28° 08' 05" N and longitude 84° 04' 15" E to 84° 05' 28" E. The project site is accessible via Prithvi Highway and already has a completed access road up to the H/W site of the project. The project Survey licenses were received from Department of Electricity Development, Ministry of Energy, Government of Nepal with an initial installed capacity of 22MW. After the study, the potential capacity of the project was found to be optimum at 23.5MW corresponding to the design discharge of 43.136 m^3/s designed at Q_{40}% and gross head of 68m. Seti Khola being a gauged river, the mean monthly flow of the Seti Khola is computed using Catchment Area Ratio (CAR) method. The ratios of catchment area at Intake Site 866.12 km^2 and catchment area at Gandaki gauging station 1350 km^2 is used to determine the discharge available at Seti Khola for 40% of time, which was found to be a discharge equal to 43.136 m^3/s and has been used as design discharge for the project. Gumbel's method, Log Pearson III and Log Normal methods were used for the flood analysis among which the Gumbel's Method was selected. The flood discharge having 100 years return period obtained from the Gumbel's Method, found to be 1882 m^3/s was adopted as the upper limit for the design of the headwork components. From the analysis of the project boundary area, the project has all of its components located in the right bank of the Seti Khola. The major components of the project include weir structure, Desander, Headrace Tunnel, Surge tank, Penstock, Powerhouse and Tailrace. The weir of 55 m width and a length of 134 m along with the undersluice with a bay of 9 m width and length of 154 m is placed across the Seti Khola, making the structure responsible for diverting the flow towards the Intake. The diverted flow in discharged through the Intake orifices, which are four in number, having width 8 m and height of 2.0m. The intake is drained by a catchment area of 866.12 km^2 and directs the flow of 51.76 m^3/s towards the Desander. The project has a provision of desander with flushing canal, where particles of size up to 0.15 mm are removed from the flow through settling and flushing. The desander consists of 2 basins of dimensions 144m having transition length of 37m. Flow from the Desander is stored at the head pond to settle any remaining sediments within the flow before it is conveyed to the headrace tunnel. The headrace tunnel is of inverted D Shape and has a length of 3050 m length with 4.8m optimized diameter. At the end of the tunnel is one surge tank, the type of surge tank is restricted orifice cylindrical of diameter 18m and height of 24.8m and for optimization criteria of surge shaft, minimum submergence head available is more than minimum submergence head required. Flow then enters a penstock with diameter of 4.4m before bifurcation. At last, the flow enters the powerhouse of size 45.50m*17.25m*26.71m where two units of vertical axis Francis turbines are proposed for energy generation. The generated power is planned to be evacuated via Damauli- Lekhnath Transmission Line. The total estimated cost is approximately NRs. 5,280,155,536, out of which the cost of civil works is NRs. 2,047,480,422. According to the financial analysis, the benefit-cost ratio (BCR) is 1.96 and the internal rate of return (IRR) is 13%. The project's payback period is estimated to be 8 years

कार्यकारी सारांश

सेती खोला जलविद्युत् आयोजना एक प्रवाह अनुसारको (RoR) प्रकारको आयोजना हो, जुन नेपालको पोखरा महानगरपालिका अन्तर्गत लामगादी क्षेत्र, कास्की जिल्लामा अवस्थित छ। आयोजना क्षेत्र अक्षांश २८° ०५' ००" देखि २८° ०८' ०५" उत्तर र देशान्तर ८४° ०४' १५" देखि ८४° ०५' २८" पूर्व को बीचमा पर्दछ। परियोजनास्थलमा पृथ्वी राजमार्गमार्फत् सजिलै पुग्न सकिन्छ र हेडवर्क्स साइटसम्मको पहुँच मार्ग पहिल्यै निर्माण भइसकेको छ।

नेपाल सरकारको ऊर्जा मन्त्रालय अन्तर्गत विद्युत् विकास विभागबाट प्रारम्भिक रूपमा २२ मेगावाट क्षमताको लागि सर्वेक्षण इजाजतपत्र प्राप्त गरिएको थियो। विस्तृत अध्ययनपश्चात्, आयोजनाको उपयुक्त क्षमता २३.५ मेगावाट पाइएको छ, जुन ४३.१३६ घन मीटर प्रति सेकेण्ड को डिजाइन प्रवाह (Q40%) र ६८ मिटर को कुल हेडमा आधारित छ।

सेती खोला एक गेज गरिएको नदी भएकाले यसको औसत मासिक प्रवाह Catchment Area Ratio (CAR) विधिबाट गणना गरिएको हो। Intake साइटको पानी संकलन क्षेत्रफल ८६६.१२ वर्ग किमी र गण्डकी गेजिङ स्टेशनको १३५० वर्ग किमी को अनुपात अनुसार डिजाइन प्रवाह ४३.१३६ घन मीटर प्रति सेकेण्ड निर्धारण गरिएको हो।

बाढी विश्लेषणको लागि गम्बेल विधि, Log Pearson III, र Log Normal विधिहरूको प्रयोग गरियो, जसमा गम्बेल विधिलाई उपयुक्त मानिएको हो। सो विधिबाट १०० वर्षको पुनरावृत्ति अवधिका लागि प्राप्त बाढी प्रवाह १८८२ घन मीटर प्रति सेकेण्ड रहेको र यही प्रवाहलाई हेडवर्क्स डिजाइनको अधिकतम सीमा मानिएको छ।

परियोजनाका सबै संरचनाहरू सेती खोलाको दाहिने किनारमा अवस्थित छन्। आयोजना अन्तर्गत प्रमुख संरचनाहरूमा Weir, Intake, Desander, Headrace Tunnel, Surge Tank, Penstock, Powerhouse, र Tailrace पर्दछन्।

- Weir को चौडाइ ५५ मिटर र लम्बाइ १३४ मिटर छ भने undersluice को लम्बाइ १५४ मिटर र बे चौडाइ ९ मिटर छ। यसले प्रवाह Intake तर्फ मोड्ने काम गर्दछ।

- Intake मा ८ मिटर चौडाइ र २ मिटर उचाइ भएका ४ वटा ओरिफिस छन्, जसबाट ५१.७६ घन मीटर प्रति

सेकेण्ड पानी Desander तर्फ पठाइन्छ।

- Desander मा दुईवटा १४४ मिटर लम्बाइ भएका basin छन्, जसले ०.१५ मि.मि. सम्मका कणहरू हटाउँछ।
- त्यसपछि प्रवाह Headrace Tunnel हुँदै जान्छ, जुन Inverted D Shape मा ३०५० मिटर लम्बाइ र ४.८ मिटर व्यास रहेको छ।
- टनेलको अन्त्यमा Restricted orifice cylindrical surge tank रहेको छ जसको व्यास १८ मिटर र उचाइ २४.८ मिटर छ।
- प्रवाह त्यसपछि ४.४ मिटर व्यासको Penstock हुँदै दुई भागमा विभाजित भई Powerhouse प्रवेश गर्छ।

Powerhouse को आकार ४५.५० मि × १७.२५ मि × २६.७१ मि रहेको छ र त्यहाँ २ वटा Vertical Axis Francis Turbine Units प्रस्ताव गरिएको छ। उत्पादित विद्युत् दमौली-लेखनाथ प्रसारण लाइनमार्फत राष्ट्रिय ग्रिडमा पठाउने योजना छ।

परियोजनाको अनुमानित कुल लागत रु. ५,२८,०,१५५,५३६ रहेको छ, जसमा नागरिक संरचना (Civil Works) को लागत रु. २,०४,७,४८०,४२२ छ। वित्तीय विश्लेषण अनुसार, लाभ-लागत अनुपात (BCR) १.९६ र आन्तरिक प्रतिफल दर (IRR) १३% रहेको छ। पूँजी पुनः प्राप्ति अवधि (Payback Period) अनुमानित ८ वर्ष छ।

SYMBOLS AND ABBREVIATIONS

- **Amsl** : Above Mean Sea Level
- **B/C** : Benefit-Cost Ratio
- **CAR** : Catchment Area Ratio
- **CBS** : Central Bureau of Statistics
- **Cm** : Centimeter
- **cm^2** : Square Centimeter
- **Cumecs** : Cubic Meter per Second
- **D/S** : Downstream
- **DEM** : Digital Elevation Model
- **DHM** : Department of Hydrology and Meteorology
- **DOED** : Department of Electricity Development
- **ETABS** : Extended Three-dimensional Analysis of Building Systems
- **FDC** : Flow Duration Curve
- **GIS** : Geographic Information System
- **GoN** : Government of Nepal
- **GWh** : Gigawatts Hour
- **HEC-RAS** : Hydrologic Engineering Center – River Analysis System
- **HTA** : Hydraulic Transient Analysis
- **HVAC** : Heating, Ventilation, and Air Conditioning
- **INPS** : Integrated Nepal Power System
- **IRR** : Internal Rate of Return
- **IS** : Indian Standard
- **kW** : Kilowatt
- **MW**: Megawatt
- **NATM** : New Austrian Tunneling Method

- **PROR** : Peaking Run-of-River
- **RCC** : Reinforced Cement Concrete
- **ROR** : Run-of-River
- **TBM** : Tunnel Boring Machine

SALIENT FEATURES

S.N	Parameters	Details
1	**General**	
1.1	Name of the project	Seti Khola Hydropower Project
1.2	Name of the River	Seti River
1.3	Type of Scheme	RoR Scheme
1.4	Project Location	Pokhara Metropolitan City
1.5	Project Boundary:	
	Latitude	28^{o} 05' 00" N to 28^{o} 08' 05" N
	Longitude	84^{o} 04' 15" E to 84^{o} 05' 28" E
1.6	Gross Head	68 m
1.7	Net Head	63.9 m
2	**Hydrology**	
2.1	Catchment area at Intake Site	866.12 km^2
2.2	Catchment area at Gandaki gauging station	1350 km^2
2.3	Catchment area at Phoolbari gauging station	582 km^2
2.4	Design Discharge Q40	43.136m^3/s
2.5	Flood Discharge for Headwork's design (100 years)	1882 m^3/s
2.6	River Bed Level at Intake	576.08 m
3	**Structures**	
3A	**Weir and Undersluice**	
3A.1	Lowest Riverbed Level	576.08
3A.2	Weir Crest Level	579.082
3A.3	Weir Length	134 m
3A.4	Weir Width	55 m
3A.5	Under sluice Length	155 m
3A.6	Under sluice Openings:	
	No. of Opening	2 nos
3B	**Intake**	
3B.1	Intake Type	Side Intake
3B.2	Invert Level of Intake	577.08
3B.3	Intake Openings:	
	Number of Opening	4 nos
	Dimension of Opening	8m x 2m

S.N	Parameters	Details
3C	**Trash rack**	
3C.1	Trash rack Opening	100 mm
3C.2	Angle of Inclination(α)	72^{o}
3C.3	Thickness of Bar	10 mm
3C.4	Submergence Depth	2 m
3C.5	Width of pier at edges	1 m
3C.6	Width of pier at center	1.5m
3D	**Settling Basin**	
3D.1	Size of Particle to settle	0.15 mm
3D.2	Number of Bays	2 nos
3D.3	Inlet Transition Length	37 m
3D.5	Total Length of Basin	144 m
3D.6	Depth of Basin	7-10 m
3D.7	Width of Flushing Canal	2 m
3D.8	Height of Flushing Canal	2 m
3E	**Head pond**	
3E.1	Effective Depth	10.05 m
3E.2	Total Depth	12.05 m
3E.3	Width of Head pond	30 m
3E.4	Length of Head pond	32 m
3E.5	Velocity in Head pond	0.25 m/s
3H	**Headrace Tunnel**	
3H.1	Shape	Inverted D shape
3H.2	Length of Tunnel	3050m
3H.3	Optimized Diameter of Tunnel	4.8 m
3H.4	Velocity through Tunnel	2.2 m/s
3H.5	Slope	1:1000
3I	**Surge Tank**	
3I.1	Type	Restricted Orifice
3I.2	Diameter	18 m
3I.3	Height	24.8 m
3I.4	Invert Level	562.45
3I.5	Up surge Level	582.83 amsl
3I.6	Down surge level	569.05amsl
3I.7	Top Level	586.83 amsl
3J	**Penstock**	

S.N	Parameters	Details
3J.1	Material	Mild Steel IS 2062-B
3J.2	Length	610 m
3J.3	Velocity of Flow	3.1 m/s
3J.4	Thickness of Shell	varying from 12 to 26 mm
3J.5	Optimum Diameter	4.4 m
3K	**Powerhouse**	
3K.1	Type	Surface
3K.2	Plan Dimension	45.50m*17.25m*26.71m
3L	**Turbine**	
3L.1	Type	Francis
3L.2	Number of Unit	2 nos
3L.3	Maximum Head	63.9 m
3L.4	Rated Capacity per Unit	11.75 MW
3L.5	Turbine Efficiency	0.9
4	**Power and Energy**	
4.1	Installed Capacity	23.54 MW
5	**Project Cost and Financial Indicators**	
5.1	Total Project Cost	NRs. 5,280,155,536
5.2	Payback Period	8 yrs.
5.3	IRR	13 %

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

1. INTRODUCTION

Our project, titled "Potential Study of the Seti Khola Hydropower Project," involves various tasks including hydrological assessments, the design of hydraulic structures, component stability evaluations, and structural analysis of the powerhouse. We will focus on designing civil structures such as the barrage, intake, desander, headrace tunnel, surge tank, penstock, and powerhouse, adhering to established design standards. Throughout the project, we will make technical assumptions and refer to relevant literature. Additionally, we will employ a range of software tools: AutoCAD for creating detailed drawings, Phase 2.0 for tunnel analysis, and E-Tabs for conducting structural analysis of the powerhouse.

1.1 Background

Nepal, with its fast-flowing rivers and steep mountains, is naturally gifted when it comes to hydropower potential. The country holds an impressive theoretical capacity of around 83,000 MW—out of which about 42,000 MW is considered economically viable. By 2025, Nepal has already tapped into around 3,400 MW, and more than 95% of the electricity produced comes from hydropower. This clean, renewable energy source is not just keeping the lights on; it's fueling development, strengthening the economy, and helping reduce our dependence on imported fossil fuels.

Zooming into Gandaki Province, this region is rich in rivers like the Kali Gandaki, Modi, and Seti—many of which are glacier-fed and flow year-round. Pokhara, the heart of this province, is nestled among these rivers and surrounded by the breath-taking Annapurna and Machhapuchhre ranges. This makes it an ideal place for hydropower projects, especially those of small to medium scale.

One of the standout initiatives here is the Seti Khola Hydropower Project. Flowing right through Pokhara, the Seti River originates from the glaciers around Annapurna III and Machhapuchhre. The project is designed to harness this river efficiently using a combination of hydraulic structures like a barrage, intake, desander, tunnels, and powerhouse components.

The Seti project isn't just about generating electricity—it's about creating local jobs, improving energy access, and supporting sustainable development in the region. With projects like this gaining momentum, Gandaki Province and Pokhara are well on their way to becoming major hydropower hubs, playing a key role in Nepal's bigger vision of producing 15,000 MW by 2035, much of it for export.

1.2 Objectives

1.2.1 General objectives

- To carry out the potential study of the Seti Khola Hydropower Project

1.2.2 Specific Objectives

- To perform hydrological analysis for the project area.
- To perform design of hydraulic components for the project.
- To perform cost and quantity calculations for construction of the designed components.
- To perform stability analysis of the hydraulic components.
- To perform structural analysis of the powerhouse.
- To compute the economic viability of the project.

1.3 Scope of Study

1. A detailed topographical and hydrological study will be conducted to serve as the foundation for the hydraulic design.
2. Hydraulic design will focus on selected major components of the project.
3. Stability analysis will be performed for specific components among those designed.
4. The powerhouse will undergo comprehensive design and structural analysis.
5. An economic analysis of the overall project will also be carried out.

1.4 Limitations

- Limited historical flow records and gap of 15 years of data affecting flood frequency analysis accuracy.
- Incomplete sediment load data making design challenging.
- No long-term climate change projections incorporated into water availability estimates.

2. DESCRIPTION OF STUDY AREA

The study area is located in Pokhara Metropolitan city ward no 12, 13, 1 of Kaski District in the Western Development Region of Nepal, presently restructured belonging to Bharat pokhari Municipality ward no 1 and 2.

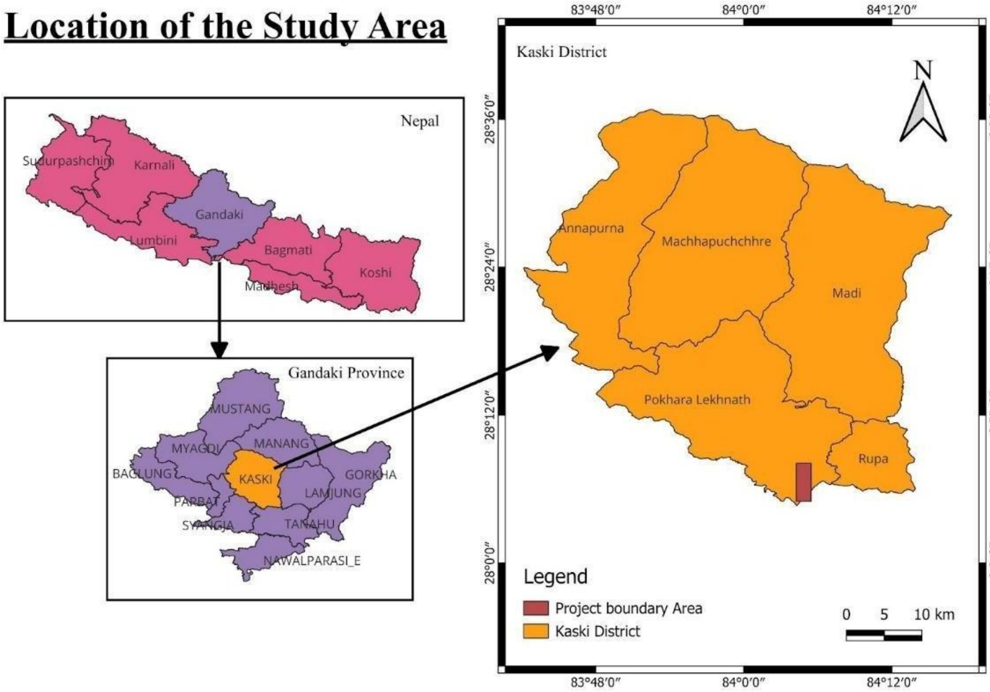

Figure 1: Project Location Map of Study Area

The project site can be viewed from Toposheet No. 2884 13A and 2884 13C (1:25000 scale). Geographically, the area for this project is located in the following license boundary:

Longitude : 84^{o} 04’ 15” E to 84^{o} 05’ 28” E

Latitude : 28^{o} 05’ 00” N to 28^{o} 08’ 05” N

By the physiographic regions, the project area belongs to the middle mountains and the upper catchment of the Seti River extends up to the elevation of 7133m above the mean sea level.

The Seti Khola is a major tributary of the Trishuli River. It is a perennial river originating from the snow fields and glaciers around the twin peaks of Annapurna III and Machapuchre in the south facing slopes of the main Himalayas. The catchment of the Khola is predominantly a rainfed one and does not lie under the permanent snow cover. However, the uppermost catchment experiences some seasonal snowfall during winter.

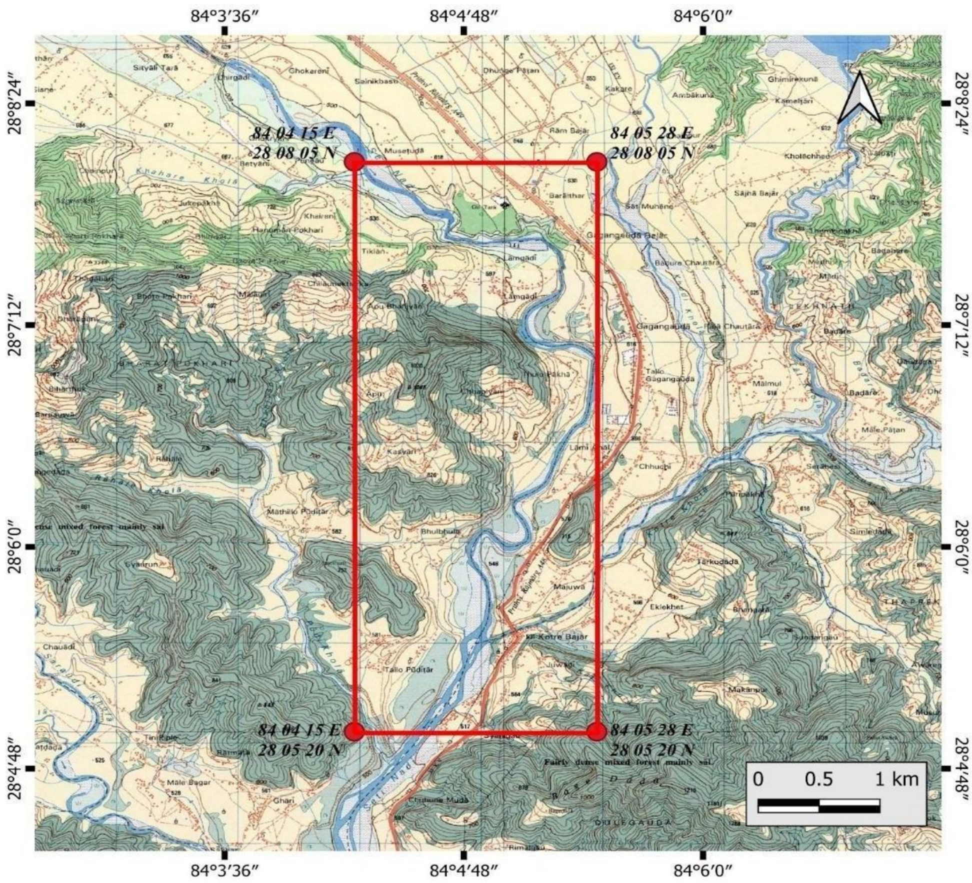

Figure 2: Project Location and License Boundary on Topo map

The project site is accessible from Badal Chowk near Lekhnath Bazzar which is situated at Prithivi Rajmarg, on the way to Pokhara from Kathandu in Kaski district, about 76 km West from Mugling. Mugling Bazar is located at about 112 km from Kathmandu and 36 km from Narayanghat on the East West Mahendra Highway.

The nearest airport is at Pokhara (Kaski), which is about 16 km from Badal chowk. There are several airlines operating daily flights between Pokhara and Kathmandu and it takes about 25 minutes to travel.

Every day, hundreds of buses leave Pokhara for various destinations such as Kathmandu, Bhairahawa, Raxaul, etc.; and there are plenty of local bus services connecting the project site with the Prithvi chowk in Pokhara Metropolitan; and Lekhnath chowk in Lekhnath Municipality.

3. LITERATURE REVIEW

3.1 Status of Hydropower in Nepal

Nepal is richly endowed with hydropower resources, boasting an estimated potential of around 43,000 megawatts (MW). Yet, despite this abundance, only a small portion of that potential has been realized. As of March 2025, Nepal's total installed electricity capacity is 3,421.956 MW, with 3,255.806 MW (95.1%) coming from hydropower. The remaining capacity includes 106.74 MW from solar, 53.41 MW from thermal, and 6 MW from co-generation. Hydropower is seen as a catalyst for regional development, particularly in underdeveloped western Nepal. The sector could contribute 87% GDP growth if 20% of Nepal’s 42,000 MW economically feasible potential is harnessed. The major hydropower are:

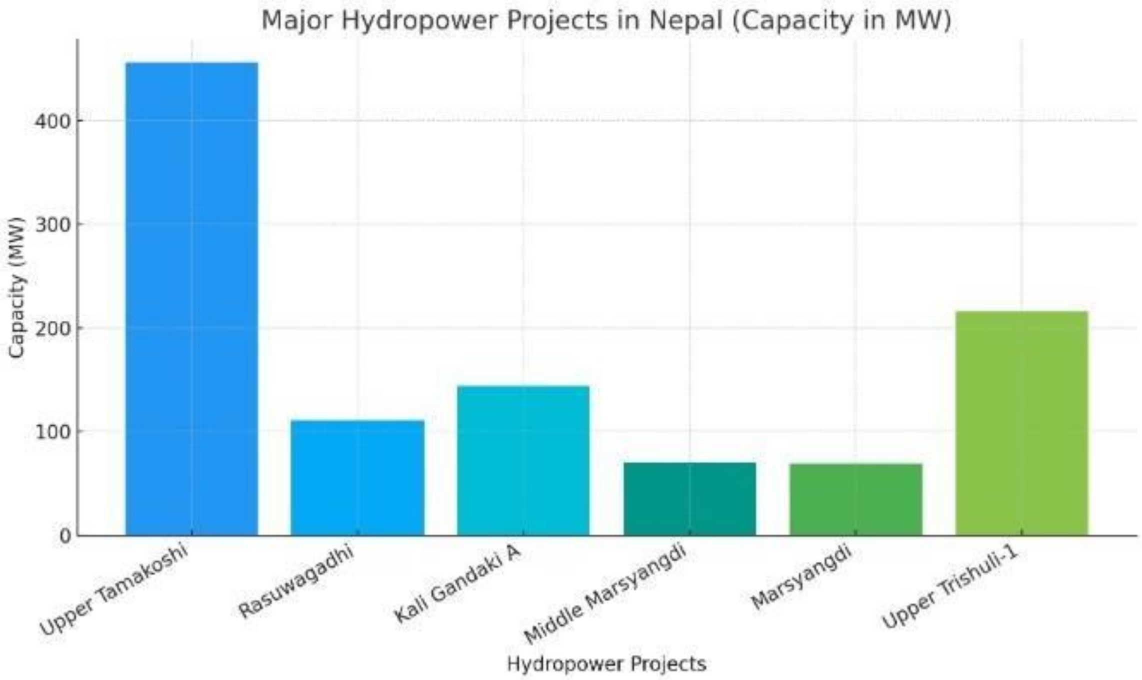

Figure 3: Hydropower Project

1. **Upper Tamakoshi (456 MW)** – Nepal’s largest hydropower project, fully operational 2021
2. **Rasuwagadhi (111 MW)** – Recently commissioned in 2024
3. **Kali Gandaki A (144 MW)** – One of the oldest major projects, operational since 2002
4. **Middle Marsyangdi (70 MW) & Marsyangdi (69 MW)** – Significant contributors to the national grid
5. **Upper Trishuli-1 (216 MW)** – Under construction with South Korean investment, expected completion by 2030

3.2 Export and Domestic Demand

Nepal has been exporting surplus electricity to India, with permitted exports reaching 632.6 MW in 2025 . Domestic demand is growing at 7–10% annually, with peak demand exceeding 2,000 MW in the wet season .However, Nepal still faces seasonal shortages in the dry season, forcing imports from India. Actually Nepal aims to generate 28,000–30,000 MW, with 15,000 MW for export by 2035AD.

Though there are a lot of challenges. The challenges are:

- Seasonal variability (surplus in monsoon, deficit in winter).
- High transmission losses (~16%) compared to global standards.
- Climate change risks (glacial melt, sediment load, floods).
- Bureaucratic delays and financing hurdles

Efforts are underway to overcome these issues. The government has prioritized hydropower development and is encouraging private sector participation. Infrastructure improvements are also underway, which should help streamline the development process. The two main policies made by government are:

- The government is promoting energy banking with India to balance seasonal surpluses/deficits.
- Private sector participation is increasing, but policy instability remains a hurdle.

3.3 Overview of Hydropower Systems

Hydropower is more than just generating electricity—it's about harnessing the natural force of water and transforming it into usable energy. At its core, hydropower engineering involves capturing the kinetic and potential energy of flowing or falling water, converting it into mechanical energy through turbines, and finally into electrical energy via generators. But it's not just about the turbines and generators; a hydropower project is a complex system with multiple components— dams, penstocks, powerhouses, and transmission lines—all working together seamlessly.

To better understand how these projects function, we classify them based on different criteria: head (water drop height), capacity (power output), operational role, storage capability, and construction design. Each classification helps engineers, policymakers, and investors determine the best approach for a given location and energy need.

3.3.1 Classification by Head

The "head" in hydropower refers to the vertical distance water travels from the intake to the turbine. This height determines the type of turbine used and the overall design of the plant. Nepal, with its steep Himalayan terrain, mostly has medium to very high-head projects.

- Very Low Head (<15m): Rare in Nepal, these plants use Kaplan turbines and are typically found in flat terrains.
- Low Head (15-60m): Common in smaller rivers, using Francis or Kaplan turbines (e.g., some mini-hydropower plants).

- Medium Head (60-150m): The most common type in Nepal, seen in projects like Upper Tamakoshi (456 MW).
- High Head (150-300m): Uses Pelton or Turgo turbines (e.g., Rasuwagadhi).
- Very High Head (>300m): Requires specialized multi-jet Pelton turbines (e.g., Upper Trishuli-1).

The higher the head, the more energy can be extracted from the same water flow, making highhead projects more efficient in mountainous regions.

3.3.2 Classification by Capacity

Hydropower plants vary dramatically in size—from tiny micro-hydro systems powering a single village to massive dams supplying entire cities.

- Micro Hydropower (<100 kW): Often community-run, providing electricity to remote villages (e.g., Ghandruk Micro-Hydro).
- Mini Hydropower (100 kW – 1 MW): Serves small towns or industries.
- Small Hydropower (1 MW – 25 MW): Feeds into regional grids (e.g., Chilime – 22.1 MW).
- Medium Hydropower (25 MW – 100 MW): Significant contributors to the national grid (e.g., Marsyangdi – 69 MW).
- Large Hydropower (>100 MW): Major national projects like Upper Tamakoshi (456 MW) and upcoming Arun III (900 MW).

Nepal's focus is shifting toward larger projects to meet growing demand and export potential.

3.3.3 Classification by Operation

Not all hydropower plants work the same way—some run constantly, while others kick in only when demand spikes.

- Isolated (Off-Grid) Plants: Serve remote areas without grid access, common in Nepal's rural regions.
- Grid-Connected Plants: Linked to the national power system (INPS). These are further divided into:
- Base Load Plants: Run continuously, providing steady power (e.g., Kulekhani).
- Peak Load Plants: Activated during high-demand periods (e.g., storage-based projects).

Most of Nepal's grid-connected plants are Run-of-River (ROR), meaning they depend on natural river flow rather than stored water.

3.3.4 Classification by Storage

Hydropower plants also differ based on whether they store water or rely on immediate river flow.

- Run-of-River (ROR) / Peaking ROR (PROR):
    - ROR: No large storage; generates power based on river flow (e.g., Khimti, Bhotekoshi).
    - PROR: Small poundage allows limited storage for daily/weekly peaking (e.g., Kali Gandaki-A, Marsyangdi).
- Storage Plants:
    - Have large reservoirs (dams) to store water for dry-season use (e.g., Kulekhani).
    - More expensive but provide stable year-round power.

Nepal currently has very few storage plants, leading to dry-season shortages despite surplus power in monsoons.

3.3.5 Classification by Construction

The physical design of a hydropower project depends on geography and purpose.

- Valley-Type Plants:
    - Feature a dam creating a reservoir (e.g., Kulekhani).
    - Powerhouse is usually at the dam's base.
- Diversion-Type Plants:
    - Water is diverted through canals/tunnels to a distant powerhouse (e.g., Upper Tamakoshi).
    - Common in steep terrains where damming a valley isn't feasible.

Most of Nepal's projects are diversion-based due to the mountainous landscape.

3.4 Flood Analysis in Hydropower Projects

Floods pose a significant risk to hydropower infrastructure, potentially damaging dams, turbines, and powerhouses. To mitigate these risks, engineers conduct flood analysis—a process that predicts flood intensity and frequency using historical data and statistical models. This analysis helps design spillways, diversion channels, and flood-resistant structures, ensuring hydropower projects remain safe and operational even during extreme weather events.Three key statistical methods are commonly used in flood analysis:

3.4.1 Gumbel Method (Extreme Value Type I Distribution)

The **Gumbel Method** estimates the probability of extreme flood events by analyzing historical peak flood data. It assumes that the largest annual floods follow a predictable pattern, allowing engineers to calculate flood risks for different return periods (e.g., 50-year or 100-year floods).

3.4.1.1 Steps for Gumbel method:

- **Step 1:** Collect historical flood peak data.
- **Step 2:** Fit the data to a **Gumbel distribution** (a statistical model for extreme values).
- **Step 3:** Calculate the **mean ($\bar{x}$)** and **standard deviation (σ)** of flood peaks.
- **Step 4:** Determine the **frequency factor (K)**, which adjusts for different return periods.
- **Step 5:** Estimate the flood discharge for a given return period using:

3.4.1.2 Pros & Cons:

Simple and widely used for long-term flood prediction.

Works well with consistent historical data**.**

Underestimates extreme floods in regions with changing climate patterns.

Less accurate if data is limited.

3.4.1.3 Application in Nepal:

Used in projects like **Upper Tamakoshi** to design spillway capacity.

3.4.2 Log Pearson Type III Method

The **Log Pearson Type III (LP3) method** is a **statistical flood frequency analysis** technique that improves upon simpler models by accounting for **skewness** (asymmetry) in flood data. Unlike the Gumbel method (which assumes symmetry), LP3 is better suited for rivers with **irregular flood patterns**, such as those in Nepal affected by monsoons, glacial melt, and sudden cloudbursts.

This method is recommended as one of the method of flood discharge analysis as per guideline of DOED. Equation for discharge with return period T.

For Log Pearson III Method,

$$Z_T = Z_{avg} + K_z \sigma_z$$

$$= \log()$$

Where,

$_v$ = Mean of the Z values = Percentage Chance

=Standard Deviation of the Z variate sample = $\sqrt{\left\{\frac{\sum(Z-Z_{avg})^2}{N-1}\right\}}$

$(-v)3$

= G = Coefficient of skew of variate Z = $\frac{(-v)3}{(\quad -1)(-2)()3}$

3.4.2.1 Pros & Cons:

Better for irregular flood patterns (common in Himalayan rivers).

Official standard in Nepal for hydropower flood design.

Complex calculations (needs specialized software).

3.4.2.2 Application in Nepal:

Used in **storage dam projects** like **Budhi Gandaki** to assess long-term flood risks.

3.4.3 Log Normal Method

A simpler alternative to Log Pearson III, the **Log Normal method** assumes flood data follows a **logarithmic normal distribution**. It's useful for quick assessments but less precise for extreme events.

3.4.3.1 How It Works:

- **Step 1:** Convert flood peaks to logarithmic values.
- **Step 2:** Compute the **mean and standard deviation** of the log-transformed data.
- **Step 3:** Estimate flood discharge using normal distribution tables.

3.4.3.2 Pros & Cons:

Easy to apply for preliminary studies.

Works well with **moderate flood variability.**

Less accurate for rare, extreme floods.

Not ideal for **high-risk projects.**

3.4.3.3 Application in Nepal:

Sometimes used in **small hydropower projects** with limited data.

3.4.3.4 Requirement of Flood Analysis for Hydropower in Nepal

- **Himalayan rivers** are prone to **glacial outburst floods (GLOFs)** and monsoon surges.

- **Climate change** is increasing flood unpredictability.
- **Designing spillways and dams** requires accurate flood estimates to prevent **catastrophic failures.**

3.4.3.5 Which Method to Choose?

Table 1: Comparision of various methods

Method	Best For	Limitations
Gumbel	Long-term data, stable climates	Underestimates extreme floods
Log Pearson III	Skewed data, DOED compliance	Data-intensive, complex
Log Normal	Quick feasibility studies	Less precise for rare floods

Flood analysis is **critical for hydropower safety** in Nepal, where extreme weather and glacial melts pose growing risks. While the **Gumbel method** offers simplicity, **Log Pearson III** is the **gold standard** for accuracy. Engineers must choose the right method based on **data availability, project scale, and risk tolerance.**

3.5 Components of Hydropower

3.5.1 Weir

A diversion weir is a low-height structure constructed across a river to divert a portion of the river flow into an intake system without creating a large reservoir. It is commonly used in run-of-river (RoR) hydropower schemes, where minimal water storage and ecological disruption are priorities. Instead of impounding large volumes of water like dams, diversion weirs raise the water level just enough to facilitate controlled diversion into a headrace canal or tunnel.

Purpose and Functions

The diversion weir serves several critical functions in a hydropower scheme:

- **Elevates the Water Level:** Raises the upstream water level to divert flow into the intake structure.
- **Regulates Flow:** Helps regulate the volume of water entering the system, ensuring design discharge is maintained.
- **Sediment Control:** Works in conjunction with a desanding basin to reduce sediment entering the headrace tunnel.
- **Flow Monitoring:** Often includes flow measurement devices to monitor diverted discharge.

Structural Types

- **Fixed Crest Weir:** Simple, permanent overflow structure; most common in small to medium RoR schemes.
- **Gated Weir:** Incorporates radial or vertical gates to control flow, useful in variable flow regimes.
- **Ogee Weir:** Curved profile offering better hydraulic efficiency; often used in higher-flow designs.

Material Selection

Diversion weirs can be constructed using:

- **Reinforced Concrete:** Most common material for strength and durability.
- **Masonry or Stone:** In small-scale or traditional projects.
- **Composite Materials:** In remote or temporary projects where cost and logistics are critical.

3.5.2 Barrage:

A barrage is a type of hydraulic structure built across a river to control water levels and flow by using adjustable gates. Unlike a weir, which is a fixed structure, a barrage allows operators to regulate river discharge and divert water as needed for irrigation, water supply, or hydropower. In hydropower applications, barrages are often used in low-head schemes or where greater control over flow regulation is required.Barrages offer significant operational flexibility in hydropower projects, particularly in managing variable flows and sediment. While not always necessary for run-of-river systems, understanding barrage design is important for future scalability and integrated water resource management. In specific scenarios, upgrading from a diversion weir to a gated barrage could provide long-term benefits, though it requires careful hydraulic and economic justification.

Purpose and Functionality

The primary functions of a barrage in hydropower include:

- **Flow Regulation:** Adjusts river discharge by controlling the opening and closing of gates, especially important during flood seasons or dry periods.
- **Water Diversion:** Helps divert controlled amounts of water to power canals or tunnels for energy generation.
- **Water Level Maintenance:** Maintains the upstream water level needed to achieve the required hydraulic head.
- **Sediment Flushing:** Can allow flushing of sediments during high flows using undersluices or scouring sluices.

- **Multipurpose Use:** Often used for irrigation and navigation in addition to hydropower.

Components of a Barrage

A typical barrage structure includes:

- **Gated Bays:** Vertical or radial gates that control flow across the river width.
- **Undersluices/Scour Sluices:** Used to flush sediment and maintain a clean intake.
- **Divide Walls and Fish Ladders:** To separate flow zones and allow fish migration.
- **Deck and Piers:** Supporting structure for the gates and operational platform.
- **Energy Dissipators:** Stilling basins, baffle blocks, or apron slabs to reduce flow energy.

Design Considerations

Designing a barrage involves hydraulic, structural, sedimentation, and operational analyses:

- **Discharge Capacity:** Must safely pass maximum design flood (often 100-year or PMF).
- **Gate Operation:** Automated or manual gates for real-time flow regulation.
- **Foundation Stability:** Requires thorough geotechnical studies to prevent piping or uplift failures.
- **Sediment Management:** Layout designed to reduce sediment deposition near the intake.
- **Scour Protection:** Adequate protection downstream with riprap or reinforced apron.

3.5.3 Undersluice

An undersluice is a low-level gated opening provided in diversion structures like weirs and barrages. Its primary function is to remove sediments, especially coarse bedload materials like sand, gravel, and silt, from the river flow before it enters the intake. This helps maintain the efficiency and longevity of downstream components such as headrace tunnels, penstocks, and turbines in a hydropower system. Undersluices are vital sediment management components in hydropower projects, especially in Himalayan rivers with high silt loads. Their strategic design and timely operation ensure protection of the intake and power generation components.

Purpose and Importance

Undersluices play a critical role in sediment management. Their main functions include:

- **Sediment Flushing:** Scours and flushes sediments from the riverbed, especially during high flows.
- **Intake Protection:** Prevents excessive sediment from entering the intake and damaging turbines.

- **Water Quality Control:** Ensures relatively cleaner water enters the intake, reducing turbidity-related issues.

Design Features

Undersluices are usually located near the intake side of the weir or barrage and include the following components:

- **Low-Level Gates:** Radial or vertical gates that open to allow sediment-laden flow to pass.
- **Scouring Channel:** A trough-like channel to guide sediments downstream.
- **Divide Wall:** Separates the undersluice section from the intake to improve hydraulic performance.

Key design considerations include:

- **Gate Width & Number:** Based on sediment load and river width.
- **Invert Level:** Set lower than the riverbed to induce scouring action.
- **Energy Dissipation:** Stillings basins or baffle blocks downstream to reduce flow velocity.
- **Operation Timing:** Typically operated during high flows or flood events for effective flushing.

Relevance to Run-of-River Projects

In Run-of-River (RoR) hydropower projects, especially those located in mountainous regions like the Seti Khola basin, undersluices are highly relevant due to the high sediment load carried by rivers during monsoon and flood events. RoR schemes lack large reservoirs to naturally settle sediments, making them more vulnerable to the abrasive effects of silt and gravel on hydraulic structures and turbines. Undersluices provide an effective sediment flushing mechanism by allowing the controlled removal of heavier bedload materials from the riverbed before water is diverted into the intake. This not only protects the downstream infrastructure such as desanding basins, tunnels, and turbines, but also helps maintain the long-term efficiency and sustainability of the project. Therefore, integrating well-designed and strategically operated undersluices is essential in RoR projects to ensure smooth operation, reduced maintenance, and prolonged equipment lifespan.

3.5.4 Intake

The intake structure in a hydropower project is a crucial component that diverts water from the river into the power system while preventing unwanted materials like sediments, debris, and floating objects from entering. In Run-of-River (RoR) hydropower projects, where water is diverted without significant storage, the intake must function efficiently under fluctuating flow conditions. A well-designed intake ensures a steady supply of the design discharge while minimizing sediment entry, which is particularly important in rivers with high sediment load, such as those originating from glacial and monsoon-fed sources.The intake structure is a critical component of any hydropower system, especially in Run-of-River schemes where sediment management and flow regulation are primary challenges. A well-designed intake not only improves operational efficiency but also reduces turbine wear, maintenance costs, and power generation interruptions.

Types of Intake

In hydropower projects, especially Run-of-River (RoR) schemes like the Seti Khola Hydropower Project, the intake structure plays a critical role in ensuring the smooth diversion of water while minimizing the entry of sediments and floating debris. The design and selection of the intake structure depend heavily on site-specific factors such as river morphology, flow variation, sediment concentration, and the terrain of the project area. Several types of intake structures are used in practice, each with distinct advantages suited to different hydrological and geographical conditions. These include:

- **Side Intake:** Commonly used in RoR projects, side intakes are constructed along the riverbank and are typically aligned tangentially or obliquely to the river flow. This orientation helps to naturally reduce the entry of sediments, especially when combined with a guide wall and undersluice system. It is particularly effective in rivers with high sediment loads, like those in the Himalayan region.

- **Tyrolean Intake:** Best suited for steep, sediment-heavy mountain streams, Tyrolean intakes consist of closely spaced metal bars or slots placed across the intake channel. These slots allow only clear water to enter while rejecting coarse bedload materials, pebbles, and floating debris. This type is simple, low-maintenance, and highly effective in remote areas where sediment exclusion is a major concern.

- **Canal or River Intake:** This intake is located directly on the riverbank and diverts water into an open canal or headrace. It usually includes a trash rack and flow control gates. This type is suitable for low- to medium-sediment rivers and is relatively easy to construct and maintain.

- **Reservoir Intake:** Used in storage-based hydropower plants, this type of intake is submerged within a reservoir and may feature multiple openings at different elevations. This allows operators to control the quality and temperature of water withdrawn. It is not common in RoR projects but is essential in large dam-based schemes.

- **Shaft or Vertical Intake:** This type of intake draws water vertically down into a tunnel or penstock, usually from a forebay or reservoir. It is often used in high-head hydropower projects situated on steep terrain, where horizontal conveyance is impractical. These intakes typically include vortex suppression devices to ensure stable flow.

- **Bell-Mouth Intake:** Characterized by a flared, trumpet-like opening, bell-mouth intakes are designed to reduce turbulence and vortex formation, ensuring smooth entry of water into the system. These are mostly used in submerged conditions such as forebays or deep reservoirs, and they are highly efficient in terms of hydraulic performance.

- **Gated Intake:** Gated intakes come equipped with mechanical gates that control the flow of water entering the system. They offer operational flexibility and are often used in combination with other intake types to enable flow regulation, sediment flushing, and emergency shutdown.

Purpose and Functions

The intake structure plays a critical role in the hydropower system, performing several key functions:

- **Water Diversion:** Ensures a controlled and steady flow into the conveyance system.
- **Sediment Exclusion:** Prevents excessive silt and bedload materials from reaching the desanding basin and penstock.
- **Debris Control:** Trash racks and screens prevent logs, leaves, and other floating objects from entering the system.
- **Flow Regulation:** Gates and control mechanisms allow for adjustments to discharge as per operational needs

Design Features

The design of an intake structure depends on site-specific hydraulic conditions and sediment transport characteristics. The key features include:

- **Trash Racks and Screens:** Installed at the entrance to prevent debris from entering.
- **Control Gates:** Regulate water flow into the intake and can be closed during high sediment loads.
- **Silt Excluders:** Designed to prevent excessive fine sediments from entering the system.
- **Approach Channel:** Guides the flow smoothly towards the intake, reducing turbulence and sediment disturbance.

The location of the intake is also a crucial factor in its performance. In RoR schemes, intakes are typically placed at a stable river section where sediment deposition is minimal, often near a diversion weir or barrage.

3.5.5 Settling Basin:

A settling basin, also known as a sedimentation tank or desilting basin, is a critical component of run-of-the-river hydropower schemes, particularly in sediment-laden rivers like those found in the Himalayan region. Its primary function is to remove bed load and suspended sediments— especially abrasive materials like sand and silt—from the water before it enters the headrace tunnel and turbines. This not only enhances the efficiency of the turbines but also significantly reduces wear and tear, thus extending the life of electromechanical equipment.The settling basin is a pivotal element in the design of the Seti Khola Hydropower Project. Given the sediment-prone nature of the river, careful attention to design standards, sediment characteristics, and operational planning is essential. A well-functioning settling basin ensures not only technical and economic efficiency but also enhances the project's sustainability in the long run.

Design Parameters

The design of settling basins revolves around key hydraulic and sediment transport principles. Common design parameters include:

- **Settling velocity:** Governed by Stokes' Law for fine particles.
- **Retention time:** Which ensures particles have enough time to settle.
- **Flow velocity:** Which must be reduced below the critical velocity that re-suspends particles.

As per IS 9761:1995 and HEC-6 guidelines, the following criteria are commonly used in Nepalese hydropower projects:

Particle size to be removed: ≥ 0.15 mm

Basin length: Determined by desired particle settling time.

Basin Types and Arrangements

Depending on project scale and sediment characteristics, settling basins may be configured as:

- **Conventional horizontal-flow basins**
- **Inclined-plate settlers (Lamella plates):** For space efficiency
- **Vortex settling chambers:** For coarse sediment removal

In Nepal, multi-chamber basins with flushing galleries are increasingly common to accommodate monsoonal sediment loads and facilitate easier maintenance.

Sediment Flushing and Maintenance

Sediment flushing is essential to maintain the capacity of settling basins. Modern basins include:

- **Bottom flushing outlets**
- **Automated sediment monitoring systems:** This allows for real-time adaptation during high sediment events, especially crucial in rivers like the Seti Khola that carry significant glacial silt.

Importance of Settling Basins

Numerous studies highlight the importance of settling basins in maintaining long-term sustainability and operational efficiency of hydropower plants. Sediment exclusion through welldesigned settling basins can reduce maintenance costs and improve energy output consistency. In the Himalayan rivers, where sediment concentration peaks during monsoon, settling basins become indispensable.

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

Design Criteria:

i) The settling basin shall be designed to be functional, easily operable and economical, both for construction and operation.
ii) The design shall attempt to remove as much of the coarser fractions of the suspended load as possible so that the hydraulic transport capacity of the water conveyance system can be maintained and the sediment load to the turbines, valves, etc. is reduced to acceptable limits.
iii) The settling basin shall be designed to ensure efficient flushing of the settled sediments so, that frequent flushing during floods, when the sediment content of the rivers is at peak, is not required.

Design considerations:

Following factors should be considered in the design of the settling basin

i) Characteristics of suspended sediments, particles of hard or soft rock or soil origin
ii) Concentration of sediments with the river flow
iii) Required removal percentage of suspended sediments

Design assumption:

The design of the settling basin shall be based on the following assumptions:

i) The sediment concentration of flow entering the settling basin is equal to the design sediment concentration of the river flow.
ii) Supply of the design flow to the power canal takes place with simultaneous settling and flushing of sediments or with sediment settling followed by intermittent flushing.

iii) The water level in the basin is horizontal.

iv) In continuous flushing, the depth of flow in the basin is constant, but the flow regime is non-uniform due to change in flow discharge along the basin length during sediment flushing.

v) Average velocity of flow in the settling basin is constant and does not change in time or space.

vi) The settling velocity is not affected by the temperature of water.

vii) The vertical distribution of sediments may be triangular, rectangular or trapezoidal and flushing of settled sediments takes place in uniform regime of flow.

Design of settling basin:

Calculate surface area,

$$= K/w$$

Where,

A_s = the required surface area of a settling basin

w = target sediment particle size with a fall velocity

Q = the design flow rate

K = coefficient to account for turbulence (range between 1.2 to 1.5)

1. Calculate L and B using the relation / = 4 to 10

$$A_s = L.B$$

Check for necessary length and breadth using other approaches

2. Calculate the limiting flow velocity (V_c) in the basin

$$= \surd$$

Where,

D = size of gravel to be removed in mm

a = 0.36

for d>1 mm (about 0.04 in) = 0.44

for 1 mm (about 0.04 in) > d >0.1 mm (about 0 in) = 0.51

for 0.1 mm (about 0 in) >d

Note: The maximum horizontal velocity of flow to allow the sand grain to fall out should not be greater than 0.3 m/s

3. Calculate depth (D) of basin

$$= \frac{}{H}$$

Where,

Q = design flow rate (m^3/s)

B = width of the settling basin

V = horizontal flow velocity (m/s)

4. Check the calculated values of L, B,H

Check Width,

$$= 4.75\sqrt{}$$

M.A. Velikanov's Method,

$$= \frac{(\lambda\, 2v\, 2\, (\sqrt{h} - 0.2)\, 2\,)}{7.51w\text{^}2} m$$

Where,

λ depends on the removal ratio, defined by the function $\lambda = f(W)$

W denotes the ratio of settled sediment to the total load entering with the flow and is given by:

$$= 100 - 100\frac{Cp}{C}\text{Percent}$$

5. Check the efficiency of the particle settled

Efficiency of the settling basin can be calculated using **Hazen equation and Vetter equation.**

6. Compute the sediment depth in the basin

$$\text{Volume of sediment} = \frac{\text{Sediment load}}{\text{Density} * \text{Packing factor}}$$

Basin plan area =

$$\text{Depth of settlement, } = \frac{\text{Volume of sediment}}{\text{Basin Plan Area}}$$

7. Length of basin considering turbulence effect:

$$L = \frac{D^{\frac{4}{3}}}{\omega\sqrt{D} - 0.132V} D^{4/3}$$

Where,

L = length of basin

ω = fall velocity

V = horizontal flow velocity

D = Depth of basin

8. **Flow depth of chamber during flushing**

For given flushing velocity, the flow depth of chamber during flushing is

$$D_f = \frac{Qf}{B*Vf}$$

Where,

D_f = flow depth of chamber during flushing

Q_f = Adopted flushing discharge

V_f = Flushing velocity

3.5.6 Head Pond

A head pond—also referred to as a pondage or forebay—is a small water storage reservoir located immediately upstream of the headrace intake in a hydropower project. It plays a vital role in regulating flow, optimizing turbine operations, and managing short-term fluctuations in river discharge. In run-of-river schemes, where there is limited or no seasonal storage, the head pond provides essential daily or hourly flow regulation. Additionally it prevents from the formation of vortex at inlet pipe or tunnel which decrease the chance of forming cavitation.

Functions and Importance

Numerous studies emphasize the multifunctional role of the head pond in enhancing hydropower project efficiency:

- **Flow Regulation:** The head pond helps manage variable river discharge and ensures a more consistent flow to the turbines during short-term changes in water availability.
- **Sediment Settling (Secondary Role):** While not its primary function, a head pond can aid in initial sedimentation, especially coarse bedload, before water enters the settling basin.
- **Turbine Operation Optimization:** Short-term pondage allows power plants to meet peak electricity demand by storing water during low-demand hours and releasing it during peak periods.

- **Emergency Buffer:** It provides a buffer for sudden operational stops or maintenance shutdowns.

Design Considerations

Designing an efficient head pond requires understanding hydrological, geological, and operational parameters. Key considerations include:

- **Pond Volume and Storage Duration:**Typically sized for 1 to 4 hours of full plant discharge for daily peaking plants (Baral, 2020).
- **Embankment Design:** IS 7114 recommend using impermeable cores and adequate freeboard to prevent overtopping.
- **Spillway and Bypass Arrangements:** Important to handle sudden flood surges or excess inflow during monsoon seasons.
- **Sediment Management:** Sediment traps and coarse screens at the inlet of head ponds are critical in Himalayan rivers.

Design consideration of the head-pond:

The head-pond should be dimensioned to meet out its function and normal operation of the plant. The minimum capacity of head-pond is normally kept corresponding to a storage of 2 to 3 minutes of the design discharge with an operating depth of 2 to 3 meters. The dimensioning of head-pond is so adjusted to meet out required volume of storage and accommodate its appurtenances. Thus, the hydraulic design parameter of head-pond is determined below:

i) Capacity of head-pond should be 20 to 80 Q_d, where Q_d is the design discharge of headrace channel in cumecs.
ii) Width of head-pond is determined in accordance with the topographical condition at the site. In case of the steep terrain the head-pond can have less width and more length and should be aligned
iii) along the contour. In case the penstocks emerge from another end of the head-pond, the width of the head-pond should be enough to accommodate the penstock. The penstock can also take off perpendicular to the length of the head-pond depending upon the location of the powerhouse.
iv) Maximum water depth of the flow in head-pond is generally kept from 2 -3 meter below the steady state level with an average flow velocity in head-pond less than 0.5 m/s or even less so than it can be expected to settle out the harmful sediment's particles.
v) After fixing the width and depth of the tank, the length can be determined. The length should be aligned along the existing contour.
vi) Submergence depth of intake:

The minimum submergence required at the center line of penstock below Minimum Drawdown Level (MDDL) against air entertainment may be calculated by the following formulae given by Gordon,

$$h = 0.5 + 2\mathrm{Fr}$$

Where,

H = depth of submergence at intake center line below MDDL

D = depth of opening at intake gate axis

Fr = Froude number = $v\ \surd$

V = velocity of flow through intake

g= acceleration due to gravity

In order to achieve the required submergence, the bottom of the head-pond floor is normally depressed at the location of the penstock. The head-pond floor be further depressed by about 0.5m to 0.75m below the bottom level of penstock to accommodate the silt flushing sluice.

vii) Transition between HRC and head-pond

A suitable expansion transition having a side splay of 5:1 and bed slope of 4(H):1(V) may be provided to join headrace channel with head-pond.

viii) Spillway

In case of small hydropower project an ungated spillway crest of suitable length at the steady state level of head-pond is designed for the design discharge of head race channel for a limited head of about 0.3 to 0.5 m over the crest in order to dispose of the incoming discharge at the time of sudden tripping of the machines. The spillway crest is ogee shaped and is aligned along the wall of head-pond in such a way so that it could be discharged safely through a spill channel/ pipe into nearby drain or river.

3.5.7 Head Race Tunnel:

The headrace tunnel is a fundamental hydraulic structure in hydropower systems, particularly in run-of-river projects situated in hilly or mountainous terrain. It transports water from the settling basin or forebay to the surge tank or penstock at high efficiency, using gravity flow.

Role and Importance

- **Efficient Water Conveyance:** The headrace tunnel enables efficient transfer of water over uneven terrain without significant energy loss.

- **Alignment Optimization:** It allows the designer to bypass sharp river bends or valleys, optimizing the hydraulic head and reducing environmental impact.

- **Protection from External Hazards:** Compared to surface canals, tunnels offer protection from landslides, floods, and sediment intrusion.

Design Considerations

Designing a headrace tunnel involves several hydraulic, geotechnical, and structural aspects:

- **Tunnel Length and Gradient:** The gradient must ensure subcritical (free surface) or pressurized flow, depending on the tunnel type. Ideal slope: 1:1000 to 1:500 for free-flow tunnels.

- **Cross-Section Shape:** Circular: Common for pressurized flow (TBM tunnels).D-shaped or horse-shoe: Preferred for non-pressurized flow (drilled and blasted tunnels).Cross-section size depends on design discharge, velocity limits (1.5–3.5 m/s), and sediment concerns.

- **Flow Type:Free-flow:** Lower construction cost and simpler maintenance.Pressurized: Allows steeper gradients and higher discharge but requires more structural strength.

- **Lining:** Unlined (only in competent rock): Cost-effective but riskier in sediment-prone zones.Shotcrete, RCC, or Steel lined: Increases durability and reduces friction losses.

- **Velocity Control:** High velocities cause erosion; low velocities allow sediment deposition.Typical design velocity: 1.5 to 3.0 m/s, depending on sediment size and material.

Construction Methods

- **Drill and Blast:** Traditional and widely used in Nepal (e.g., Modi, Marsyangdi projects).

- **Tunnel Boring Machine (TBM):** Expensive but faster and more precise; rarely used in smaller run-of-river projects.

- **New Austrian Tunneling Method (NATM):** Combines flexibility and safety in weak geological conditions.

3.5.8 Surge Tank

A surge tank is a vital protective structure in high-head hydropower schemes, installed between the headrace tunnel and the penstock. Its primary function is to absorb pressure fluctuations and regulate water hammer effects caused by rapid load changes in turbines. In steep and mountainous terrains like the Seti Khola basin, surge tanks play a critical role in maintaining hydraulic stability and structural safety.

Importance and Functionality

According to Sanjeev Baral (Hydropower Engineering) and Singh (2008), the surge tank performs the following key roles:

- **Water Hammer Control:** When load on the turbine decreases or shuts suddenly, water in the headrace tunnel continues moving due to inertia, causing a sudden pressure rise (water hammer). The surge tank absorbs this excess pressure.
- **Flow Regulation:** When turbine load increases rapidly, the surge tank provides additional flow to meet demand until steady flow from the headrace catches up.
- **System Stability:** It reduces pressure oscillations and protects the penstock and turbine from transient forces.

Types of Surge Tanks

Surge tanks are classified based on design and hydraulic behavior. Common types include:

- **Simple Surge Tank**An open vertical shaft connected to the headrace just before the penstock. Best suited for small to medium flow variations. Economical and widely used in Nepal.
- **Restricted Orifice Surge Tank**Has a constricted opening at the base.Improves damping of oscillations but may delay flow response.Suitable when space is limited or when pressure damping is critical.
- **Differential Surge Tank**Features an inner riser pipe for rapid flow response and an outer chamber for damping.Offers a balance between fast flow adjustment and oscillation control.Used in high-head schemes with rapid load changes.
- **Inclined Surge Shaft**Used when vertical space is limited.Designed along the slope, maintaining function with site topography.

Design Considerations

Several factors influence the design of a surge tank:

- **Location:** Should be as close to the penstock as possible.
- **Height and Diameter:** Depends on flow rate, head, tunnel length, and permissible oscillation.
- **Velocity of Flow:** Must be controlled to avoid cavitation or vacuum conditions.
- **Stability Analysis:** Surge tank behavior is modeled using mass oscillation theory or St. Venant equations, and modern tools like Hydraulic Transient Analysis (HTA) are used for precise simulation.

3.5.9 Penstock

The penstock is a high-pressure conduit that delivers water from the surge tank (or directly from the forebay) to the turbine in a hydropower plant. It converts the potential energy of stored water into kinetic energy for power generation. In medium- and high-head run-of-river projects like Seti Khola, the penstock is a key structural and hydraulic component that must be carefully designed for pressure, durability, and efficiency.

Function and Importance

Energy Transmission: Penstocks direct water under pressure to the turbine, ensuring maximum conversion of head into mechanical power.

Hydraulic Control: Proper sizing and layout minimize energy loss due to friction and turbulence.

System Safety: Penstocks must withstand internal pressure fluctuations, including water hammer, and external forces such as landslides or seismic loads.

Types of Penstocks

Penstocks are classified based on material, layout, and configuration:

a) Based on Material

- **Mild Steel Penstock:** Most common in Nepalese projects.Suitable for medium to high pressure.Requires protective coatings (anticorrosive paint, bitumen).
- **Fiberglass Reinforced Plastic (FRP) or HDPE:**Used in low- to medium-head systems.Corrosion-resistant and light but not suited for very high pressures.
- **Concrete (Embedded) Penstocks:**Used in very large projects or short high-capacity conduits.Expensive and rigid.

b) Based on Configuration

- **Surface Penstocks:** Laid above ground, supported by anchor blocks and saddles.Common in hilly terrain.
- **Buried Penstocks:** Preferred when temperature fluctuation or aesthetics are a concern.
- **Inclined Penstocks:** Follow terrain slope, used in projects with steep head drops.

Design Considerations:

a) Hydraulic Design

- **Flow Velocity:** Typical design velocity: 2–5 m/s (Baral, 2020).
- **Friction Loss:** Calculated using Darcy-Weisbach or Hazen-Williams formulas.
- **Water Hammer Pressure:** Analyzed using Joukowsky equation to determine surge pressures during valve closure.

c) Structural Design

- **Wall Thickness:** Determined based on internal pressure, using the Hoop Stress formula:

$$t = \frac{p \times D}{2 \times f \times \eta}$$

Where:

t = wall thickness

p = internal pressure

D = internal diameter

f = allowable stress

η = safety factor

- **Anchor Blocks and Support Saddles:** Needed at every bend, valve, or change in gradient.
- **Air Vents and Surge Protection** Prevents vacuum formation and controls air entrapment. Especially crucial during start-up/shutdown of turbines.

3.5.10 Anchor Block

An anchor block is a reinforced concrete structure used to restrain the movement of pressure conduits—especially penstocks—by resisting the hydraulic thrust forces generated at bends, bifurcations, reducers, and other locations where the direction or velocity of water changes. In hilly and mountainous terrains like the Seti Khola basin, where pressure conduits are typically laid on steep slopes, anchor blocks are crucial for mechanical stability and safety.

Role and Importance of Anchor Blocks

Anchor blocks are essential components in hydropower schemes due to the following roles:

- **Resist Hydraulic Thrusts:** At bends, valves, and transitions, high-velocity water exerts sudden directional and pressure forces. Anchor blocks resist these forces and keep the penstock in place (Baral, 2020).
- **Stabilize the Penstock System:** They prevent longitudinal sliding, uplift, or bursting of pipes under pressure surges and transient forces.
- **Control Expansion:** Anchor blocks act as fixed supports, separating pipe segments fitted with expansion joints or saddles to allow for thermal movement

Types of Anchor Blocks

Anchor blocks are designed and classified based on their functional locations:

- **Bend Anchor Blocks:** Used at horizontal or vertical bends in penstocks. Must resist radial thrusts generated by centrifugal force and change in momentum.

- **Valve Anchor Blocks:** Installed near shut-off valves or air valves where sudden pressure fluctuations are likely. Designed for high transient forces (e.g., water hammer effects).

- **Reducer or Expander Anchor Blocks:** Located at transitions where pipe diameter changes. Resist axial thrust due to pressure differential across cross sections.

- **End Anchor Blocks:** Fixed point at the beginning or end of the penstock system (e.g., near surge tank or turbine inlet).

Design Considerations

Anchor blocks are typically constructed using reinforced concrete (RCC) and designed based on hydraulic thrust calculations and site-specific conditions:

Forces to Consider

a) Hydraulic Thrust (T): From internal water pressure and change in flow momentum. Pipe Weight and Water Weight

External Load: Including soil pressure, seismic activity, or landslides in mountainous areas **b)**

Design Formula for Thrust Force.

$$= \frac{\times}{c}$$

Where:
T = thrust force
P = internal pressure

A = internal area of pipe

θ = bend angle (for elbows)

Additional design check: sliding, overturning, uplift resistance, and bearing capacity.

c) Foundation Anchor blocks must be embedded into natural ground or bedrock.In poor soils, additional ground improvement or rock anchors may be used.

d) Materials and Reinforcement Concrete: Grade M25 or higherSteel: Fe500, designed per IS 456:2000 and IS 3370

3.5.11. Powerhouse

Power house is a building provided to protect the hydraulic and electrical equipment. Generally, thes whole equipment is supported by the foundation or substructure laid for the power house. In case of reaction turbines some machines like draft tubes, scroll casing etc. are fixed with in the foundation while laying it. So, the foundation is laid in big dimensions. When it comes to super structure, generators are provided on the ground floor under which vertical turbines are provided. Besides generator horizontal turbines are provided. Control room is provided at first floor or mezzanine floor.

3.5.11.1 Architectural layout of power house

Three essential constitutes (bay) of superstructure of powerhouse are:

1. Unit Bay
2. Erection or Loading Bay
3. Control Bay

Unit Bay

- Length:

For determination of unit bay length, 1.5m to 2m clearance is added on either side of the hydro mechanical equipment and the unit spacing is determined. Length of erection bay is taken as 1 to 1.5 times the unit bay. The total length of power house for placement of Hydromechanical equipment can be determined by the formula:

$$L = \times (n\,pn) + +$$

Where, L = Total length of powerhouse required for placement of hydro-mechanical equipment

= Number of units

= Length of erection bay

K = Length required for the E.O.T crane to handle the last unit. (3 to 5m)

- Width:

Width of superstructure column should be 2 to 2.5m from the extremities of hydro-mechanical equipment on the downstream and 3 to 4m on the upstream.

- Height:

Total height of the power can be calculated as

$$H = H_1 + H_2 + H_3$$

Where, H= total height of power house

H_1 = Vertical Length of draft tube from center of turbine

$H_2 = + h + =$ c/c distance between generator and turbine H_3 =

Height above generator

L_t = Length of stator frame

h_j = Height of load bearing bracket

K= 5.5 to 7m depending upon size of machine

Loading Bay

Loading bay, also known as erection bay or service bay, is a space where the heavy vehicles can be loaded and unloaded, the dismantled parts of the machines can be placed and where small assembling of the equipment 's can be done. The loading bay should be sufficient to receive the large parts like rotor and runner. The loading bay floor will be having a width at least equal to the center-to-center distance between the machines.

Control Bay

Control bay is the main room and control other equipment 's like runner, gate valves, generator etc. it may be adjacent to the unit bay i.e., machine halls as it sends instructions to the operation bay from where the operation control is received.

3.5.11.2 Selection of turbine

Turbine selection and plant capacity determination require detailed information on head and possible plant discharge. It has been seen that various parameters can be changed to achieve the best installation. The usual practice is to base the selection on the annual energy output of the plant and the least cost of the energy of scale of hydropower installation. The turbine that we have selected is a vertical axis Pelton turbine based on several criteria. Factors to be considered while selecting a turbine:

Available head and its functions

- Very high head (> 350 m) – Pelton turbine (no other)
- High head (150 – 300 m) – Pelton or Francis turbine (For higher specific speed, Francis
- turbine is more compact and economical than Pelton turbine)
- Medium head (60 – 150 m) – Francis turbine
- Low head (Below 60 m) – Between 30-60 m both Kaplan and Francis turbines can be used. The former is more expensive but yields higher efficiency at part load and overload. Kaplan turbine is generally used under 30 m. Propeller turbines are commonly used for heads up to 15 m. They are adopted only when there is practically no-load variation.

Specific speed

High specific speed is essential where the head is low, and the output is large because otherwise, the rotational speed will be low which means the cost of the turbo generator and powerhouse will be high. On the other hand, there is practically no need of choosing a high value specific speed for high installation because even with a low specific speed high rotational speed can be attained with the medium capacity plant.

Rotational speed

Rotational speed depends upon a specific speed. Also, the rotational speed of an electrical generator with which the turbine is to be directly coupled depends on the frequency and number of pair poles. The value of the specific speed adopted should be such that will give the synchronous speed of the generator.

Efficiency

The turbine selected should be such that it gives highest overall efficiency for various operating condition.

Deposition of the turbine shaft

Experience has shown that the vertical shaft arrangement is better for large sized turbine; therefore, it is almost universally adopted. In case of large sized turbine, horizontal shaft arrangement is almost employed.

Conveyance or Maintenance

Maintenance of the reaction turbine is costlier than the impulse turbine.

Water Quality

The quality of water is more crucial for reactive turbine than reaction turbine.

3.5.11.3 Design codes for analysis of powerhouse

For the analysis and design of the building references have been made to Indian Standard code since National Building Codes of Nepal do not provide sufficient information and refers frequently to the Indian standard codes.

Indian Standard codes used in the analysis and design of this building is described below:

i. IS:875- 1987 (Reaffirmed 2003)- Code of Practice for Design Loads (Other than Earthquake) for Buildings and Structures: This code is divided into five different parts for five different kinds of loadings. The different parts of the code that will be used are:
    a. Part 1: Dead Loads- Unit Weight of Building Materials and Stored Materials
    b. Part 2: Imposed Loads Imposed load is the load assumed to be produced by the intended use or occupancy of a building including the weight of moveable partitions, distributed, concentrated loads, loads due to impact and vibrations and dust loads (Excluding wind, seismic, snow, load due to temperature change, creep, shrinkage, differential settlements etc.).
ii. IS 1893 (Part 1): 2002 Criteria for Earthquake Resistant Design of Structures (General Provision and Building): This code deals with the assessment of seismic loads on various structures and earthquake resistant design of buildings
iii. IS 13920: 1993 (Reaffirmed 2003) Ductile Detailing of Reinforced Concrete Structures Subjected to Seismic Force: This standard covers the requirements for designing and detailing of monolithic reinforced concrete buildings so as to give them adequate toughness and ductility to resist sever earthquake shock without collapse.
iv. IS 456: 2000 (Reaffirmed 2005) Plain and Reinforced: This Indian Standard code of practice deals with the general structural use of plain and reinforced concrete based on Limit State Design Method.
v. SP 16: Design Aids for Reinforced Concrete to IS 456-1978: This handbook explains the use of formulae mentioned in IS 456 and provides several design charts and interaction diagrams for flexure, deflection control criteria, axial compression, and compression with bending and tension with bending for rectangular cross-sections.

4. METHODOLOGY

The framework outlining the systematic steps to be carried out during the execution of this project is as depicted below:

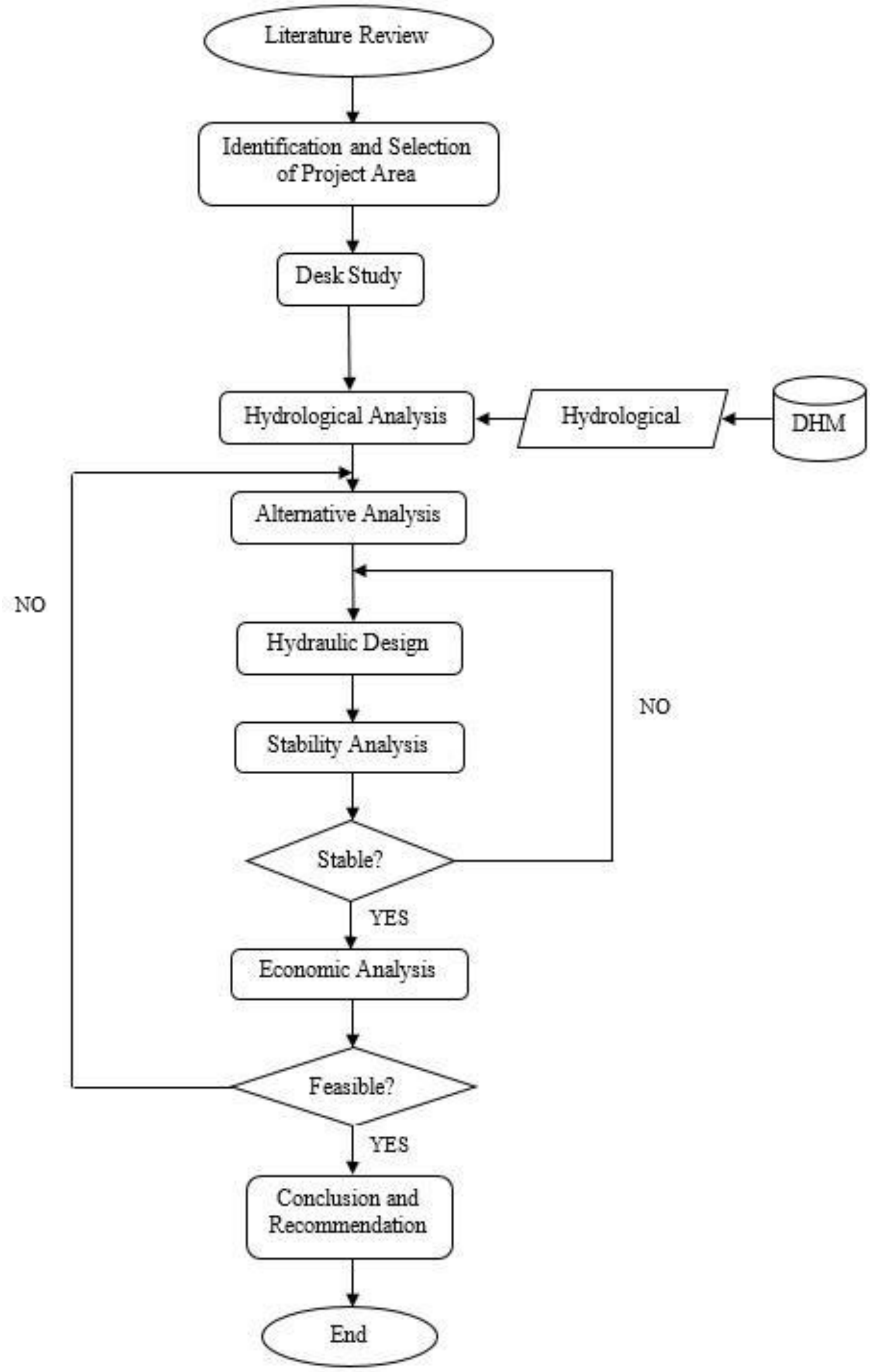

Figure 4: Methodology

4.1 Literature Review and Project Area Identification

The study will begin with an extensive review of relevant literature to build foundational knowledge in hydropower planning, design, and analysis. Textbooks, technical manuals, DoED design guidelines, Indian Standard Codes, and research papers will be consulted. This will help define the scope, design parameters, and applicable standards. The project area—Seti Khola—has been selected based on its hydrological potential and favorable topography, verified through topographic maps and satellite imagery.

4.2 Desk Study

A desk study will be carried out to collect secondary data and develop a preliminary understanding of the site. Data sources will include hydrological records from the Department of Hydrology and Meteorology (DHM), topographic maps, and DEMs. Relevant intern reports and academic literature will also be reviewed. Based on these inputs, a work schedule will be developed, and responsibilities will be allocated among team members. Regular consultations with the project supervisor will guide the refinement of the approach.

4.3 Alternative Analysis

The objective of this phase is to explore various technical alternatives to optimize project layout and performance. Key decisions to be made include the choice between canal, tunnel, or pipe alignments, selection of powerhouse and intake locations, and whether to incorporate a Peaking Run-of-River (PRoR) scheme. Tunnel alignments, surge tank necessity, and penstock routing will be compared based on feasibility, efficiency, and constructability. The most viable alternative will be selected for further analysis and design.

4.4 Hydrological Analysis

Hydrological analysis will involve the processing of discharge data obtained from DHM. Flow Duration Curves (FDCs) will be generated, and flood frequency analysis will be conducted using Gumbel, Log Normal, and Log Pearson Type III methods. Software tools such as HEC-RAS, QGIS, and MS Excel will be used for flood level estimation, rating curve development, and data visualization. Power potential calculations will also be carried out based on design discharge and net head.

4.5 Hydraulic Design

The design of major civil components—including barrage, intake, desander, headrace tunnel, surge tank, penstock, and powerhouse—will be performed based on hydrological inputs and topographical data. Design calculations will be carried out using MS Excel and guided by established hydropower design manuals and IS codes. Component dimensions will be selected to ensure adequate capacity and hydraulic efficiency while addressing site-specific constraints.

4.6 Stability Analysis

Stability analysis will be conducted for selected structures to assess their performance under various loading and hydraulic conditions. This includes analysis of the barrage and undersluice using classical approaches for sliding, overturning, and uplift. ETABS software will be utilized for structural analysis of the powerhouse to evaluate its behavior under dead, live, and seismic loads. Unstable components will be redesigned iteratively to meet the required safety factors.

4.7 Economic Analysis

A preliminary economic assessment will be carried out to determine the project's financial feasibility. Quantities for excavation, concrete, reinforcement, and formwork will be estimated, followed by cost estimation based on unit rates obtained from market research and internship experience. Project revenue will be estimated from energy generation projections. Economic indicators such as the Benefit-Cost Ratio (B/C) and Internal Rate of Return (IRR) will be used to evaluate viability.

4.8 Report Compilation and Presentation

The outcomes of the analysis and design processes will be compiled into a structured proposal report. This report will include key findings from each stage, supported by calculations, drawings, tables, and charts. The final document will serve as the basis for defense presentation and further development of the project in subsequent phases.

5. ALTERNATIVE ANALYSIS

Alternative analysis presents a study of possible and viable options for project development, with the objective of optimizing efficiency, effectiveness, and technical feasibility for the Seti Khola Hydropower Project. This chapter outlines the key configurations assessed, considering topographical, geological, hydrological, and economic factors.

5.1 Available Alternatives

All viable alternatives were analyzed based on merit and demerit to determine the most appropriate configuration. The following components were considered in the optimization study:

- Type of alignment – Tunnel, Canal, or Pipe;
- Location of Headworks and Tunnel Adit;
- Location of Powerhouse and Tailrace;
- Penstock and Surge System Alignment;
- Inclusion or Exclusion of Surge Tank;
- RoR (Run-of-River) vs. PRoR (Peaking Run-of-River) scheme.

5.1.1 Type of Alignment: Tunnel, Canal, or Pipe

Tunnel:
Considering the steep terrain on the right bank of Seti Khola and the availability of competent rock for tunneling, a free-flow headrace tunnel is the most feasible option. The tunnel alignment avoids surface hazards, minimizes land acquisition and resettlement, and provides better hydraulic efficiency for the high discharge (54.73 m^3/s) required. The final alignment results in a 3,200 m tunnel with suitable overburden and geological cover.

Canal:
The canal option is not viable for Seti Khola due to challenging topography with steep slopes, cross drainage issues, and the risk of erosion and instability. Furthermore, canal alignment would require traversing cultivated lands and settlements, increasing the risk of social impact and construction complexity.

Pipe:
While pressure pipe systems offer flexibility and are effective in compact terrains, their higher head losses, complex fabrication, and unsuitability for high discharge volumes make them less favorable for this low-head (68 m gross head) project. Given the cost and technical concerns, pipe alignment is not adopted for the headrace system.

Conclusion:
A **free-flow tunnel** on the **right bank** of Seti Khola was selected as the most reliable and efficient headrace conveyance option.

5.1.2 Surge Tank, Forebay, or Special Turbine Regulation

In hydropower systems with long tunnels and varying loads, surge tanks are essential to manage hydraulic transients. For Seti Khola:

- The selected turbine is Francis type, capable of pressure control via relief valves and slow closure mechanisms.
- Despite the option of omitting a surge tank using advanced turbine regulation (e.g., air cushion chambers, bypass valves), this is feasible only with full manufacturer support and proven operational performance.
- Due to the risk of water hammer and system instability, a **conventional open-type surge tank** at the tunnel outlet has been adopted to ensure safety and performance reliability.

5.1.3 Powerhouse and Tailrace Location

Two alternative powerhouse sites were considered:

- **Option I: Lower Powerhouse (Selected Option)**: Located on the right bank downstream, this site allows a shorter tailrace (~450 m) and utilizes the full available gross head (68 m). While the penstock is longer (620 m), excavation volume is moderate, and access to the grid and switchyard is favorable.

- **Option II: Upper Powerhouse**: Located further upstream, this site offers a slightly shorter penstock (570 m) but requires a longer tailrace (~700 m) and results in reduced head (60.5 m gross). This leads to lower generation capacity and higher excavation requirements.

Conclusion:
Option I with a **lower powerhouse** and **higher net head (63.9 m)** was selected in place of Option II

Table 2: Summary of Alignment Options

Particulars	**Option I (Selected)**	**Option II**
HW and Channel	Upper HW with direct tunnel intake	Lower HW with 200 m connection channel
Headrace Tunnel	3,050 m	3,150 m
Intake Location	Right Bank of Seti Khola	Right Bank
Surge Tank	Open Surface	Open Surface
Penstock Length	610 m	570 m
Powerhouse	Lower Right Bank	Upper Right Bank
Tailrace	~450 m	~700 m
Gross Head	68 m	60.5 m
Net Head	63.9 m	56.2 m
Installed Capacity	23.54 MW	20.7 MW
Land/Social Impact	Minimal	Higher Forest & Land Impact
R&M Cost	Low	Low

Conclusion:
Considering the importance of head retention in this marginal high-discharge, low-head project, **Option I** was finalized for further design and development.

5.2 RoR vs. PRoR Options

The project was assessed for potential as a **Peaking Run-of-River (PRoR)** plant with storage for 4-hour peaking. However:

- According to current **NEA (Nepal Electricity Authority)** policy, PRoR schemes are not acceptable for projects like Seti Khola.
- Therefore, a **pure Run-of-River (RoR)** configuration was selected.

In practice, Nepal's policy has been ROR-dominant. Industry experts note that there is "no balance between Run-of-River or Peaking Run-of-River projects and storage projects", and many licensed PROR projects are effectively "barred" by the lack of clear policy.
Under this regulatory uncertainty, NEA has signaled that PROR options will not be approved unless they strictly meet all tariff and performance criteria. Thus, for Seti Khola – and similar medium-sized rivers – NEA has only endorsed the pure ROR configuration, consistent with its guidelines.

5.3 Finalized Project Configuration

Based on technical assessments, environmental considerations, and economic viability, the final configuration for Seti Khola Hydropower Project is summarized below:

Table 3: Project Component Finalization

SN	Component	Description	Remarks
1	Diversion Structure	Weir with undersluice	Gated Arrangement
2	Intake	Side intake on right bank	Suitable
3	Headpond & Desander	Flatbed desander with intermittent flushing	On right bank
4	Headrace Tunnel	Free-flow tunnel (3,050m)	Right bank
5	Surge Tank	Surface surge tank at tunnel outlet	
6	Penstock	Buried steel penstock (610 m)	Shortest feasible
7	Powerhouse	Surface powerhouse on right bank	Near flat terrain
8	Tailrace	Short tailrace pipe (~450 m)	Discharges to Seti Khola

This finalized configuration provides optimal use of available resources while minimizing costs, social impact, and environmental disturbance. The alignment now forms the basis for detailed design, cost estimation, and environmental assessment for the Seti Khola Hydropower Project.

6. HYDROLOGICAL STUDIES

6.1 Hydrological Analysis

Hydrological analysis is a fundamental component of hydrological studies, providing crucial insights into the behavior of water within a specific geographical area over time. This analytical process involves the examination and interpretation of various hydrological data and parameters to understand the distribution, movement, and availability of water resources.

6.1.1 Catchment Characteristics

Seti Khola is one of the major tributaries of Trishuli River in Gandaki basin. It is a perennial river originated from the snow fields and glaciers around the twin peaks of Annapurna III and Machapuchre in the south facing slopes of the main Himalayas. The basic characteristics of Seti Khola was assessed using the QGIS tools based on the digital elevation model (DEM) data derived from contour map published by Survey Department of Nepal Government as well as from Google earth. The catchment area of Seti Khola Hydroelectric project at intake is 866.12 km^2.

The total catchment area can further be divided as follows:

Elevation	Area
Above 5,000 m	48.485 km^2
Between 3,000-5000 m	181.768 km^2
Between 3,000-1,000 m	476.57 km^2
Below 1,000 m	159.789 km^2
Total Catchment area	866.12 km^2

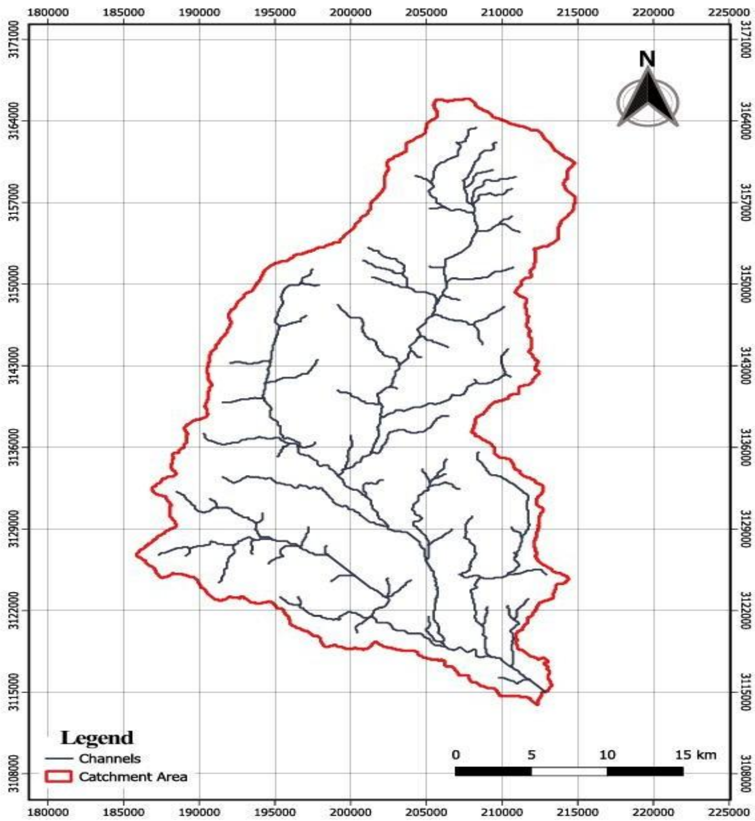

Figure 5: Catchment Area of Intake

6.1.2 Catchment Area Ratio Method

Catchment Area Ratio (CAR) method is a fundamental tool used to estimate various hydrological parameters for a specific catchment or watershed area. This method is particularly useful for redicting streamflow characteristics, such as peak discharge, hydrograph shape, and runoff volume, which are essential for water resources management, flood forecasting, and infrastructure design.

The Catchment Area Ratio method operates on the principle that the hydrological response of a catchment is influenced by its physical characteristics, primarily its size and shape. By comparing the characteristics of the catchment under study to those of a reference catchment with known hydrological parameters, we can make reasonable estimations about the hydrological behavior of the target catchment.

Here's how the Catchment Area Ratio method works:

Selecting Reference Catchments: The first step involves identifying one or more reference catchments with well-documented hydrological data, such as streamflow records, rainfall data, soil properties, land use information, and topographic characteristics. These reference catchments should ideally be located in similar geographic regions with comparable climatic conditions to the catchment under study. In this project, two hydrological station from DHM were used i.e Seti Gandaki River at Damauli and Seti at Phulbari.

Calculating Catchment Area Ratio: The catchment area ratio (CAR) is computed by dividing the area of the target catchment by the area of the reference catchment. This ratio serves as the basis for scaling hydrological parameters from the reference catchment to the target catchment.

Estimating Hydrological Parameters: Once the catchment area ratio is determined, various hydrological parameters, such as peak discharge and hydrograph shape, can be estimated for the target catchment using empirical relationships derived from the reference catchment data. These relationships may be statistical model or simple scaling factors based on the catchment area ratio.

Validation and Adjustment: It's essential to validate the estimated hydrological parameters by comparing them with observed data, if available, or by applying independent methods of estimation. Adjustments may be necessary to account for differences in land use, soil properties, or other factors that affect hydrological processes but are not captured solely by catchment area ratio.

Application and Interpretation: Once validated, the estimated hydrological parameters can be used for various purposes, such as flood risk assessment, water resources planning, and environmental impact analysis. Interpretation of the results should consider the limitations and uncertainties associated with the Catchment Area Ratio method and the assumptions underlying the estimation process.

It's worth noting that while the Catchment Area Ratio method provides a simple and practical approach to estimate hydrological parameters, its accuracy may vary depending on the similarity between the target catchment and the reference catchments, as well as the availability and quality of data. Therefore, in case of the project, the target and reference catchments are of similar nature which increases the accuracy of the data from the project processes.

6.2 Hydrograph

A hydrograph is a graph that depicts the flow rate or discharge of water in a river or stream over time. It depicts the variations in water level or flow rate caused by rainfall or other hydrological phenomena. Hydrographs are extensively used in hydrology to investigate and evaluate river system features such as flood timing, duration, and magnitude. A range of hydrological metrics, such as peak flow rate, time to peak, and total runoff volume, may be calculated using hydrographs. They are also used to create flood forecasting models and evaluate the possible influence of land use changes or other environmental factors on river systems.

The below hydrograph shows the comparative discharge in Seti Khola at different month of the year derived from Gandaki-Damauli station as well as Phoolbari station. Here we can observe that the discharge derived from the Phoolbari is greater than that of Gandaki-Damauli station. But in this project we adopted the mean discharge value obtained from the Gandaki-Damauli station because of the reason that the station Gandaki-Damauli was nearer to the proposed intake site rather than the station Phoolbari signifying catchment characteristics similarity and also the primary data obtained from the consulting firm was quite close to the discharge derived from the Gandaki-Damauli station.

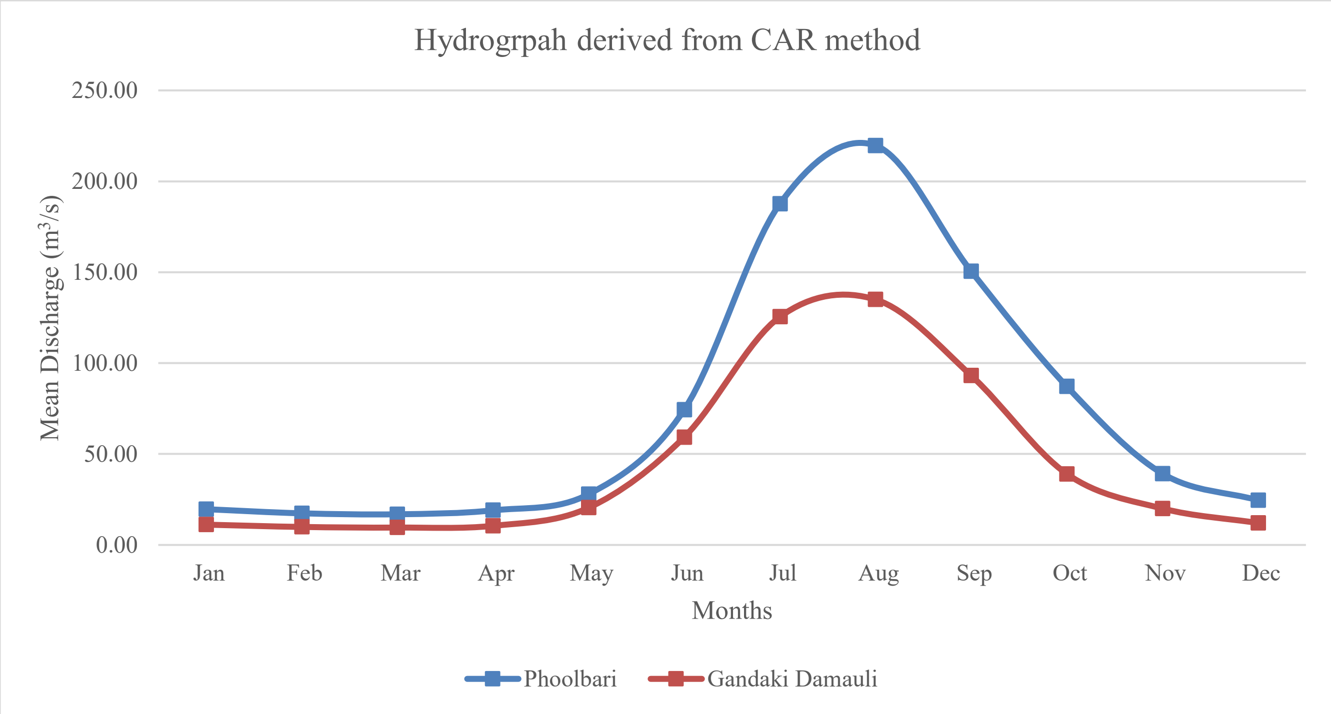

Figure 6: Hydrograph derived from both station Phoolbari and Gandaki Damauli

We can observe that the discharge in the river is low during the start of the year. The low-level discharge of the river depicts that the start of the year is a dry period where the river is close to its base flow condition up until the month of May. As May hits, the river starts increasing discharge as precipitation accumulates. This accumulation of precipitation and increase in discharge keeps on increasing and reaches its peak at the month of August. The highest point on the hydrograph represents the peak discharge, which corresponds to the maximum flow rate during a particular rainfall event.

Afterwards, we can observe that there is a gradual decrease in precipitation which affects the discharge as runoff diminishes and the river levels recede. This carries on into the next few months until the cycle is repeated for a different year.

6.3 Flow Duration Curve

The Flow Duration Curve (FDC) is one of the most fundamental pieces of information that feeds into the design of a hydropower project, so for anyone that wants to understand the how's and why's of hydropower design, understanding the flow duration curve is important. The flow duration curve is a plot that shows the percentage of time that flow in a stream is likely to equal or exceed some specified value of interest. The basic time unit used in preparing a flow-duration curve will greatly affect its appearance. For most studies, mean daily discharges are used. The flow-duration curve is a cumulative frequency curve that shows the percent of time during which specified discharges were equaled or exceeded in a given period.

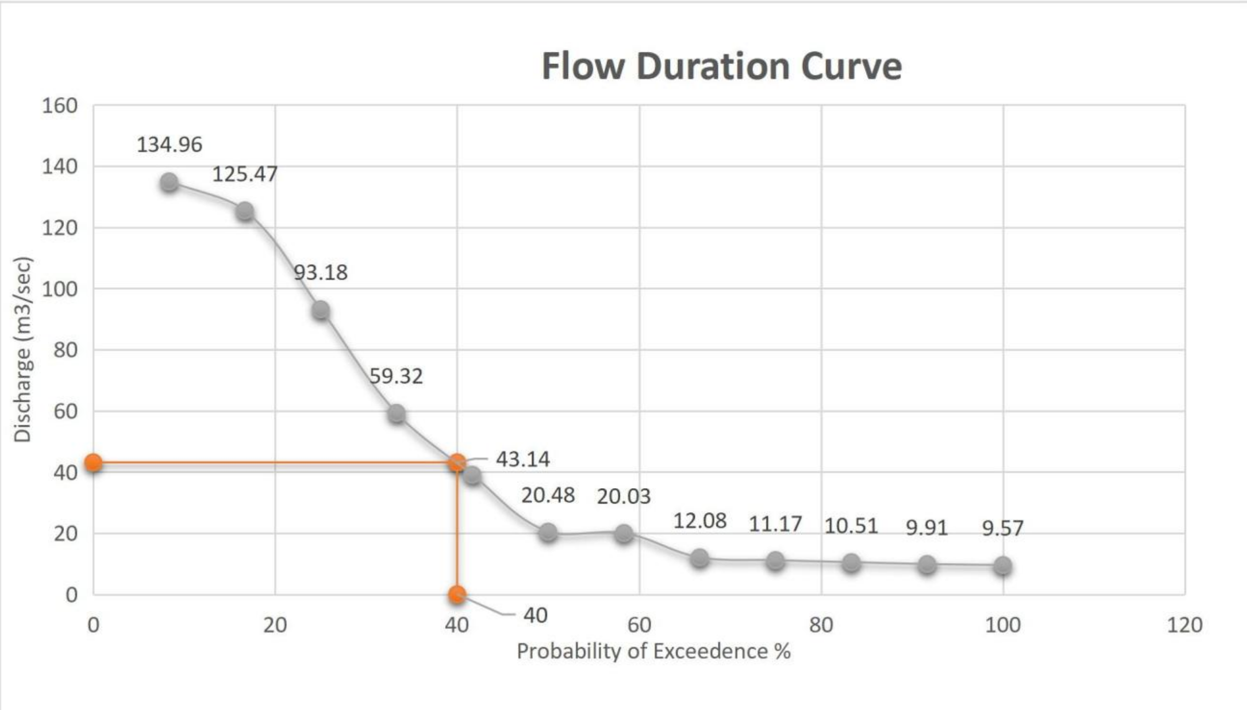

Figure 7: Flow Duration Curve

The above graph provides us the information regarding the amount of discharge at the Seti Khola with regards to percentage of time derived from Gandaki station. It shows what amount of discharge value is exceeded for what percentage of time in the river. Utilizing the above graph, we were able to certain the design discharge for the project, the amount of discharge exceeded 40% of the time, which was found to be 43.136 m^3/s.

6.4 Rating Curve

A rating curve is a graphical depiction of the relationship between a rivers or stream's stage (water level) and discharge (flow rate) at a specific place. It is commonly used by hydrologists, water resource managers, and engineers to monitor and manage water resources to estimate the discharge of a river or stream depending on its stage or water level. Typically, the rating curve is constructed by taking a series of measurements of both stage and discharge across a variety of flow conditions and then graphing the results on a graph. The generated curve depicts the link between stage and discharge and may be used to predict discharge at any given stage.

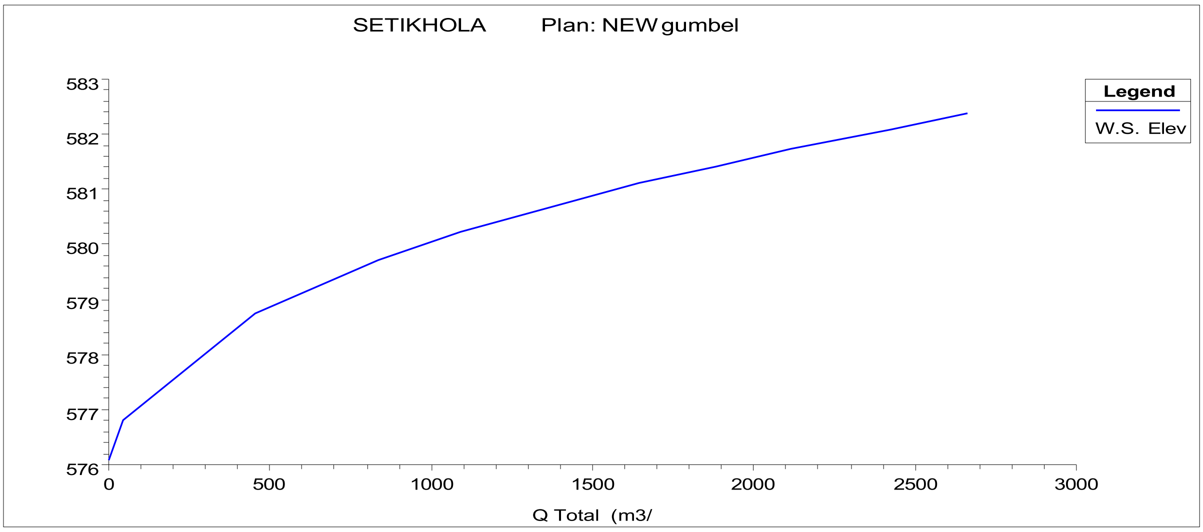

Figure 8: Rating Curve

The rating curve was generated from *HEC-RAS version6.6* which was calculated by simulating steady flow analysis in one dimension geometry defined by detail survey data provided by the consultancy firm where river longitudinal slope and cross section was derived from. The boundary condition was defined and the manning roughness coefficient was taken as 0.03 and 0.04 in the right and left over bank (Bohora B.,2024) & (Basnet K.,2024).

Table 4: Rating Curve

	Discharge (m3/s)	**Stage (m)**
	x	**y**
Bed Level	**0**	**572.89**
Q40	43.14	576.82
(Return period yr) 2	451.52	578.74
(Return period yr) 5	834.46	579.71
(Return period yr) 10	1088.01	580.22
(Return period yr) 50	1646.01	581.11
(Return period yr) **100**	1881.91	581.43
(Return period yr) 200	2116.96	581.73
(Return period yr) 500	2427.05	582.09
(Return period yr) 1000	2661.41	582.38

Rating curves are essential for estimating streamflow/discharge based on the stage (water level) measurements of the river. Rating curves are majorly utilized for flow estimation, flood forecasting and hydraulic design. Due to our project requiring hydraulic design, we utilized the above rating curve to ascertain the design considerations and accommodate the flow rates avoiding flooding or structural damage in our designed components. We used the above rating curve to find out the stage at our headworks cross-section for the Flood of 100 years which was then incorporated into our design of the structures.

6.5 Flood Analysis

Flood analysis refers to the examination of flood hazards, vulnerabilities, and potential impacts within a given area. Within this project, we have carried out flood analysis to assess the likelihood of extreme flooding events, estimating flood magnitudes. Flood analysis was carried out using the following methods:

6.5.1 Gumbel Method

Results of 100 years flood discharge from this method was **2382.1721(m3/s)**

Table 5: Gumbel's extreme-value distribution (DAMAULI STATION)

SN	Year	Q(max)	Ranked Q	Rank (m)	P=m/(n+1)	Return Period	yT	kT	QT
1	2000	423.90	1516.91	1	0.0588	17.00	2.80	2.22	1274.74
2	2001	1516.91	1149.32	2	0.1176	8.50	2.08	1.51	1029.82
3	2002	502.53	615.61	3	0.1765	5.67	1.64	1.09	881.48
4	2003	1149.32	502.53	4	0.2353	4.25	1.32	0.78	772.24
5	2004	299.19	479.27	5	0.2941	3.40	1.05	0.52	684.02
6	2005	383.42	458.33	6	0.3529	2.83	0.83	0.31	608.68
7	2006	188.92	433.67	7	0.4118	2.43	0.63	0.11	541.79
8	2007	479.27	423.90	8	0.4706	2.13	0.45	-0.06	480.60
9	2008	369.92	383.42	9	0.5294	1.89	0.28	-0.23	423.19
10	2009	228.93	369.92	10	0.5882	1.70	0.12	-0.38	368.09
11	2010	458.33	368.99	11	0.6471	1.55	-0.04	-0.54	313.96
12	2011	615.61	366.20	12	0.7059	1.42	-0.20	-0.70	259.46
13	2012	245.68	299.19	13	0.7647	1.31	-0.37	-0.86	202.87
14	2013	433.67	245.68	14	0.8235	1.21	-0.55	-1.03	141.60
15	2014	368.99	228.93	15	0.8824	1.13	-0.76	-1.24	70.63

Table 6: Discharge Table

SN	Return Period	Q (observed)	Q (Predicted)
1	1.13	228.93	70.63
2	1.21	245.68	141.60
3	1.31	299.19	202.87
4	1.42	366.20	259.46
5	1.55	368.99	313.96
6	1.70	369.92	368.09
7	1.89	383.42	423.19
8	2.13	423.90	480.60
9	2.43	433.67	541.79
10	2.83	458.33	608.68
11	3.40	479.27	684.02
12	4.25	502.53	772.24
13	5.67	615.61	881.48
14	8.50	1149.32	1029.82
15	17.00	1516.91	1274.74
16	50		**1646.02**
17	100		**1881.92**
18	200		**2116.96**

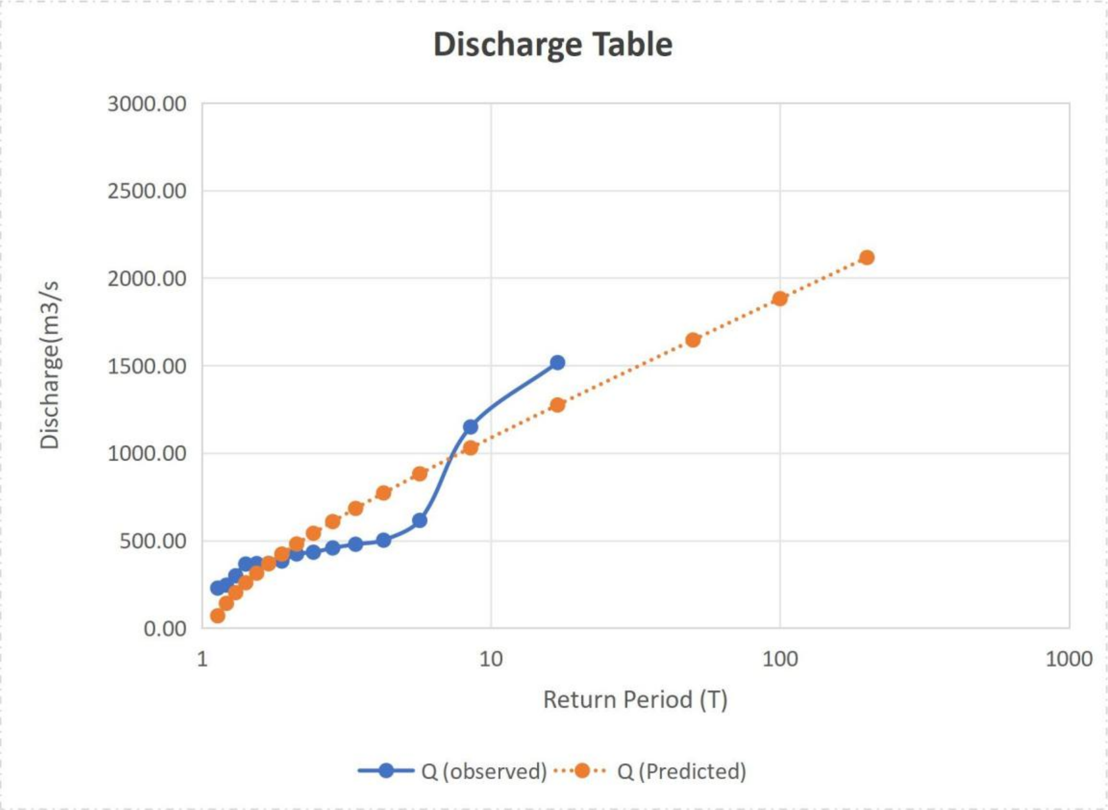

Figure 9:Discharge Versus Time graph

Table 7 : Flood Frequency by Gumbell Method

T	YT	K	XT (m3/s)
2	0.37	-0.14	451.52
5	1.50	0.95	834.46
10	2.25	1.68	1088.01
`50	3.90	3.28	1646.02
100	**4.60**	**3.96**	**1881.92**
200	5.30	4.63	2116.96
500	6.21	5.52	2427.05
1000	6.91	6.20	2661.41

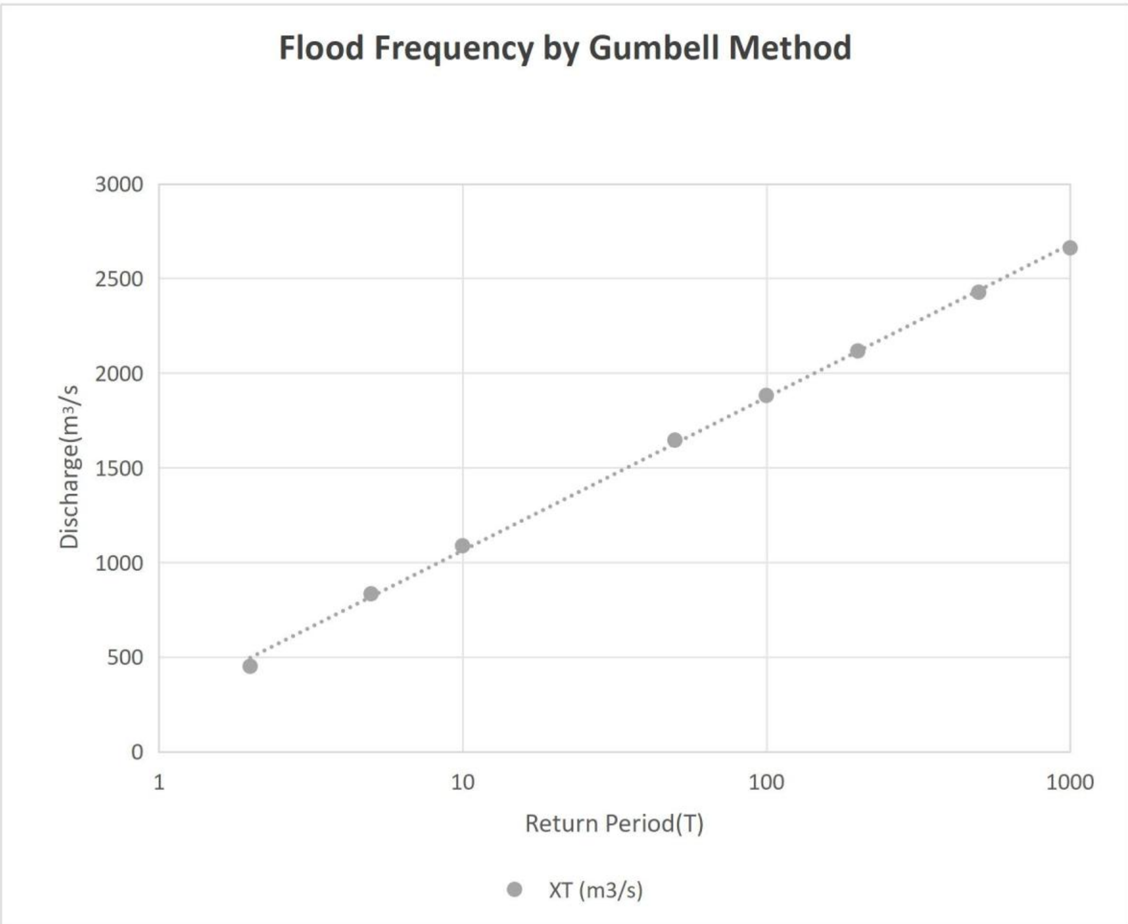

Figure 10: Graph of Flood Frequency by Gumbell Method

6.5.2 Log Pearson Type-III Method

Table 8 : Log Pearson Type-III Method

Log-Pearson Type III Distribution						
SN	**Year**	**Q**	**Ranked (Q)**	**Rank (m)**	**P=m/(n+1)**	**Log(Q)**
1	2000	423.90	1516.91	1	0.06	3.18
2	2001	1516.91	1149.32	2	0.12	3.06
3	2002	502.53	615.61	3	0.18	2.79
4	2003	1149.32	502.53	4	0.24	2.70
5	2004	299.19	479.27	5	0.29	2.68
6	2005	383.42	458.33	6	0.35	2.66
7	2006	188.92	433.67	7	0.41	2.64
8	2007	479.27	423.90	8	0.47	2.63
9	2008	369.92	383.42	9	0.53	2.58
10	2009	228.93	369.92	10	0.59	2.57
11	2010	458.33	368.99	11	0.65	2.57
12	2011	615.61	366.20	12	0.71	2.56
13	2012	245.68	299.19	13	0.76	2.48
14	2013	433.67	245.68	14	0.82	2.39
15	2014	368.99	228.93	15	0.88	2.36
16	2015	366.20	188.92	16	0.94	2.28
					avg	**2.63**

Table 9: Flood Frequency by Log Pearson(Type III)

T (Return Period)	k	yT	XT (m3/s)
2	-0.01	2.63	427.61
5	0.92	2.85	701.69
10	1.35	2.95	884.03
50	2.12	3.13	1342.68
100	**2.40**	**3.19**	**1560.61**
200	2.65	3.25	1785.88
500	2.91	3.31	2053.57
1000	3.10	3.36	2273.02

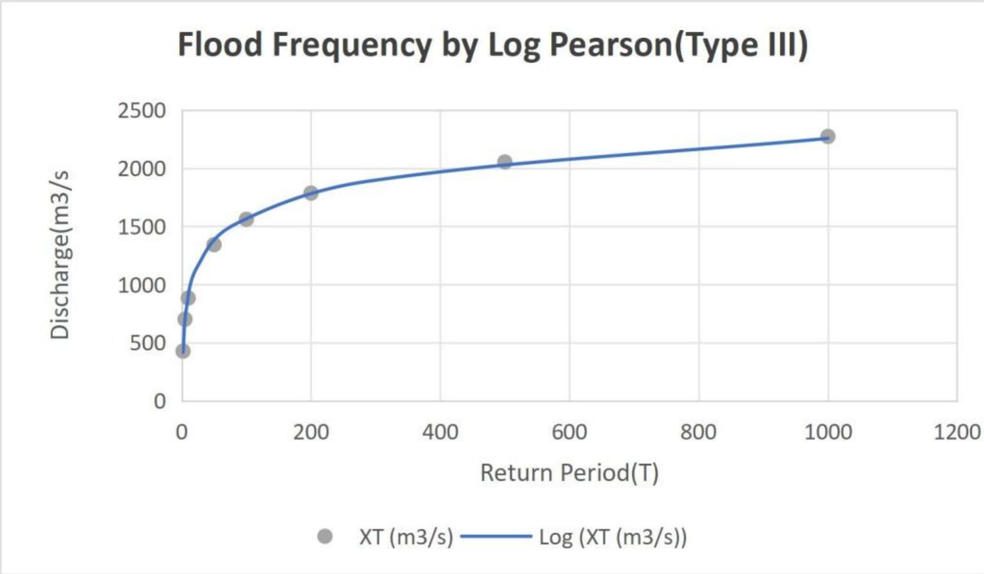

Figure 11: Graph of Flood Frequency by Log Pearson(Type III)

6.5.3 Log Normal Method

Table 10: Flood Frequency By Log Normal Method

R period	(1-1/T)	Z value	Y value	X (m^3/s)
2	0.50	0.00	2.63	429.22
5	0.80	0.84	2.83	674.57
10	0.90	1.28	2.93	854.40
50	0.98	2.05	3.11	1293.65
100	**0.99**	**2.33**	**3.18**	**1497.66**
200	1.00	2.58	3.23	1712.44

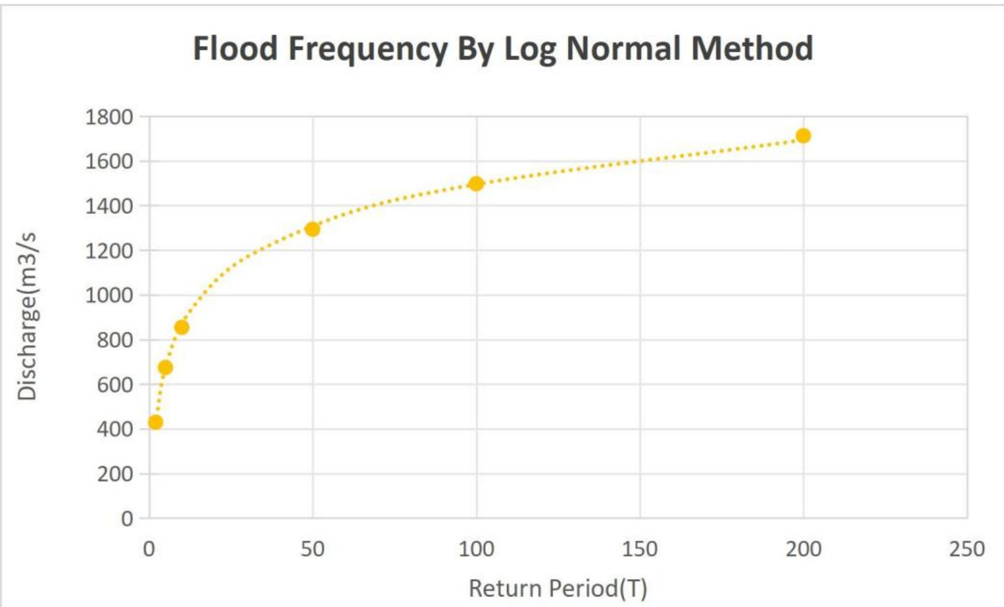

Figure 12:Graph for Flood Frequency By Log Normal Method

Among these methods, the value obtained was as follows:

Table 11: Results of 100 years flood discharge from different methods

Methods	Discharge Q_{100} (m^3/s)
Gumbel's Method	**1882**
Log Pearson III Method	**1561**
Log Normal Method	**1498**

The value from Gumbel's Method was adopted as the 100 year flood discharge and incorporated into our hydraulic design.

Gumbel Method Was adopted for 100-Year Flood Estimation in Seti Khola HEP because it helps in:

1. Designed for Extreme Events: Gumbel’s method specializes in modeling annual maximum flood values, making it ideal for estimating the rare but critical 100-year flood required for the design of flood-sensitive structures like spillways and diversion weirs in Seti Khola.

2. Conservative for Safety-Critical Infrastructure: Among the methods analyzed, Gumbel produced the highest discharge value (1882 m^3/s). This conservative estimate provides an added margin of safety—crucial for a Himalayan river basin prone to intense monsoon rainfall and potential GLOF (Glacial Lake Outburst Flood) risks.

3. Reliable for Short-to-Medium Data Records: Seti Khola catchment, like many in Nepal, has limited long-term hydrological data. Gumbel performs reliably with such datasets, avoiding overfitting while still offering a sound statistical basis for flood prediction.

4. Widely Accepted in Nepalese Hydropower Practice: Gumbel’s method has been successfully applied in several other hydropower projects across Nepal, making it a proven and trusted approach in the national hydrological and engineering community.

5. Alignment with Dam Safety Guidelines: The method’s focus on extreme values aligns with national and international design standards (such as those of NEA, World Bank, and ICOLD) that prioritize flood safety for energy infrastructure.

6. Reflects Seti Khola’s Hydrological Risk Profile: The Seti Khola basin experiences high rainfall intensity and rapid runoff response, especially during peak monsoon. Gumbel’s tail-heavy distribution better captures these extreme outliers compared to Log Normal or Log Pearson methods.

7. Practical for Design Decision-Making: The final adoption of Gumbel’s Q100 value ensures that hydraulic components—like the flood handling capacity of the spillway—are adequately sized, reducing the risk of structural failure during extreme events.

6.6 Analysis of Rainfall

6.6.1 Arithmetic Mean Method

The arithmetic mean simply averages the rainfall from all stations **without considering their area influence**.

$$\text{Arithmetic Mean Rainfall}=\frac{\sum \text{Rainfall at all stations}}{\text{Number of stations}}$$

$$=\frac{3768.80+5402.90+2868.50+2844.20+3971.70+4363.40+2418.90+3719.90+451.00}{9}$$

=3,312.14mm

Limitation:

This method treats all stations equally, ignoring spatial variability and area coverage.

6.6.2 Thiessen Polygon Method (Area-Weighted Average)

The Thiessen method assigns weights based on the area represented by each station. Above table already provides the **weighted rainfall** (Rainfall × Area). The average rainfall is calculated as:

$$\text{Thiessen Mean Rainfall}=\frac{\sum(\text{Weighted Rainfall}}{\sum(\text{Area})}$$

Table 12: Thiessen Polygon Method

Station No.	Station Name	Rainfall (mm)	Area (km²)	Weighted Rain (mm·km²)
824	Siklesh	3,768.8	758.38	2,858,196.79
814	Lumle	5,402.9	371.62	2,007,805.35
805	Syangja	2,868.5	428.89	1,230,260.31
619	Ghorapani	2,844.2	314.52	894,546.44
804	Pokhara Airport	3,971.7	201.90	801,899.18
818	Lamachaur	4,363.4	139.27	607,700.22
613	Karkineta	2,418.9	161.22	389,964.00
811	Malepatan	3,719.9	71.03	264,239.75
820	Manang Bhot	451.0	495.37	223,413.19

Totals:

- **Sum of Areas:** 2,942.2 km²
- **Sum of Weighted Rain:** 9,278,025.23 mm·km²
- **Mean Rainfall:** 3,153.432 mm (9,278,025.23 ÷ 2,942.2)

Note: Coordinates removed for simplicity. All values rounded to 2 decimal places.

From our table obtained from QGIS :

- **Sum of Areas:** 2,942.2 km²
- **Sum of Weighted Rain:** 9,278,025.23 mm·km²
- **Mean Rainfall:** 3,153.432 mm (9,278,025.23 ÷ 2,942.2)

$$\text{Thiessen Mean Rainfall} = \frac{9{,}278{,}025.23}{2{,}942.2} = 3{,}153.44 \text{ mm}$$

Interpretation:
This method accounts for the spatial distribution of rainfall by giving more weight to stations covering larger areas

6.6.3 Isohyetal Method

The **Isohyetal method** involves drawing **contours of equal rainfall (isohyets)** and calculating the area-weighted average between them. Since we don't have a map, we can approximate it using the following steps:

Steps:

1. **Sort stations by rainfall** (ascending/descending).
2. **Draw isohyets** (contours) between stations, interpolating rainfall values.
3. **Calculate the area between isohyets** (requires a map or spatial data).
4. **Compute weighted average rainfall**:

$$\text{Isohyetal Mean} = \frac{\sum(\text{Average Rainfall between isohyets} \times \text{Area})}{\sum(\text{Area})}$$

$$\text{Isohyetal Mean} = \frac{\sum(\text{Weighted Rainfall}}{\sum(\text{Area})} = \frac{\mathbf{326.}}{\mathbf{21}} = \mathbf{3768.43mm}$$

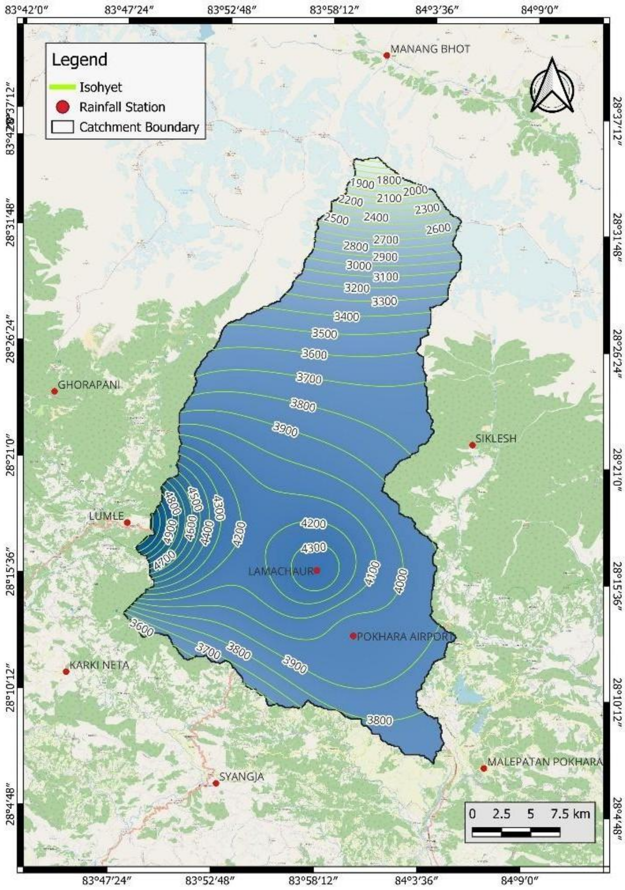

Figure 13: Isohyetal Method from QGIS

Table 13 : Comparison of All Methods of Rainfall

Method	Avg. Rainfall (mm)
Arithmetic Mean	3,312.14
Thiessen Polygon	3,153.44
Isohyetal	3768.43

Observations:

1. **Arithmetic Mean (3,312 mm)** – Simplest, but ignores area influence.
2. **Thiessen (3,154 mm)** – Better, but assumes uniform rainfall over polygons.
3. **Isohyetal (3,769 mm)** – Most realistic, as it accounts for rainfall gradients

We choosed Isohyetal Method for Seti Khola

Reason for choosing Isohyetal Method for Seti Khola

1. Mountainous Terrain Impact
    - o The Seti River basin has steep elevation changes (Manang Bhot: 451 mm vs. Lumle: 5,402 mm).
    - o Isohyets can follow elevation contours, improving accuracy.
2. Captures Orographic Effects
    - o The windward side (Lumle, Lamachaur) gets heavy rainfall, while Manang (rain-shadow) gets very little.
    - o The Isohyetal method interpolates gradients better than Thiessen.
3. Better for Hydrological Modeling
    - o Rainfall-runoff models (e.g., HEC-HMS, SWAT) need spatially distributed rainfall.
    - o Isohyets provide continuous rainfall distribution, unlike Thiessen's discrete polygons.

6.7 Himalayan Sediment Yield Technique

The Himalayan Sediment Yield Technique involves estimating sediment yields for various physiographic zones and multiplying the sediment yield by corresponding catchment area with the following equation:

Sediment Yield (t/km^2/year) = Zone Specific Yield x Catchment Area

The technique involves using an estimate of sediment yield (tones/km^2/yr) from each physiographic region and multiplying the sediment yield by the appropriate area. In this technique, the Nepal is divided into five different physiographic zones as follows

Table 14:Specific Sediment Yield of different Physiographic zones

Physiographic Zone	Elevation(m)	Specific Sediment Yield (t/km^2/year)
High Himalaya	Above 5000	500
High Mountain	3000-5000	2500
Middle Mountain	1000-3000	5000
Siwaliks	500-1000	7500

The total catchment area can further be divided as follows:

Total Catchment area	= 866.612 km^2
Catchment area above 5,000 m elevation	= 48.485 km^2
Catchment area between 3,000-5000 m elevation	= 181.768 km^2
Catchment area below 3,000 upto 1000m elevation	= 476.57 km^2
Catchment area below 1,000 m elevation	= 159.789 km^2

Therefore, the sediment yield of Seti Khola is calculated 4059930 t/year.

7. DESIGN OF HYDRAULIC COMPONENTS

The hydraulic design of the main structures for the hydropower project has been developed using parameters from site data and standard design references, including IS codes, USBR guidelines, and literature relevant to Himalayan River hydrology. The key components considered in this chapter are the barrage, undersluice, side intake, trashrack, and settling basin (desander). The detailed computations and drawings are presented in the annex section.

Before going to the design details of the component the required power and energy generation should be known. Below is the table where Seti Khola HEP power energy generation calculation is given;

Table 15: Power Energy calculation

Months of English Calender	**Seti River Discharge (m3/ sec)**	**Environmental Flow (m3/ sec)**	**Available Qmax River (m3/ sec)**	**Max Plant Discharge (Q)$m^{3/s}$**	**Head loss**	**Net Head (m)**	**Days of Month**	**Power Output (kW)**	**Gross Energy (kWh)**	**Outage & Losses (%)**	**Deemed Generation & Contract Energy**	**Winter/ Summer Energy**	**Remarks**
Jan	18.93	1.62	17.31	17.31	1.07	66.93	31	10249	7625355	4%	7320341	48151669	**33.4 %**
Feb	16.79	1.62	15.17	15.17	0.84	67.16	28	9013	6056484	4%	5814225		
Mar	16.22	1.62	14.60	14.60	0.67	67.33	31	8695	6469426	4%	6210649		
Apr	17.81	1.62	16.19	16.19	0.84	67.16	30	9619	6925800	4%	6648768		
May	34.71	1.62	33.09	33.09	1.89	66.11	31	19355	14399873	4%	13823878		
Jun	100.55	1.62	98.93	43.14	**4.96**	**63.04**	30	23540	16948800	4%	16270848	95891861	**66.6 %**
Jul	212.68	1.62	211.06	43.14	**4.1**	**63.90**	31	23540	17513760	4%	16813210		
Aug	228.77	1.62	227.15	43.14	**4.1**	**63.90**	31	23540	17513760	4%	16813210		
Sep	157.95	1.62	156.33	43.14	**4.1**	**63.90**	30	23540	16948800	4%	16270848		
Oct	66.26	1.62	64.64	43.14	**4.1**	**63.90**	31	23540	17513760	4%	16813210		
Nov	33.96	1.62	32.34	32.34	2.72	65.28	30	18678	13448475	4%	12910536		
Dec	21.49	1.62	19.87	19.87	1.62	66.38	31	11668	8681050	4%	8333808		
Total Energy							**365**		**150045343**		144043530	**144043530**	**100 %**
								Pant Factor			**74.7%**		

7.1 Hydraulic Design of Weir and Undersluice

Normal water level at the headworks is calculated from the rating curve and its value is 573.61 masl for design discharge of 43.136 m3 /s. The crest level of weir is maintained at 575.81.The entire headworks area are designed for flood of 100 year return period whose value is 1882 m3 /s that corresponds to the elevation of 581.42 masl. The total length of weir portion is 134 m and its width is 55 m. Cutoff walls has been designed using the Khosla's Theory and they have been provided at the both the upstream and downstream of the diversion structure with the depth of 4m. The weir is designed for overflowing the 100% of flood discharge through its overflow section of 55 m length. The undersluice has been designed for the passage of bed load downstream of the diversion structure and to prevent the bed load entering the intake. It doesn't only flush the bed material deposited in the upstream of diversion structure but also creates pocket which helps to maintain the uniform flow towards the intake. The Sill level of underluice is kept at 572.89 masl and the dimensions of undersluice are 155 m long and 9 m wide. Stilling basin has also been designed to dissipate the energy of discharge passing the overflow section before the discharge is returned to the downstream river channel. The floor level of stilling basin has been sett to 568m. The stilling basin has been designed for both the weir and undersluice having the length of 35m and width of 66m. Flood walls are designed for design flood of RL. 581.42 masl and its height is maintained at 582 masl which will be enough to withstand the flow of extreme flood and protect the headworks components of the project.

7.2 Side Intake and Trashrack Design

The intake is designed to accommodate a discharge of 56.0768 m^3/s, which is 130% of the design discharge (43.136 m^3/s), accounting for additional flow required for flushing and system losses. The Normal Water Level (NWL) at the intake location is 576.817 masl, while the canal water level is 578.7 masl. The crest of the weir is set at 578.82 masl, and the riverbed level is located at 576.082 masl. The final width of the intake adopted is 32.0 m with an effective flow area of 64 m^2. The flow velocity at the intake is maintained at 0.921 m/s, which is under the recommended upper limit of 1.0 m/s to prevent erosion and ensure smooth conveyance.

The intake design ensures sufficient capacity with a calculated flow of 58.921 m^3/s using the standard orifice flow formula, which is greater than the required design discharge. Losses due to the intake are minimal, with head losses estimated at approximately 0.0035 m based on the velocity head approach. The coefficient of discharge is assumed to be 0.6, which is typical for side intake structures.

The trashrack is designed to screen out debris and ensure operational safety. It spans a total submerged width of 37.5 m and has a submerged area of 78.86 m^2. The effective open area is 83.3% of the total, resulting in a net area of 65.716 m^2. The approach velocity through the trashrack is limited to 0.853 m/s. The bars are 20 mm thick with a clear spacing of 100 mm and are inclined at an angle of 72°. Using a rounded bar shape coefficient (kr = 1.67), the head loss through the trashrack is found to be minimal at 0.00689 m, ensuring efficient screening with negligible energy loss.Piers at the intake are designed with a width of 1.0 m for end grooves and 1.5 m for center grooves, ensuring structural integrity and proper flow distribution across the bays.

7.3 Settling Basin (Desander) Design

The settling basin is designed to effectively remove particles of up to 0.15 mm diameter, which are common in the sediment load of Himalayan rivers. The total design discharge through the settling basin is 49.6064 m^3/s, including a 15% flushing allowance over the main design discharge of 43.136 m^3/s. The basin system comprises two parallel bays, each carrying 24.8032 m^3/s.

The settling basin is sized using iterative calculations based on turbulence and settling velocity principles. Each bay has a length of 144 m and a width of 18 m, resulting in a total area of 2592 m^2. The flow depth is maintained at 10 m, ensuring adequate sedimentation volume and hydraulic stability. The flow velocity is maintained at 0.2 m/s, and the critical velocity is calculated to be 0.170 m/s. A conservative particle fall velocity of 0.0175 m/s is adopted based on Zanke's method.

The efficiency of the basin is calculated to be approximately 84%, ensuring reliable sediment removal. Smooth hydraulic transitions are provided for efficient flow regulation and sediment transport. The approach transition begins with an entry width of 5 m and expands to 18 m over a 37 m horizontal length with an opening angle of 10°, followed by a vertical transition of 36.07 m at a 12° angle. On the outlet side, the transition tapers from 32 m to 2.5 m over 56 m horizontally with a steeper vertical transition of 8 m at 30°.

The trashrack at the settling basin outlet is designed with a width of 18 m, a depth of 1.5 m, and an inclination angle of 70°. The clear spacing between the 20 mm bars is 50 mm, resulting in a net area of 18.927 m^2 and a velocity through the rack of 1.31 m/s. The effective open area is 71.4%, ensuring smooth outflow while preventing debris passage.The headpond's spillway design includes a broad-crested weir with a coefficient of 1.7 and a limiting head of 1.0 m. The required spillway length is calculated as 29.2 m, which fits within the 30 m available width of the headpond, confirming the design adequacy for flood handling and overflow conditions.

7.4 Head Pond

The headpond is designed to regulate the incoming discharge from the intake and provide a steady flow into the desanding basin, while also serving as a buffer to settle coarse particles and to manage flow fluctuations. The total design discharge considered for the headpond is 86.272 m^3/s, which is double the main design discharge of 43.136 m^3/s, accounting for both operational flexibility and potential sediment flushing needs. A detention time of 2 minutes is adopted based on standard design recommendations.

To ensure stable hydraulic performance, a uniform flow velocity of 0.25 m/s is maintained, which satisfies the requirement of being greater than 0.2 m/s to prevent sediment deposition within the headpond. The total volume stored in the headpond is calculated as 10,352.64 m^3.

The adopted diameter of the downstream pipe is 4 m, and the submergence head is taken as 6 m based on Baral (2013).The width of the headpond is computed using the continuity equation, giving 31.953 m, which is rounded and adopted as 32 m—consistent with the width of the settling basin to maintain design uniformity and simplicity in construction. The required length of the headpond, calculated from volume and cross-sectional area, is 29.96 m and is rounded to 30 m.

The final dimensions of the headpond are:

- **Total depth:** 12.05 m (including submergence, settling zone, and freeboard)
- **Width:** 32 m
- **Length:** 30 m
- **Flow velocity:** 0.25 m/s

The adopted velocity and dimensions ensure smooth flow conditions while effectively settling larger particles before the flow enters the settling basin. The submergence head of 6 m ensures adequate hydraulic gradient and energy head for downstream flow through the conduit.

7.5. Headrace Tunnel:

The hydraulic design of headrace tunnel is analyzed under static conditions using numerical modelling method i.e. Phase2 software and adopting Generalized Hoek Brown Failure criterion.

Summary of HRT Inverted D:

Lining material -20cm concrete lining with M25 concrete

Length -3050m

Width -4.8m

Depth – 4.8m

Rock Classes- III & IV

Supports used in Headrace Tunnel:

For the design of the support system for Head Race Tunnel, an initial support system was estimated based on empirical correlations and, thereafter, the model was analyzed with a 2-D numerical analysis from Phase2 software. Then from the result
of Phase-2 software support system based on numerical analysis was derived. Static analysis was done one phase 2 for Q value 0.1 of rock type phyllitic quartzite

Supports are:

i. Steel sets- ISMB 150 * 75
ii. Shotcrete with steel fibers – M35, t = 50 mm (Class III), t = 150mm(Class IV)
iii. Concrete lining – 30 cm M25 concrete in invert of tunnel
iv. Rock Bolting – 3m length in spacing of 2 m (Class III) and 3m length in spacing of 1.5m (Class IV)

7.6. Surge Tank:

An underground surge shaft is proposed near the end of headrace tunnel at an offset position at the junction of headrace tunnel and penstock. The surge tank allows mass oscillation, and thus, reduces the impacts of highly fluctuating pressure transients caused by load rejection and load acceptance. Providing optimum lateral rock cover, the offset location from the headrace tunnel has been identified considering the geology and topography of the area and allowing effective simultaneous construction work in surge tank and headrace tunnel.

The diameter of the surge shaft is calculated satisfying the Thoma's stability criterion and considering the topographical conditions. In the surge analysis, the surge tank diameter selected is 18 m and these height of the surge tank is determined to be 24.8 m with respect to upsurge and down surge of water level inside the surge tank including necessary free board and submergence.

The upsurge is estimated considering the maximum operating water level of the reservoir, whereas the minimum downsurge is computed considering minimum operating water level in the reservoir.

Table 15: Surge Tank Design Summary

Design of Surge Tank					
SN	**Descriptions**	Formulae	Value	Unit	Remarks
1	**Data Available:**				
	Acceleration due to gravity (g)	g	9.81	m/s^2	
	Discharge through Headrace Hunnel (HRT)= (Q)	Q	43.136	m^3/s	
	Length of Headrace Tunnel (HRT)= (L)	L	3050	m	
	Diameter of Headrace Tunnel (HRT)= (d)	d	4.8	m	
	Manning's Coefficient (n)	f	0.015		Tunnel design, Range, (0.012-0.018)
	Bend coefficent (Kb)	b	0.1		Assume
	Gross Head (Hg)	Hg	68	m	
	No. of bends in Tunnel (N)	n	1		say
	Head loss in head works	sum of each loss (intake:pipe)	0.2	m	still to sum
	Free Board of Surge tank	F.B	2	m	assume (As per Baral's Book)
	Entrance Coefficient (Ke)		0.5		
	Transient Coefficient (Kt)		0.2		
2	**Head loss in Tunnel loss**				
	Area of Headrace Tunnel (At)=((πd^2)/8)+(d^2)/2	Ap=(πd^2)/4	20.57	m^2	
	Velocity in Headrace Tunnel (v)=Q/A	v=Q/A	2.10	m/s	
	Wetted perimeter (P)=(2d)+(πd/2)	loss1= (Q*f*v^2)/2g	17.1398		
	Hydaulic Radius (R) =At/P	loss2= (f*L*v^2)/2gd	1.20		

	Manning's Head Loss = (v^2)*(n^2)*(L/(R^(4/3))	loss3= (b*n*v^2)/2g	2.37		
	(V^2/2g)*(Kentrance + Kbend + Ktransition)	loss	0.18	m	
	Total Head loss in headrace Tunnel		2.55	m	
	Head loss upto Surge Tank (hf)= Head loss in (head works+Tunnel)		2.75	m	
	Net Head (Ho) = Hg-hf	Ho = Hg-hf	65.25	m	
	Min. area of surge tank Ast>(Ap*L*V^2)/(2g*hf*Ho)	Ast>(Ap*L*V^2)/(2g*hf*Ho)	78.47	m^2	Thomas formula of surgetank
	Area of Surge tank Ast= (45*d^(10/3))/Ho	Ast= (45*d^(10/3))/Ho	128.65	m^2	Baral (2013), page 452
	Diameter of Surge tank (Dst) Dst=Sqrt.(4*Ast/π)	Dst=Sqrt.(4*Ast/π)	12.80	m	
	Final area of Surge tank considering FOS =1.6 (say)	Fos =2 (say)	205.84	m^2	Assume FOS From IS:7396
	Final diameter of surge tank= Dst	Dst	16.19	m	
	Maximum amplitude, Zmax Zmax=(Q/Ast)*((L*Ast)/(g*At))^0.5	Zmax=(Q/Ast)*((L*Ast)/(g*A))^0.5	11.69	m	
	Head Loss correction Factor (Po) = (Total hf / Zmax)	Po = (Total hf / Zmax)	0.23	m	
	Zmax,upsurge Zup = Zmax*(1-(2/3)Po - (1/9)Po^2	Zup = Zmax*(1-(2/3)Po - (1/9)Po^2)	9.79	m	
	Zmax,downsurge Zdown= Zmax*(-1+2*Po)	Zdown= Zmax*(-1+2*Po)	-6.20	m	
	Since the upsurge and downsurge height is very high, the dia. of surge tank needs to be increased to deduct those heights				
	From Table:				
	Taking diameter of surge tank Dst	Dst	18	m	
	Upsurge head from Static level Zupsurge	Zupsurge	8.76	m	
	Downsurge head from static level Zdownsurge	Zdownsurge	5.02	m	
	Normal Water level at Intake RL of NWL at intake	RL of NWL at intake	576.817	amsl	from intake design
	Due headloss, RL of static level at surge tank = RLof NWL at intake - head loss(hf)	RLof NWL at intake - head loss(hf)	574.07	amsl	
	RL of Crown level of Headrace Tunnel		562	amsl	Tunnel Design
	Area of surge tank (Ast)		254.47		
3	**From Penstock Design:**		4.4		
	Diameter of penstock	Dp	4.4	m	
	Area of Penstock Ap = (π*Dp^2)/4	Ap = (π*Dp^2)/4	15.21	m^2	
	Velocity in penstock Vp= Q/Ap	Vp= Q/Ap	2.84	m/s	

	Min. submergence head req. in ST Hsub =1.5* (Vp^2/2g) or 1.5 Dp, whichever is greater	Hsub =1.5* (Vp^2/2g) or 1.5 Dp, whichever is greater	6.6	m	
4	**Final Output**				
	Therefore, RL of min. submergence level req.= RL of crown level of HRT+ Hsub	RL of crown level of HRP+ Hsub	568.6	amsl	
	RL of downsurge from static level = RL at static level of surgetank -Zdown,surge	RL at static level of surgetank - Zdown,surge	569.05	amsl	OK
	Min. submergence head available		7.05	m	OK
	RL of top level of Surge Tank= RL of static level+Zup,surge+F.B+2m extra	RL of static level+Zup,surge+F.B	586.83		
	RL of Crown level of Headrace Tunnel		562	amsl	
	Min. submergence head available = RL of Zdown,surge - RL of Crown level of HRT	RL of Zdown,surge - RL of Crown level of HRP	7.05	m	OK
	Total Height of Surge Tank (Hst) =Top level of ST- Crown level of HRT	Top level of ST- Crown level of HRP	24.83		
	Adopted height of Surge Tank (Hst)		24.80	m	
	Optimised diameter of Surge Tank Dst	Dst	18	m	
	Time of oscillation T= 2π*((Ast*L)/(g*Ap))^0.5	T= 2π*((Ast*L)/(g*Ap))^ 0.5	389.69	sec	
	In 100 sec, Surge height		3.35	m	
	Invert level		562.45	amsl	
5	**Restricted Orifice Design**				
	Coefficient of Resistanc for Orifice (ɳ)		2.4		Range, (1.2-2.8)&(1.5-4) (water In & Out)
	Coefficient of discharge of Orifice (Cd)		0.8		Range, (0.6-0.9)
	Head loss in Orifice (Hor) = ɳ*(Vt-Vp)^2		1.31	m	
	Area of Orifice (Ao) = ((Q^2)/(Cd^2*2*g*Hor))^0.5		5.31	m^2	
	Diameter of Orifice (Do) = ((Ao*4)/π)^0.5		2.60	m	
	Number of Turbine (N)		2		
	Discharge of Turbine (Qo) = Qt/N		21.568	m^3/s	
	Adopted diameter of Orifice (Do)		3	m	

The maximum upsurge and downsurge at the Surge Shaft was adopted to be 582.83 amsl and 569.05 amsl respectively.

7.7. Penstock:

A penstock is a high pressure pipeline between forebay (surge tank or reservoirs) and the turbine. The design principle of penstocks is the same as that of pressure vessels & tanks but water hammer effect has to be considered. For short length, a separate penstock for each turbine is preferable. For a moderate head & long distances a single penstock is used to find two or more turbines through a special branching pipe called Manifold.

i. **Number of Penstock**

   The number of penstocks used at any particular installation can be single or multiple. The general trend at older power stations was to use as many penstocks between the forebay/surge tank and the powerhouse as the number of units installed. The recent trend is to use a single penstock, unless the size or thickness of the penstock involves manufacturing difficulties. When a single penstock feeds a number of turbines, special sections called manifolds are used at the lower end of the penstock to direct flow to individual units. The design of such sections is an intricate job and has to be analyzed carefully. The advantages of using a single penstock over the use of multiple penstocks are:

   - The amount of material required to manufacture is less, making it economical.
   - The cost of civil engineering components such as penstock supports and anchors is less.

ii. **Material of Construction**

   Factors for the choice of material are: head, topography & discharge. Various materials used are steel, R.C., asbestos cement, PVC, wood stave pipes, banded steel, etc. The following factors have to be considered when deciding which material to use for a particular project:

   - Required operating pressure
   - Diameter and friction loss
   - Weight and ease of installation
   - Accessibility of site Cost of the penstock
   - Design life
   - Availability and Weather conditions

7.7.1 Penstock Alignment

Penstock alignment refers to the orientation or path of the penstock, which is a type of pipe or conduit that carries water from a high point to a lower point for the purpose of generating hydroelectric power. The alignment of a penstock is an important consideration in the design and construction of a hydroelectric power plant. The optimal alignment will depend on various factors such as the topography of the terrain, the distance between the source of the water and the power plant, and the overall cost of the project.

In general, a straight or direct alignment is preferred for penstocks as it reduces the overall cost of the project by minimizing the amount of pipe or conduit required. However, if the terrain is steep or rugged, a zigzag or curved alignment may be necessary to reduce the steepness of the penstock and maintain a consistent flow of water. The alignment of the penstock can also affect the efficiency of the power generation process. A smooth and consistent flow of water is necessary to ensure that the turbine operates efficiently and produces the maximum amount of power. Any bends, curves, or sudden changes in direction in the penstock can cause turbulence, which can reduce the efficiency of the power generation process. Therefore, the alignment of the penstock should be carefully considered in the design and construction of a hydroelectric power plant to ensure that it is cost-effective and efficient.

7.7.2. Stresses and Allowable stresses

Longitudinal and circumferential stresses are the two major stresses that act on the penstock pipe. Since the circumferential stress is greater than the longitudinal stress, so the circumferential stress is adopted for the penstock pipe.

$$\text{Longitudinal stress} = \frac{d}{4}$$

$$mn = \frac{d}{2}$$

Shell thickness of the Penstock Pipe,

For internal pressure, the pipe shell thickness is given by:

$$= \frac{d}{2\eta} + \epsilon$$

Where,

t = thickness of the pipe shell in cm

P = internal pressure in kg/cm^2

R = internal radius in cm

= allowable stress in kg/cm^2

η = welding joint efficiency= 0.9

= Corrosion allowance=0.2cm

Similarly, the ASME code gives the following formula as:

$$= \frac{R}{(\eta - 0.6\qquad)} + 0.15$$

Where,

t = thickness of the pipe shell in cm

Pa = internal pressure in kg/cm^2

R = internal radius in cm

= allowable stress in kg/cm^2

η = welding joint efficiency

= Corrosion allowance=0.15cm

However, the minimum plate thickness of the pipe shell is calculated by considering the internal pressure,

Using Pacific Gas & Electric's formula

$$mn = \frac{\quad}{288}$$

Using Bureau of Reclamation Formula

$$mn = \frac{+200}{400}$$

Where,

T = minimum thickness of pipe in mm

D = internal diameter of pipe in mm

The larger value of the thickness is adopted.

A buried high pressure steel penstock (HPP) is proposed immediately after surge tank to transport pressurized water from the outlet portal of a headrace tunnel to a turbine. The HPP is suggested to be buried to avoid negative environmental impacts such as differential expansion and contraction, freezing, and damage from falling debris and trees. The buried penstock continues until it reaches the inlet of the vertical shaft tunnel, where it acts as a penstock shaft.

7.7.3. Penstock Optimization

Penstock optimization is the process of designing a penstock, which is a pipe that carries water to a hydropower plant, in the most efficient way possible. There are two main factors to consider:

- **Cost of materials and construction:** A larger diameter penstock will require more materials and be more expensive to build.
- **Energy loss due to friction:** Water moving through a penstock experiences friction, which reduces the amount of energy available to generate electricity.

Here are some of the techniques that are used in this project for penstock optimization:

A. Optimization Using Cost and Revenue:

Penstock is a closed conduit waterway which conveys water from intake to power house. The optimum diameter of penstock is determined through the optimization between the cost associated with friction head loss and the material & installation cost of penstock. Head loss cost itself depends on power purchase cost, project life and average hours of operation of the power plant and other parameters.

Then a simplified arrangement is developed in MS Excel where the material cost, the cost associated with the head loss in penstock and other parameters which may influence these costs can be analyzed. These costs are plotted in graph and the optimum diameter is calculated graphically. Finally, the major parameters affecting the optimum diameter of penstock are identified. It was found that the tariff and material, installation and maintenance cost of penstock are the vital parameters which play the major role during optimization of penstock.

The penstocks diameter is determined through an optimization process. For a given design flow, the penstock diameter is inversely proportional to the fluid velocity and thus inversely proportional to the square of the head loss. A parabolic curve relating the cost of the Energy loss over the powerhouse lifetime for different diameters can then be drawn. Another curve relating the pipe cost of the penstock to its diameter is superimposed on the same graph. Then another curve is obtained which is the sum of both curves which is the total cost curve. Then the optimal diameter is taken from the lowest point of the total cost. The graph is shown below:

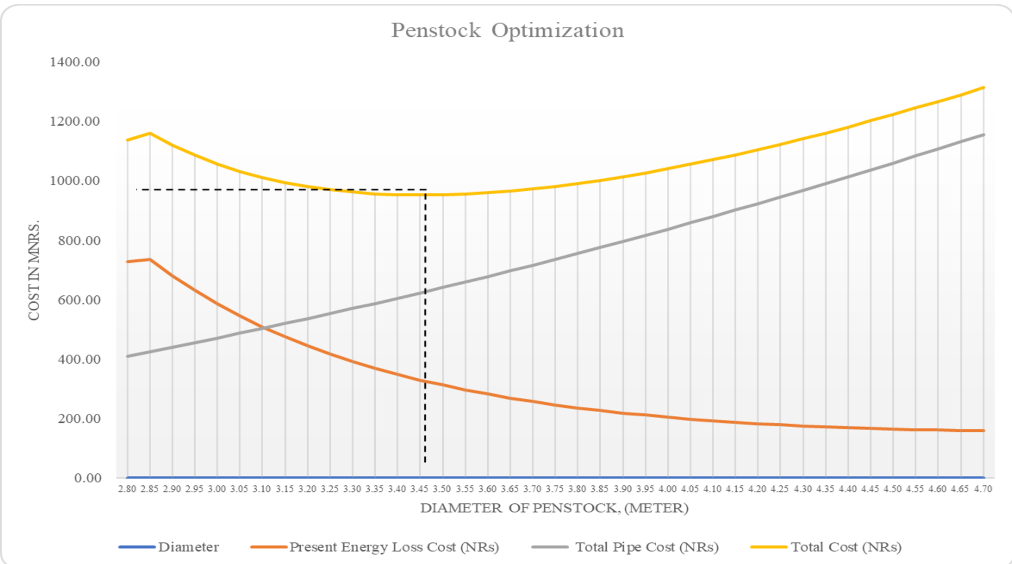

Figure 14: Graphical representation of penstock optimization

The figure show the graph plotted between pipe diameter and cost. The optimum diameter is obtained by drawing a straight line from the lowest point of the total cost curve.

B. Optimization using IS 11625-1986

Economical Diameter Formula:

$$D = \frac{2.36*106*3*n2**p*p}{\left[1.39 + 0.6 + \frac{121\,H\ 1+}{} *p \quad *1 \quad (\quad)\right]}$$

Where,

Q = Discharge

n = Rugosity coefficient in Manning's formula

C_o = Unit Cost of concrete lining in Rs. / m^3

C_e = unit cost of excavation in Rs. / m^3

C_p = cost of 1 kWh of energy in Rs.

C_s = cost of steel in Rs. / Kg

D = diameter of the penstock

e = overall efficiency of plant

e_t = Joint efficiency of penstock

H = head on penstock including water hammer in m

i = percentage by which steel in penstock is overweight due to provision of stiffeners, corrosion allowance, etc.

p = ratio of annual fixed operation and maintenance charges to construction cost of penstock

p_t = annual load factor

Derivation of the formula for calculating the economic diameter of penstock

a. Cost Of Power Lost

Head Loss in Penstock/ Meter Length

Actual headloss calculation in shown in the ANNEX below in energy calculation table.

Head loss due to friction is given by Manning's formula:

$$H = \frac{v^2 n^2}{R^{4/3}} = \frac{10.29\ {}^2 n^2}{16/3}$$

Annual cost of power lost (Et) = 9.804 * Q * h_f * e*p_f * 8760c_p

$$E_t = \frac{0.88*106\ Q^3\ n^2\ e\ p_f\ c_p}{D^{16/3}}$$

b. Annual Charges on Capital Cost

Cost of excavation

The cost due to excavation for laying the penstock is calculated considering the tunnel diameter to be 33 D in excess of penstock diameter total cost/unit length of penstock is given by:

$$\frac{}{4}(+0.33)^2 * \; = 1.39^2$$

Cost of Concrete Lining

Cost of concrete lining in penstock has been calculated taking thickness of lining as 0*165 D, Thus, cost of concrete lining is given by:

$$(+0.165) * 0.165 * \; = 0.6^2$$

Cost of Steel in penstock

$$\text{Steel lining thickness, } = \frac{pd}{2} = \frac{0.1H}{2}$$

$$nk = \frac{1+(2\ 7850)*}{*} = \frac{2 * 9.81}{} \quad \frac{120.93H^2 * (1+)}{}$$

Annual Charges on Capital Cost

The cost of penstock is expressed by:

$$_p = \frac{[^2\ 1(39 + 0.6 + 120.93H\ 1 + \quad (\quad)] * p}{}$$

c. Economic Diameter

$$\text{Total Annual cost (E)} = {}_p +$$

Economical diameter is obtained by:

$$\frac{(_p +)}{D} =$$

$$^{22/3} = \frac{2.36 * 10^6 * {}^3 * n^2 * \; p *}{\left[1.39 + 0.6 + \frac{121H\ 1+(\quad)}{} * p\right]}$$

d. Optimization Using E. Mosonyi

Economical Diameter Formula:

$$D = \left(\frac{(0.01+0.001* \; **)*2*3}{1000*1*H}\right)$$

Where,

= Friction coefficient of pipe

= Allowable stress C_2

= Rate in kWh

= Discharge

C_1 = Annual rate of maintenance

H_g = Design Head

7.8. Powerhouse

Various options for the location of powerhouse have been investigated for this project. The proposed powerhouse location has been mostly directed by the constraints mentioned in the alternatives that are discussed below. The layout of surface, semi-underground as well as fully underground powerhouse was taken into consideration. Considering the rock type, sufficient cover for placing powerhouse, tailrace and substation, an underground powerhouse has been proposed on the right bank of Seti River.

The powerhouse accommodates 2 generating equipment each with a capacity of 11.75MW and ancillary facilities for control and protection. Size of powerhouse is 45.5 m long, 17.25 m wide and 27 m high bottom. Considering the head and flow available in the site, Francis turbine with vertical setting has been selected.

7.8.1. Turbine Selection

Maximum net head acting on the turbine is one of the most important criteria dictating type of turbine to be used for the power station under consideration. In general, Pelton turbines are recommended for high heads, Francis for medium heads, Kaplan for low heads and bulb for very low heads.

Table: Selection of turbine according to Head and Discharge

According to the Head and Discharge			
	Value	**Unit**	**Remarks**
Gross Head(H_g)	68.00	metres	Low head and High discharge
Net Head (H_n)	63.90	metres	
Discharge (Q)	43.14	cumecs	
Choice of turbine based on Head vs Discharge Graph	**Francis Turbine**		

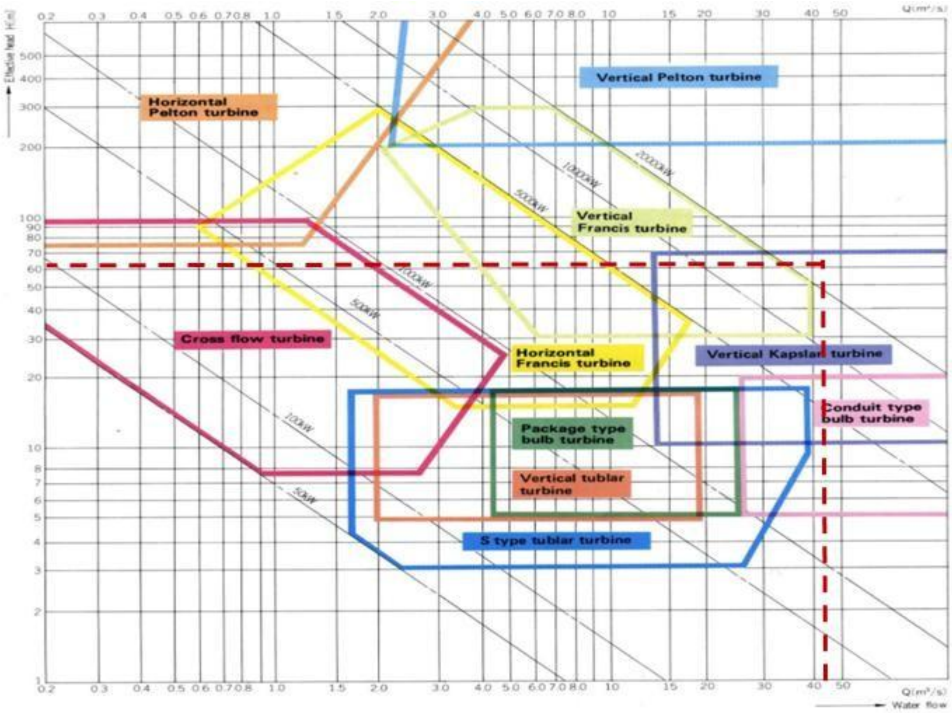

Figure 15: Turbine selection based on Head vs Discharge (DoED, 2018)

Table 16: Selection of turbine according to Specific Speed

According to the specific speed			
	Value	**Unit**	**Remarks**
Turbine efficiency	0.90		Assumption
Generator efficiency	0.97		Assumption
Transformer efficiency	0.99		Assumption
Overall efficiency	0.87		
No. of units (n)	2.00		
Discharge per unit (q)	21.57	cumecs	
Power generated by each unit (P)	11,694.88	kW	23389.7642
Frequency (f)	50.00	Hz	
No. of poles (p)	8.00		Assumption
Speed in rpm (N)	375.00	rpm	
Specific Speed (N_q)	77.06		
Choice of turbine based on head vs specific speed graph	**Francis Turbine**		

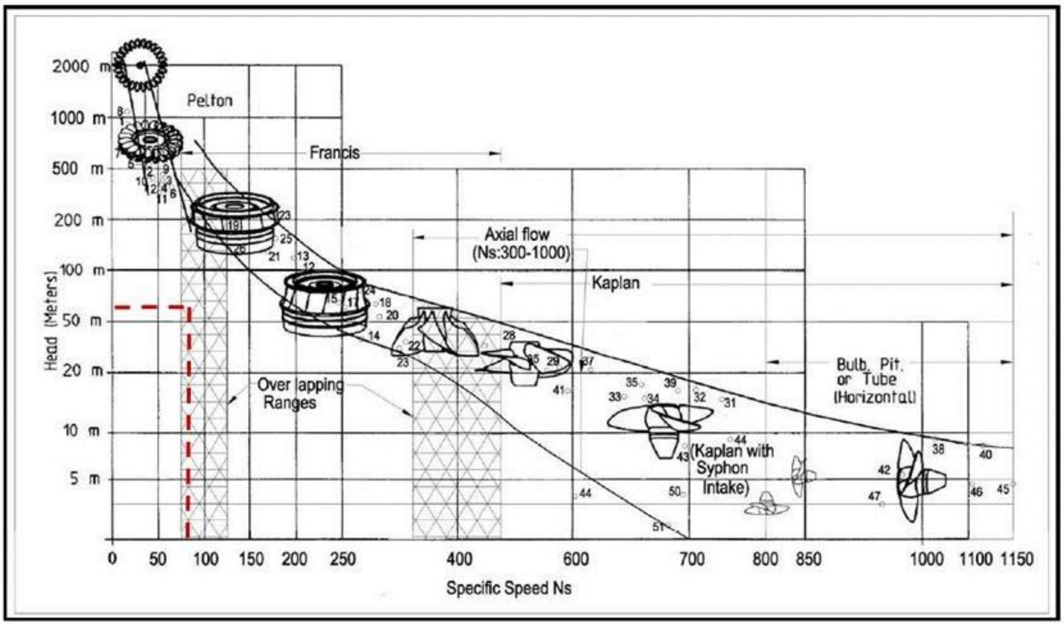

Figure 16 Turbine selection based on Head and Specific Speed (DoED, 2018)

Choice of turbine is normally dictated by rated head consideration. In case of overlapping heads where two types of turbines could be considered suitable more detailed analysis need to be carried out when a number of factors are favouring different types of turbines.

Table 17: Selection of turbine according to Speed Number

According to the speed number			
	Value	**Unit**	**Remarks**
ω	39.27		Angular velocity
ϖ	1.11		Speed factor
Q	0.61		
Ω	0.87		Speed number
Choice of turbine based on Speed number	**Francis Turbine**		

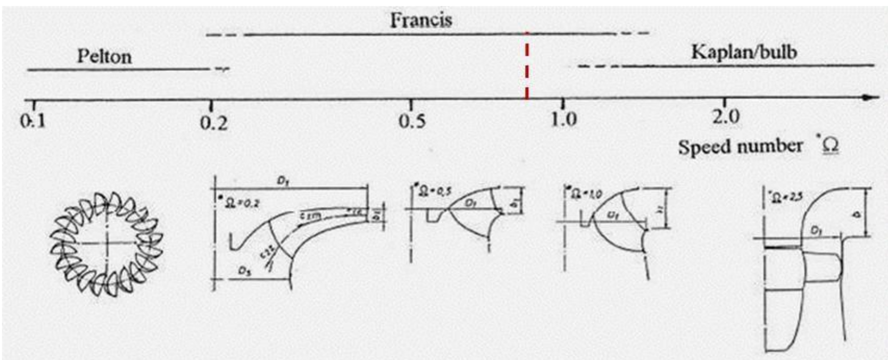

Figure 17: Turbine selection based on Speed Number (DoED, 2018)

7.8.2. Powerhouse Dimensioning

Turbine design has been done following Powerhouse Design Guidelines provided by the DoED in 2018 with reference from the methods compiled by IS 12800-part1 and Emil Mosonyi in his book Water Power Development.

The rotational/synchronous/rated of the turbine in revolutions determined from the following formula:

Table 18: Calculation of actual specific speed according to IS12800

Synchronous Speed			
Particular	**Value**	**Unit**	**Remarks**
Machine type			Francis
No of units	2.00		
Unit capacity			
Maximum Head	68.00	m	
Rated Head	63.90	m	
Power	11,694.88	KW	
Take, specific speed	230.00		Fig. 1.1 IS12800
Synchronous speed of machine	329.73		
Synchronous speed for 10 pairs	300.00		
Synchronous speed for 8 pairs	375.00		
Assuming the head variation less than 10% (higher synchoronous speed i.e 375)			
Corrected specific speed	**261.58**		

In case the turbine setting, to have a cavitation free runner at a given specific speed, is found to be very low resulting in uneconomical construction of power house, the specific speed may be reduced by decreasing the speed of rotation. In reaction turbines, the setting of turbine with respect of minimum tail water level should be fixed from the consideration The suction height of distributor of cavitation. center line above the minimum tail water level can be determined from the following formula:

Table 19: Calculation of suction head requirement

Turbine setting			
Particular	**Value**	**Unit**	**Remarks**
H_b	10.30	m	atm pressure (absolute)
H_v	0.40	m	Vapour pressure
σ (from formula)	0.29		Thoma's coefficient
σ (from graph)	0.18		Thoma's coefficient
Hs	-1.60	m	Suction head
	-2.10	m	0.5m extra

The runner discharge diameter D for Francis turbine is determined by the peripheral velocity coefficient Ku, which is defined as:

Table 20: Runner Diameter calculation

Runner Diameter			
Particular	**Value**	**Unit**	**Remarks**
Ku	0.84		Fig. 1.3 IS12800
D	2.24	m	

Metallic spiral casing should be used for gross heads generally above 30 meters. The major dimensions of the spiral casing indicated in the table below is obtained as a function of constant referred to runner diameter D from the IS code 12800-1 (1993).

Table 21: Spiral Casing calculation

Spiral Casing			
Particular	**Constant**	**Value**	**Unit**
A	*1.15*	2.58	m
B	*1.30*	2.91	m
C	*1.50*	3.36	m
D	*1.70*	3.81	m
E	*1.20*	2.69	m
F	*1.48*	3.32	m
G	*1.22*	2.73	m
H	*1.10*	2.46	m
I	*0.28*	0.63	m
L	*1.00*	2.24	m
M	*0.61*	1.37	m

Similarly the size of draft tube is determined accordance with IS 5496: 1969 which is beefily mentioned in the table below.

Table 22: Size of Draft Tube

Size of draft tube			
Particular	**Constant**	**Value**	**Unit**
h	*0.94*	2.11	m
H	*2.50*	5.60	m
L	*4.00*	8.96	m
B	*2.60*	5.82	(no pier)
Particular	**Value**	**Unit**	**Remarks**
v	3.52	m/s	
Min Submergence	0.63	m	
Keeping bed slope 1 vertical to 10 horizontals at the bottom of the draft-tube, the exit end	0.90	m	above the bottom of draft-tube.

of draft-tube			
Top of exit end of draft-tube	3.00	m	above bottom of draft tube
Top of the exit end of the draft-tube	-2.60	m	below the minimum tail water level

For generators, the air gap diameter Dg should be large enough to allow the turbine runner top cover to pass through the stator bore. Here the outer core diameter, stator frame diameter, inner diameter of generator barrel, core length of stator and its frame along with the height of load bearing bracket were obtained and calculated accordance to the code.

Table 23: Generator Parameters calculation

Generator parameters			
Particular	**Value**	**Unit**	**Remarks**
No. of pole	8.00		
Rated KVA of generator	12,994.31		power factor 0.9
peripheral rotor velocity (Vt)	83.00	m/s	Fig. 15 IS12800
Air gap diameter (Dg)	6.06	m	
Outer core diameter (Do)	7.25	m	
Stator frame diameter (Df)	8.45	m	
Inner diameter of generator barrel (Db)	10.25	m	
Core length of stator (Le)	0.22	m	
Length of stator frame (Lf)	1.72	m	
Axial hydraulic thrust (Ph)	88.72	tonnes	
Weight of generator rotar (Wr)	37.70	tonnes	
Weight of turbine runner	10.00	tonnes	
Total load	136.42	tonnes	
Load per arm	34.11	tonnes	4 arms
k	0.65		for less than 50 tonnes
Height of load bearing bracket (hj)	1.89	m	Suspended type construction

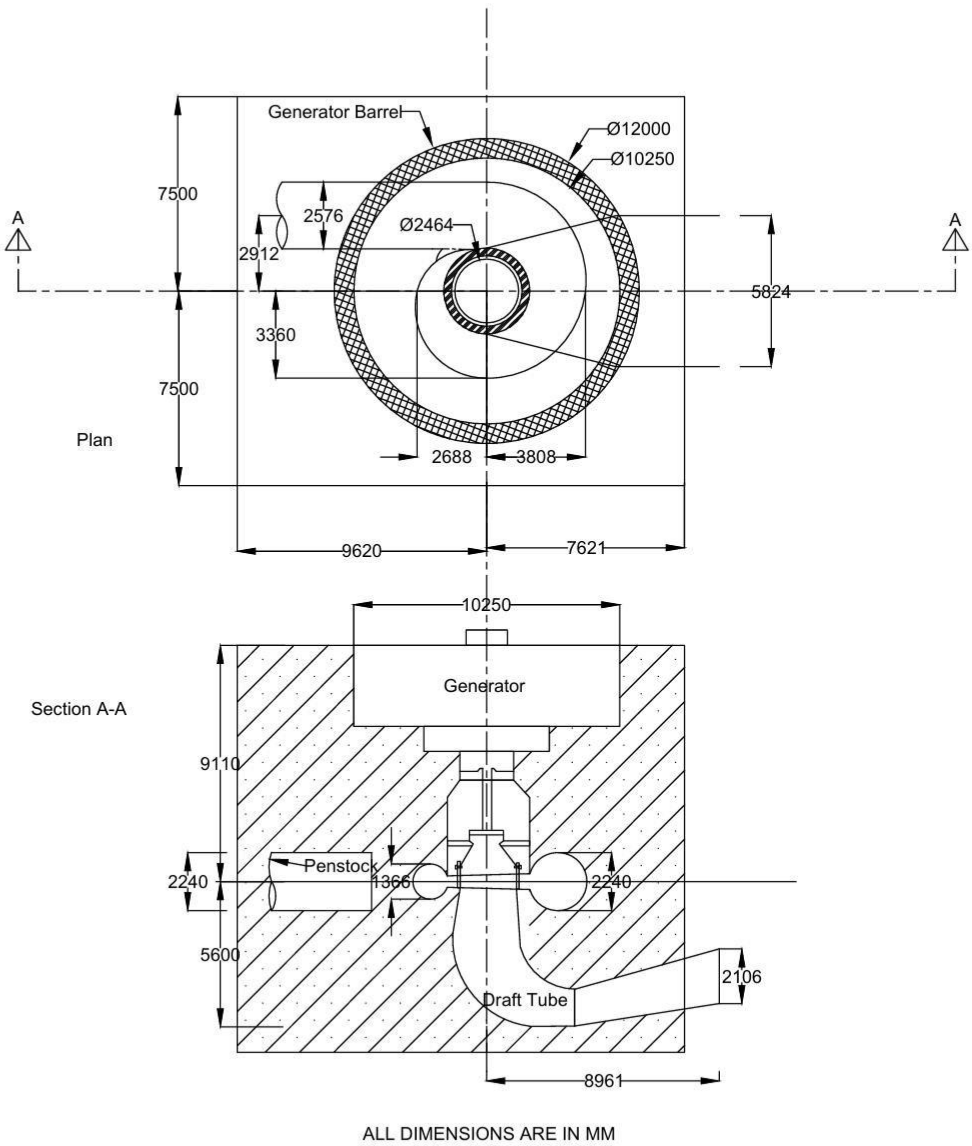

Figure 18: Dimension of unit bay (plan and section):

The overall dimension of the unit space, length of erection bay and the length required for the E.O.T crane to handle last unit are mentioned in the table below which is calculated in accordance to the IS code for preliminary dimensioning and layout of surface hydro-electric power houses part 1.

Table 24: Overall dimension of Powerhouse

Overall dimension			
Particular	**Value**	**Unit**	**Remarks**
Spiral inlet	4.20	m	
Opposite side	5.62	m	
unit spacing	11.75	m	
	15.00	m	say
length of erection bay	12.00	m	0.7 to 1.5 time unit bay size
Space required for the E.O.T crane	3.50	m	preliminary purspose (3 to 5)
Total length of machine hall	**45.50**	**m**	
d/s width from c/l	7.62	m	
u/s width from c/l	9.62	m	
Total width of machine hall	**17.25**	**m**	
Height of draft tube (H1)	5.60	m	
Height of Generator (H2)	9.11	m	for suspended type
Total height (machine)	**14.71**	**m**	
Clearance height	1.00	m	
Height form erection bay to E.O.T crane	7.00	m	IS 12800 part 1
Height between EOT and ceiling	4.00	m	
Total height (superstructure)	12.00	m	
Total Height (all)	**26.71**	**m**	

7.8.3 Preliminary layout of machine hall

- Layout of Column:

The preliminary layout of columns in machine hall is generally determined considering the building's architectural and engineering plans. The placement of column determines the span of the beams it has to support. The span should be minimized as much as possible without compromising the adequate spacing required for accessibility of personnel and vehicles as well as penstock pipe, draft tube and other auxiliary requirements.

For our project, we have adopted the center-to-center column spacing in x-axis as 5.75 meters having total of four columns along the width of machine hall. Similarly, center-to-center spacing in y-axis as 6.5 meters having total of seven number of columns along the length of machine hall of the powerhouse.

Table 25: Preliminary design of machine hall column

Particulars	**Value**	**Unit**	**Remarks/ Reference**
Wt. from crane	500.00	kN	Powerhouse Design Guideline
Load from gantry	5.00	kN	
Total Load on rail runway	2,500.00	kN	
Load from gantry on each column	12.80	kN	
Truss Load	80.00	kN	assume
Self Wt of beam	2.70	kN/m	
Weight of beam on column	17.55	kN	
Load Transferred to Column	3,115.35	kN	
Factored Load, Pu	4,673.03	kN	Assuming factor 1.5
Gross Area	236,608.86	sq. mm	Cl. No. 39.3, IS 456:2000
Dims. Of Column	486.42	mm	
Adopted Size of Column	**500x500**	**mmxmm**	
Unsupported Length of Column, L	4.00	m	
Effective Length of Column, L_e	2.60	m	both fixed support
D_{min}	500.00	mm	
Check for Short Column			
Ratio (L_e/D)	5.20		short column (<12)

- Layout of Beam:

Based on its function, two types of beams are provided in the powerhouse superstructure.

i Crane Beam: Its primary function is to support and transfer the load from EOT crane to the columns. The height of the crane beam is determined by the height required for operation of EOT crane and spans throughout the machine hall. For the purpose of this project, the crane beam is provided at the height of 8 meters from the generator floor along x-axis throughout the machine hall.

ii Cross Beams: Connecting beams runs parallel to the crane beams which provide additional support and rigidity to the building's superstructure. It is mostly provided at the height of 4-6 meters from the generator floor.

Table 26: Preliminary design of machine hall cross beam

Beam Along Y-axis			
Particulars	**Value**	**Unit**	**Reference**
Longest Span of Beam, L	6.50	m	
L/D_{eff} =26 *Modification factor			
For Modification Factor,			
Compressive Strength, f_{ck}	25.00	Mpa	Assume
Yield Stress,f_y	500.00	Mpa	Assume
$f_s = 0.58 \times f_y \times \frac{Area\ of\ steel\ requied}{Area\ of\ steel\ provided}$	290.00		Assume Area of steel provided is same as that of required
Modification Factor, α	0.70		IS 456, Fig.4
Effective Depth of Beam, D_{eff}	357.14	mm	
Adopting Effective depth of beam	360.00	mm	
Effective Cover	40.00	mm	
Overall Depth of beam, D	400.00	mm	
Width of the beam	266.67	mm	assuming 2/3 * D
Adopted Depth of Beam	400.00	mm	
Adopted Width of Beam	270.00	mm	

7.8.4. Design of Roof Truss

A truss is a framework for supporting loads that is formed of straight elements arranged in triangles. Roof trusses are characterized by an economic use of construction materials (timber, steel). Composed of individual lightweight pieces, a truss can also provide considerable advantage in transport and assembly. Literatures and online articles were reviewed to find the suitable type of truss which was Howe Truss for the span of about 17 m.

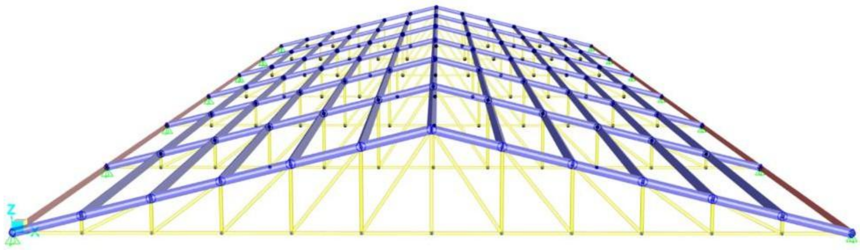

Figure 19: 3D model view of Howe roof truss in SAP2000

Procedure for Truss design in SAP2000

1. The tentative dimensions of the powerhouse machine hall where the roof is to be placed was calculated.
2. The loads acting on the truss structures were calculated manually using the IS codes whose detail calculation are mentioned in the ANNEX. Three categories of loads on the trusses were incorporated as:
    a. Dead load: It is basically the own weight of the materials used. These forces act vertically. IS 875 (Part 1): 1893 code was referred to calculate and define the dead loads. The dead loads includes:
        - Self-weight of truss structure
        - Weight of purlins and CGI roof cover sheet
    b. Wind load: IS 875 (Part 3): 2015 code was referred for defining the wind load.
    c. Live load: IS 875 (Part 2): 1987 code was referred for defining the live load.
3. The load combinations were made as follows in accordance to the DoED powerhouse guideline:
    a. Case 1: Live load + Dead load
    b. Case 2: Wind load + Dead load
4. After the truss was defined, the horizontal and vertical loads calculated were then assigned to the respective intermediate and end nodes of the truss.
5. The support of the truss was kept as hinged structure.
6. The loads were applied on the respective joints. The steel section was also defined and the analysis was done until all members passed.

Table 27: Adopted steel section on roof truss

Members	Design Section
Top Chords	ISNB 175M
Bottom Chords	ISNB 65M
Vertical Web	ISNB 65M
Inclined Web	ISNB 65M
Intermediate Purlin	ISNB 175M
End Purlin	ISNB 125M

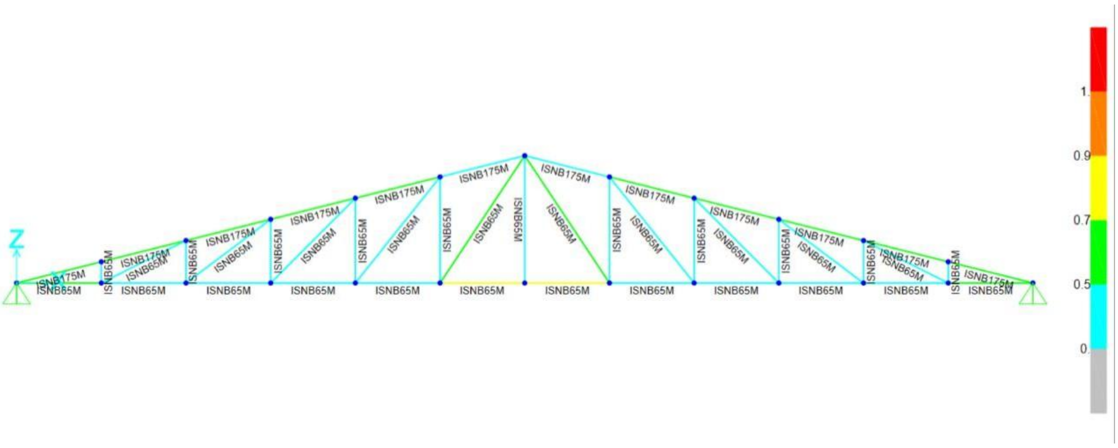

Figure 20: Demand/Capacity ratio passed steel section of roof truss

7. The stress was checked for all the load combinations using the IS: 1161-1998 code. The steel sections were checked and varied until the stress is within the limit value.

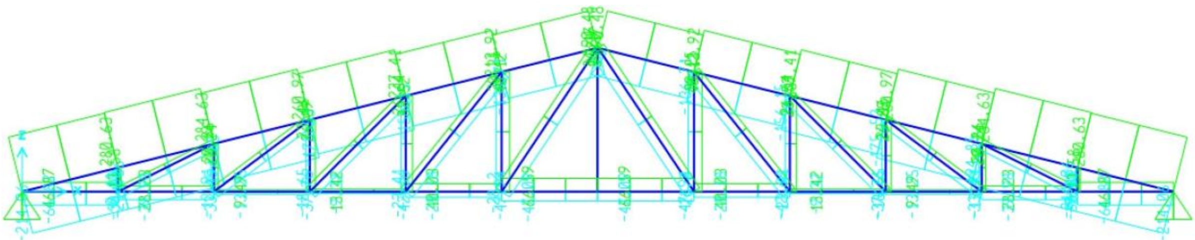

Figure 21: Maximum Axial Force on truss member

8. Finally, the horizontal and vertical reactions at the support were noted that need to be placed in the column as a point load for further analysis.

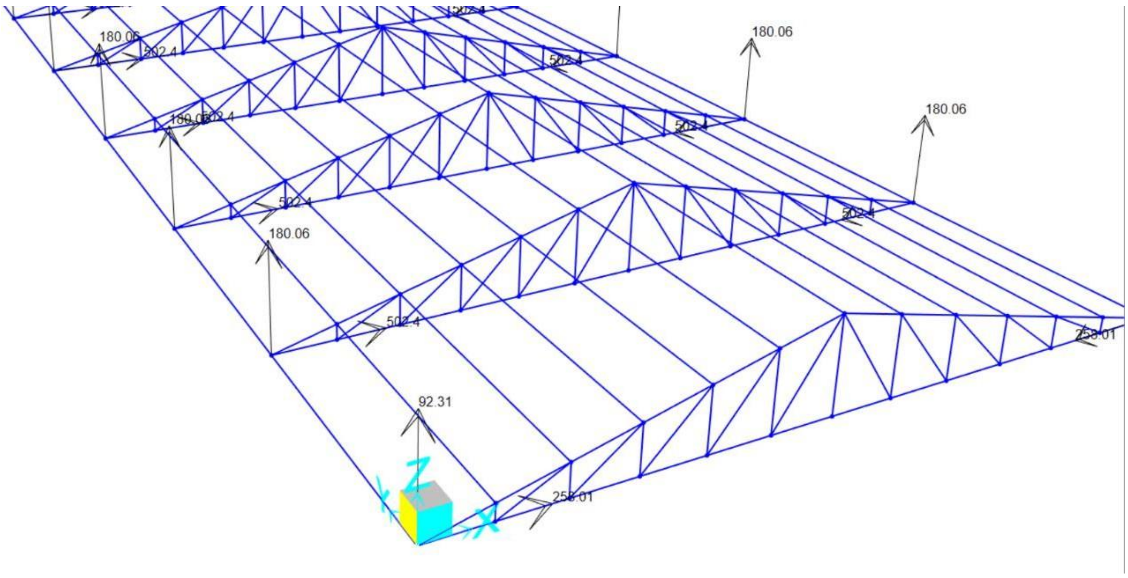

Figure 22: Maximum reaction forces of roof truss on hinge support

7.8.5. Structural Analysis of Machine Hall Superstructure

1. After the completion of the design of the roof truss, the preliminary design of the powerhouse was done in accordance with Powerhouse Design Guidelines, 2018.
2. The structure was then modeled as a reinforced concrete frame in SAP2000 with the materials for concrete (Grade: M30) and steel reinforcement bars (Fe500) defined. The beam and column sections for the powerhouse were also defined whose preliminary design calculation was done as mentioned in ANNEX.

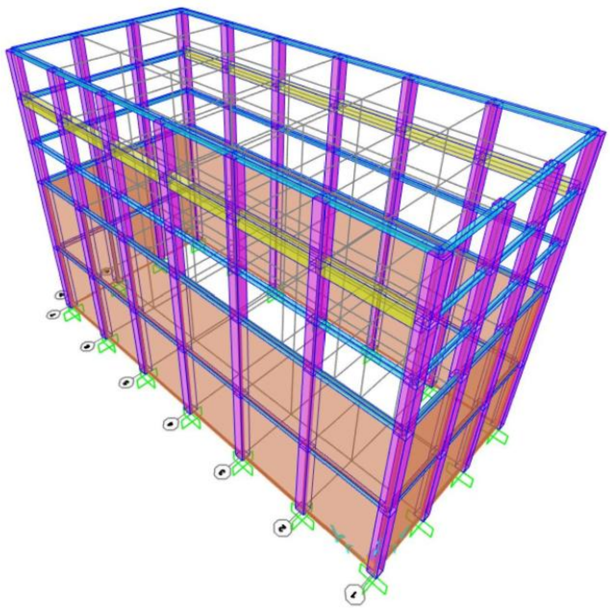

Figure 23: 3D model of Powerhouse Machine Hall in SAP2000

3. The loads from roof trusses were applied to the powerhouse model.

4. The weight of brick wall on the superstructure was applied to the beams as a uniformly distributed load whose detail calculation in mentioned in the ANNEX.

5. The crane load was added as moving load (vehicle live) to the path of crane beam.

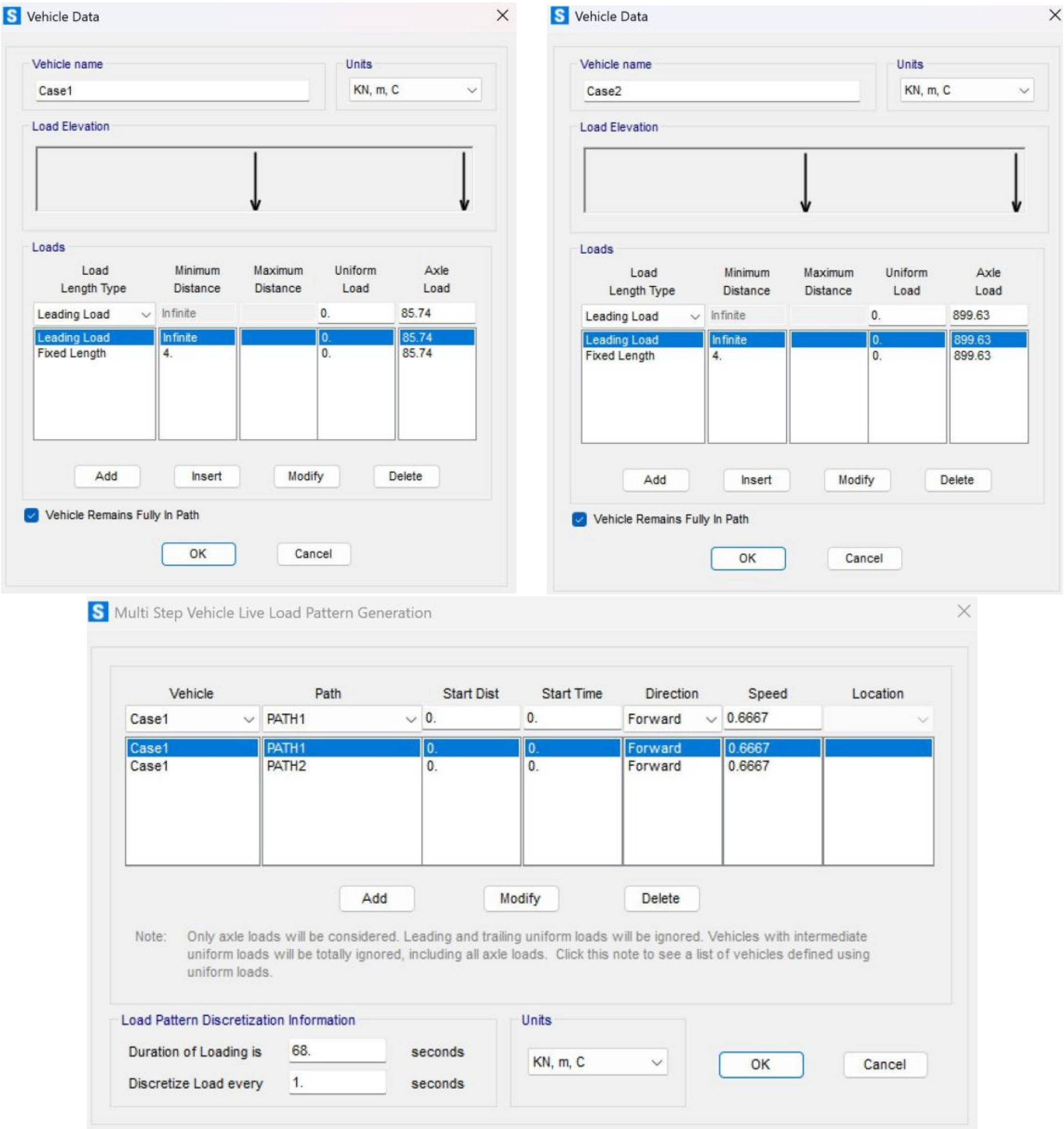

Figure 24: Assigning moving crane wheel load on define path (crane beam)

6. And finally Seismic parameters were also incorporated in the model from two prevailing codes in Nepal i.e IS 1893(part1):2016 & NBC 105:2020

Parameters	Value	Remarks	Reference
Sesmic Zone factor (Z)	0.36	Zone V (Nepal)	*IS 1893:2016, Table 3*
	0.3	Pokhara	*NBC 105:2020, cl 4.1.4*
Importance Factor (I)	1.5	Power station building	*IS 1893:2016, cl 7.2.3*
Response reduction factor, R	5	SMRF	*IS 1893:2016, cl 7.2.6*
Ductility Factor, Ru	4		*NBC 105:2020, Table 5-2*
Over-strength Factor, Ωu	1.5		
Height of building	12	m	*Considering superstructure*
Fundamental Time Period, T	0.48	RC MRF building	*IS 1893:2016, cl 7.6.2*
	0.64		*NBC 105:2020, cl 5.1.2*
Soil Type	1 & A	Rock or hard soil	

7. The model was analyzed with the default design combination of loads and checked for possible failure of the members. In case of failure of any of the members, the size of the frame element was increased until it could sufficiently carry the forces generated.

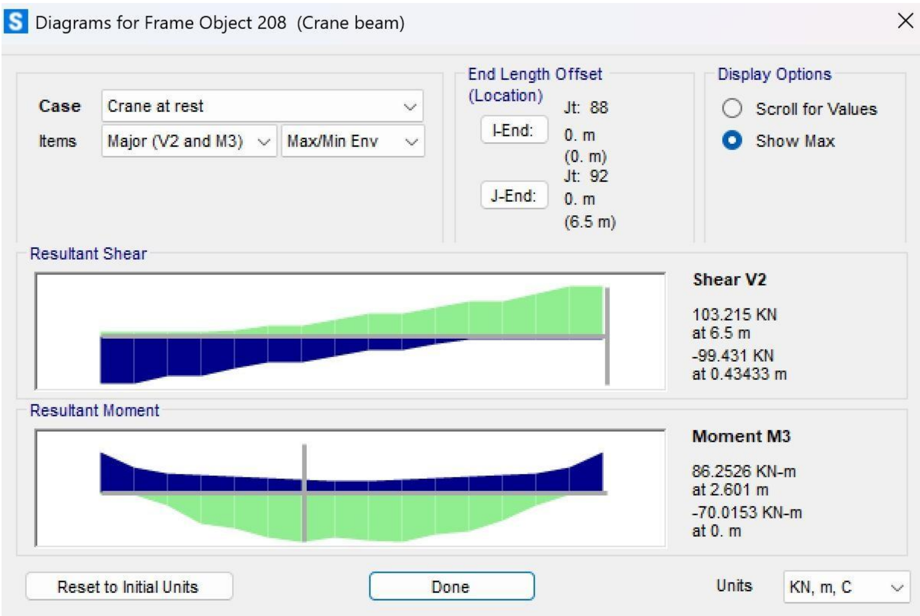

Figure 25: Maximum BM & SF diagram of crane beam due to moving crane wheel load (case1)

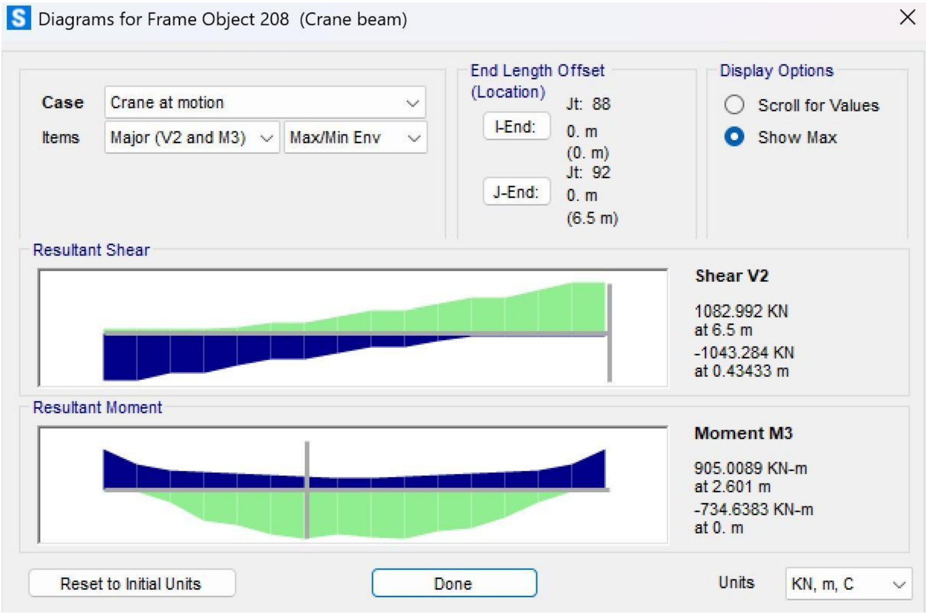

Figure 26: Maximum BM & SF diagram of crane beam due to moving crane wheel load (case2)

8. The calculated maximum forces and moments were then used to determine the required reinforcements for the sections.

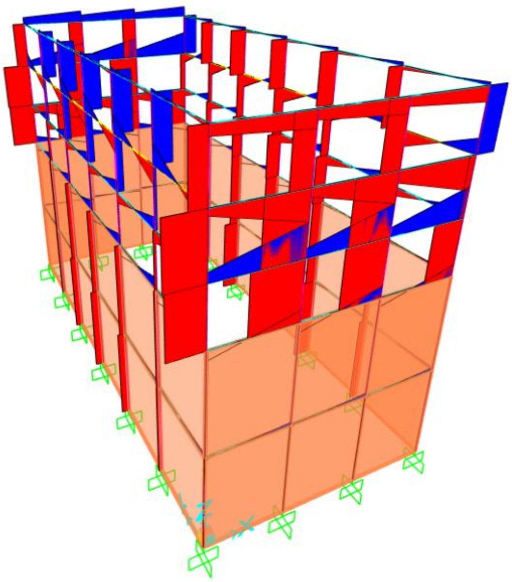

Figure 27: Maximum SF diagram of superstructure in load combination 2

8. STATIC ANALYSIS AND SUPPORT DESIGN OF HEADRACE TUNNEL

Static analysis of hydropower tunnels involves evaluating the stability and structural integrity of the tunnel structure under various static loads and conditions. This analysis is crucial for assessing the performance of tunnels under normal operating conditions and during construction. In the context of hydropower tunnels, this analysis typically focuses on the analysis of the tunnel lining and surrounding rock mass.

The main goal of static analysis in hydropower tunnels is to determine the structural adequacy of the tunnel under static loads and to ensure that it can withstand the anticipated loads without failure or excessive deformation.

8.1. Static Analysis of Headrace Tunnel:

The static analysis of headrace tunnel is analyzed under static load conditions using Generalized Hoek Brown Failure criterion. Following rock parameters and GSI values are adopted calculated at first for the estimation of support design using numerical modelling method i.e. Phase2 software.

Adopted Rock Properties:

Rock Type: Phyllitic Quartzite

Poisson's ratio, ν: 0.26

Unit weight, : 27.80 KN/m3 Modulus

of elasticity: 42910.00 MPa UCS:

105.96 MPa

Geological Strength Index, GSI: 45.00

Tectonic Stress: 5.00 MPa

Field Stresses Calculation:

The in-situ stresses can be calculated as:

Maximum overburden of headrace tunnel (H) = 432 m

Unit weight () = 27.8 KN/m3

The value of vertical stress (v) is calculated as,

$$v = \text{ H} = 27.8*432 = 12.01 \text{ Mpa}$$

The values of maximum horizontal stress (h) is calculated as,

$$h = / (1-)*v + tctnc$$

= 0.26/ (1−0.26)*12.01 + 5×cos (45)

= 7.75 Mpa

Where,

h = horizontal stress or principal stress or 1

v = Vertical stress or 3 Out-

of Plane Stress () = 3

tn = Tectonic stress (4-5 MPa) +Cos

= Rock orientation, =45 for HRT

= Poisson's ratio, =0.26 for Phyllite

8.2. Numerical modelling in Phase2:

The estimated tunnel supports are analyzed using Numerical Modeling using the Phase2 program. The software uses the finite element method to analyze various geotechnical problems such as slope stability analysis, tunneling, excavation design, and foundation design.

For the modelling a plane model of the D-shaped tunnel with excavation boundary of 5.2m*5.2m is built to determine the deformation far from the face of the tunnel and the radius of the plastic zone. The boundary was discretized and mesh surface was created. Field stress were then provided and uniform load was applied for 10 different stages with stage factors 1, 0.8, 0.4, 0.2, 0.1, 0.08, 0.04, 0.02, 0.01, 0. Factor = 1 means the magnitude will be the same as the field stress while a Factor = 0 means no load will be applied at that stage. Other values of factor can be used to increase or decrease the magnitude of a load at any stage of a model. The amount of deformation in the wall of the tunnel before the support installation is determined. Then the support is added and whether the tunnel is stable, the deformation meets the specified requirements and the tunnel lining meets certain factors of safety requirements is identified.

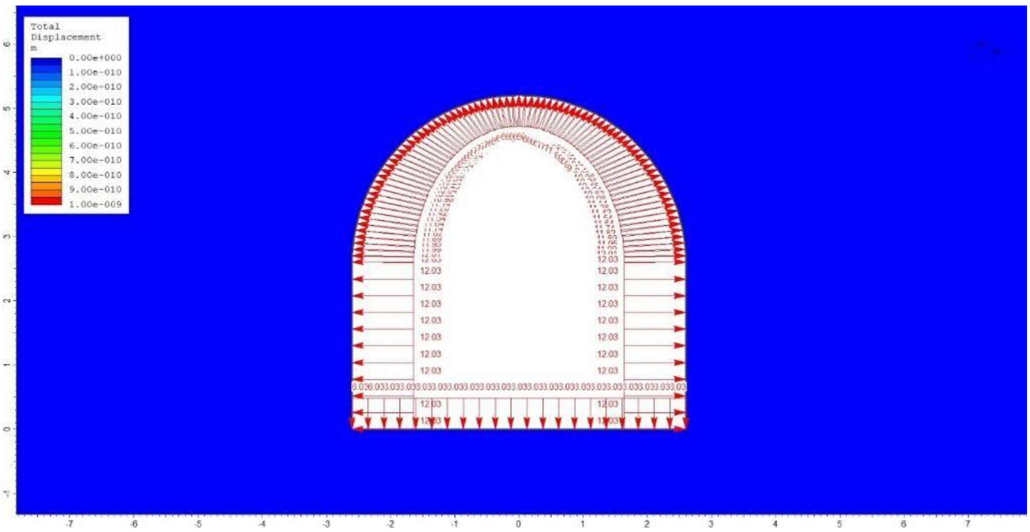

Figure 27: Tunnel face on application of load

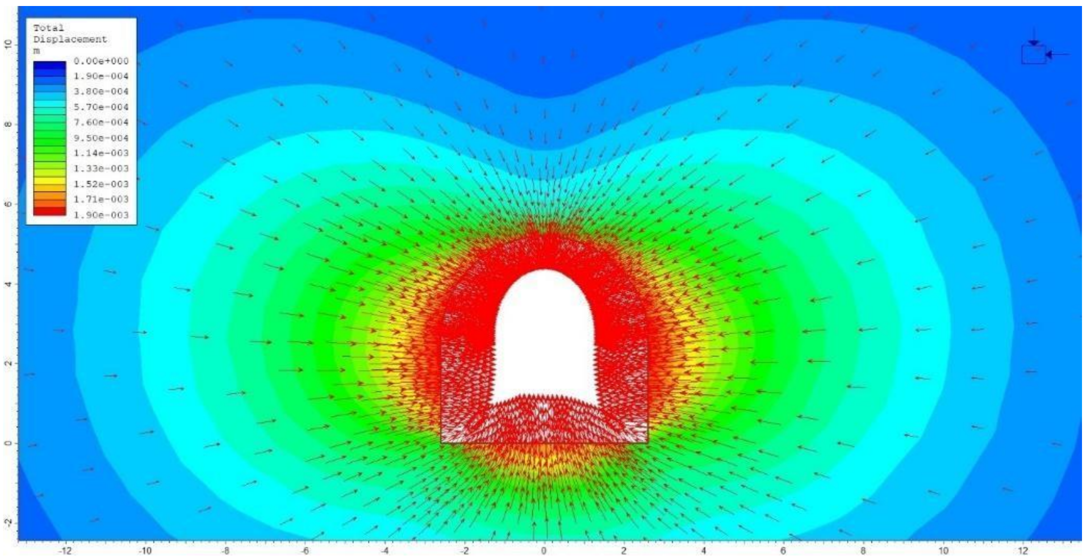

Figure 28: Deformation obtained from plastic analysis without application of support

To compute the tunnel deformation at the point of support installation, we'll use the empirical relationship developed by Vlachopoulos and Diederichs.

To use the Vlachopoulos and Diederichs method, we need two pieces of information from the finite-element analysis. We need to know a) the maximum tunnel wall displacement far from the tunnel face, and b) the radius of the plastic zone far from the tunnel face.

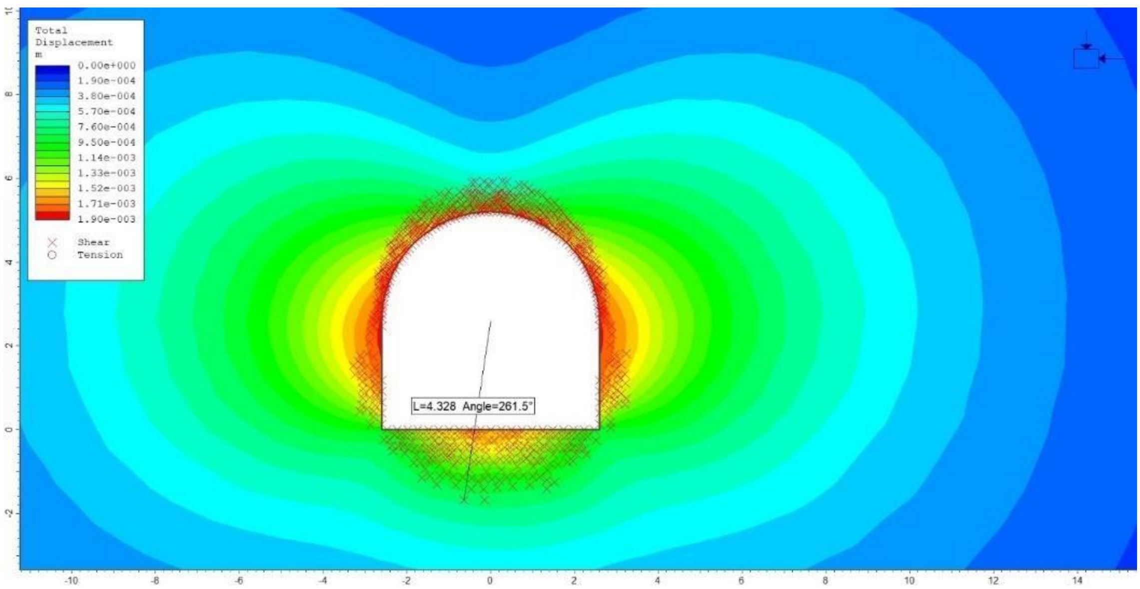

Figure 29: Plastic zone Radius

Calculation of closure:

Table 28: Calculation of Closure Value

Class III- rock type	
For Graph	
Tunnel radius	2.6
Distance from tunnel face	2
Data from Phase2 software	
plastic zone radius(m)	4.328
Max total displacement(m)	0.002
ratios for graph	
Distance from tunnel face/ Tunnel radius	0.769
Plastic zone radius/ tunnel radius	1.665
From graph the value of Closure/Maximum Closure is	0.64
Closure	0.001

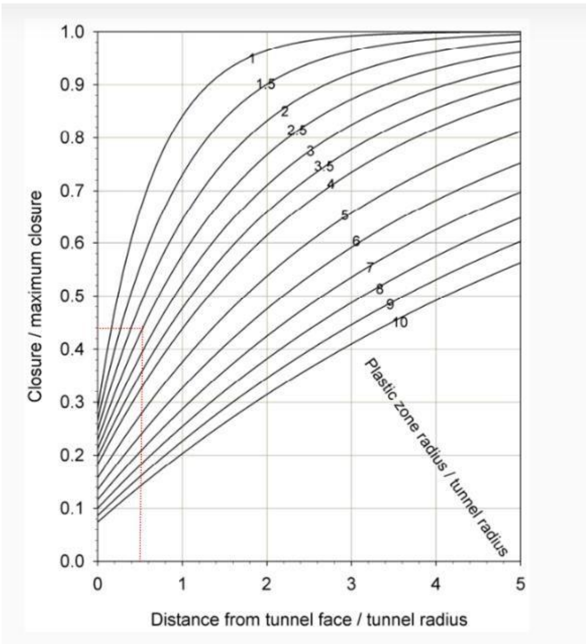

Figure 30: Closure length graph for deformation of tunnel

Now, the internal pressure factor for the given displacement can be determined.

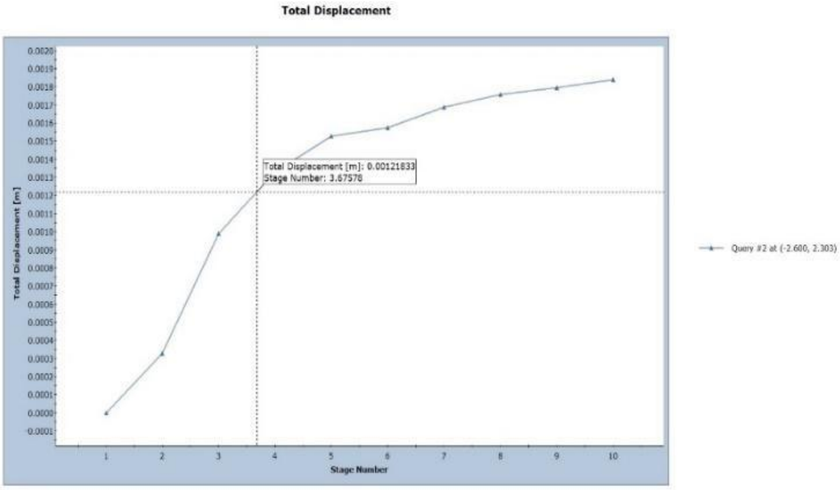

Figure 31: Stage vs Displacement curve showing the stage with the required closure

From this plot, we can see that in stage 4, the wall displacement in the face of the tunnel is 0.0012m. This represents an internal pressure factor of 0.2 as defined in the model for field stress vector distributed load. As the displacement is near to stage 4. So, Stage 4 is taken as tunnel relaxation stage.

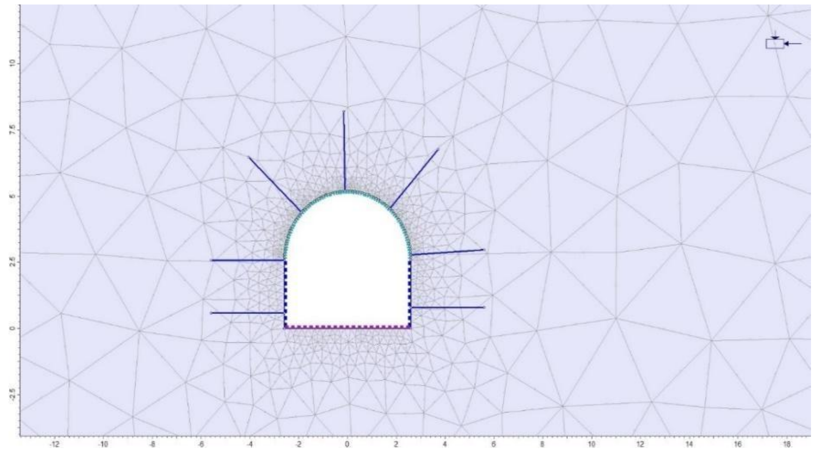

Figure 32: Installation of Tunnel support for Rock class-III

Figure above shows the tunnel supports like rock bolts, steel fibre reinforced shotcrete and the concrete lining on the invert of tunnel and it is furthered analysed.

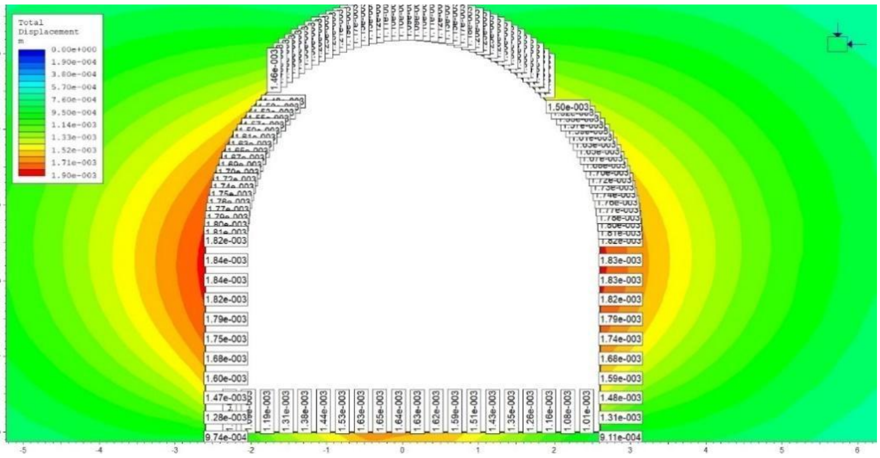

Figure 33: Displacement along tunnel face before support installation

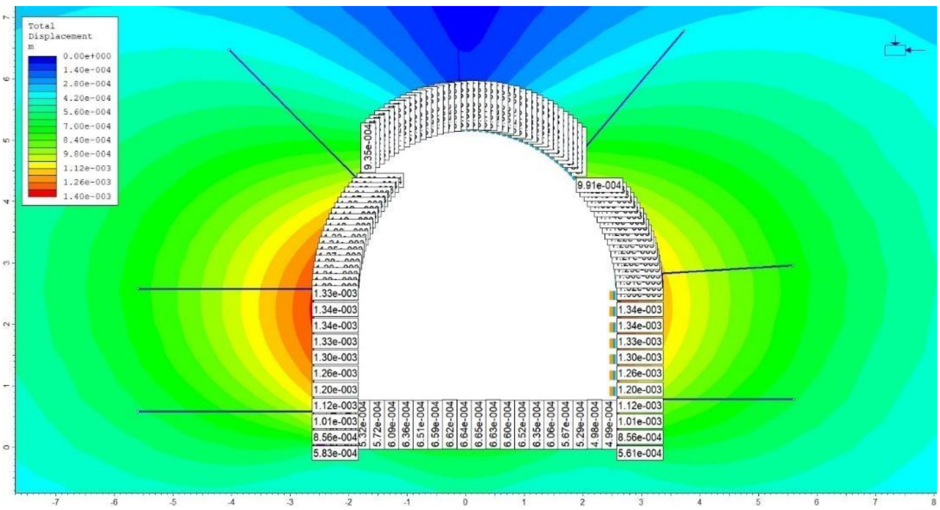

Figure 34 : Displacement along tunnel face after support installation for Rock class-III

Table 29: Value of maximum total displacement

Description	Max. Total displacement(mm)
Before support Installation	1.84
After support Installation	1.34

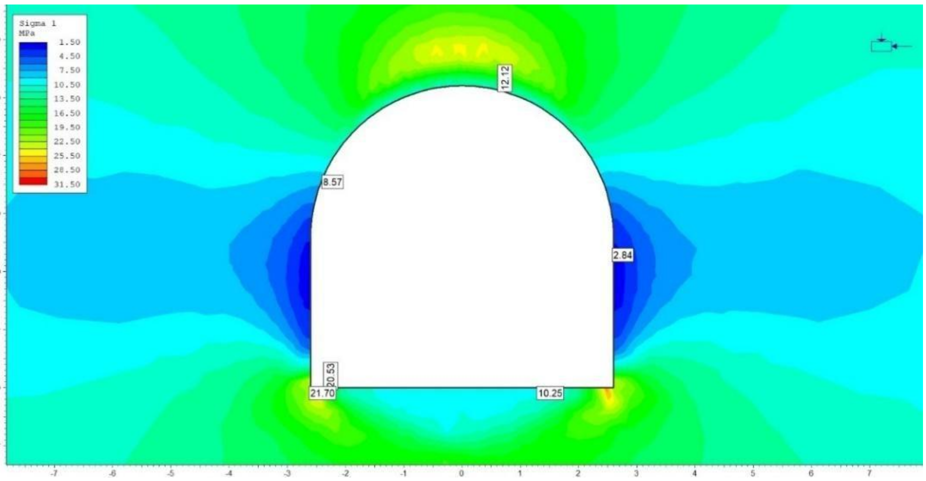

Figure 35: Stress (Sigma 1) before support installation for Rock class-III

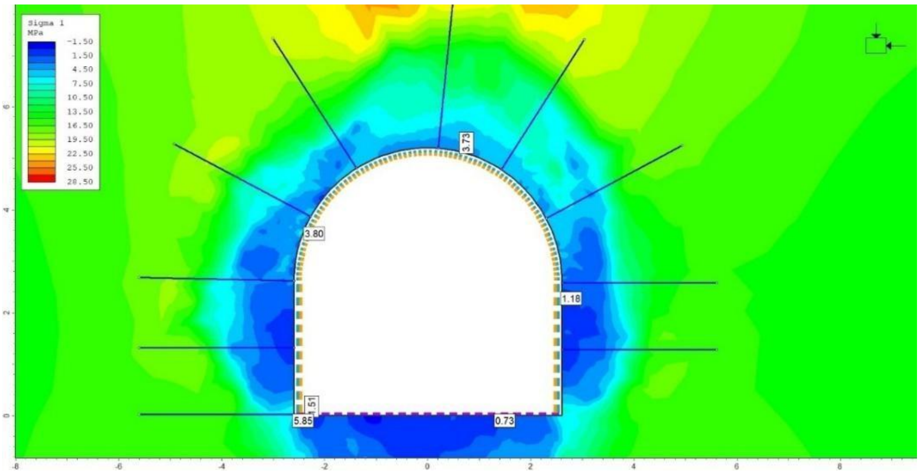

Figure 36 : Stress (Sigma 1) before support installation for Rock class-III

Table 30: Stress (Sigma 1) of Tunnel before and After Support Installation in 2D Analysis

Description	Stress (sigma 1, Mpa)	
	Maximum	Minimum
Before support Installation	21.70	2.84
After support Installation	5.85	0.73

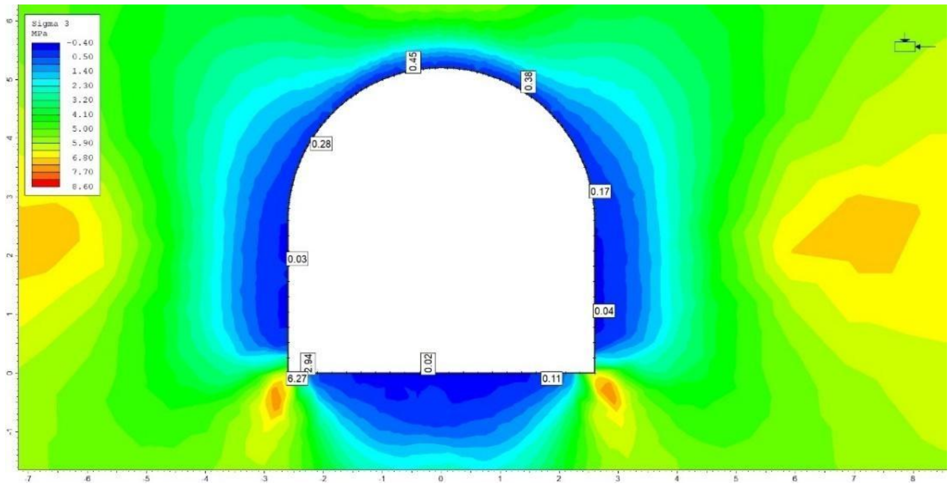

Figure 37: Stress (Sigma 3) before support installation for Rock class-III

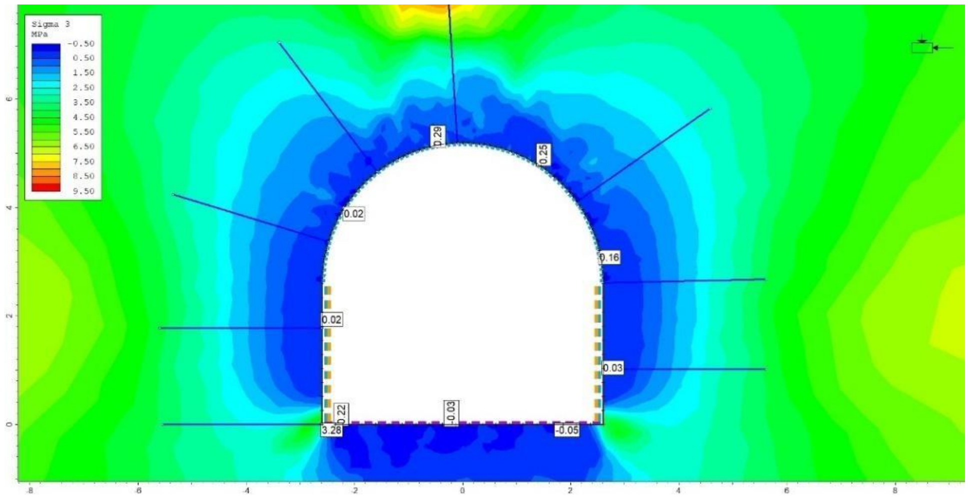

Figure 38: Stress (Sigma 3) after support installation for Rock class-III

Table 31: Stress (sigma 3) along Tunnel face before and after Support Installation in 2D Analysis

Description	Stress (sigma3, Mpa)	
	Maximum	Minimum
Before support Installation	6.27	0.22
After support Installation	3.28	0.03

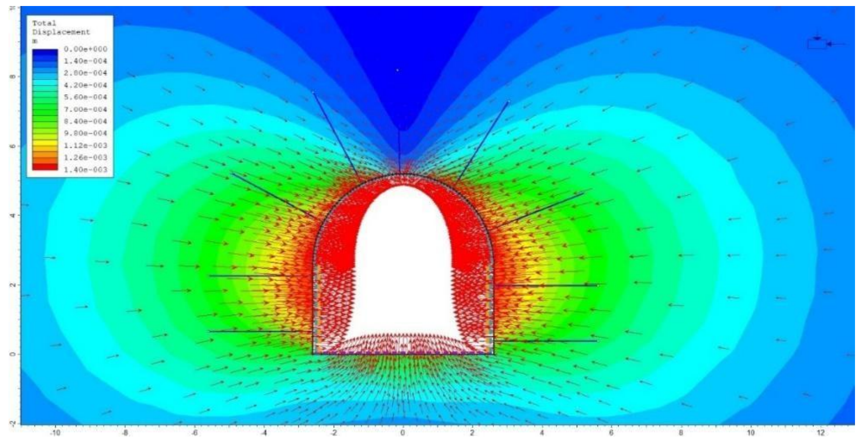

Figure 39: Deformation obtained from plastic analysis with application of support

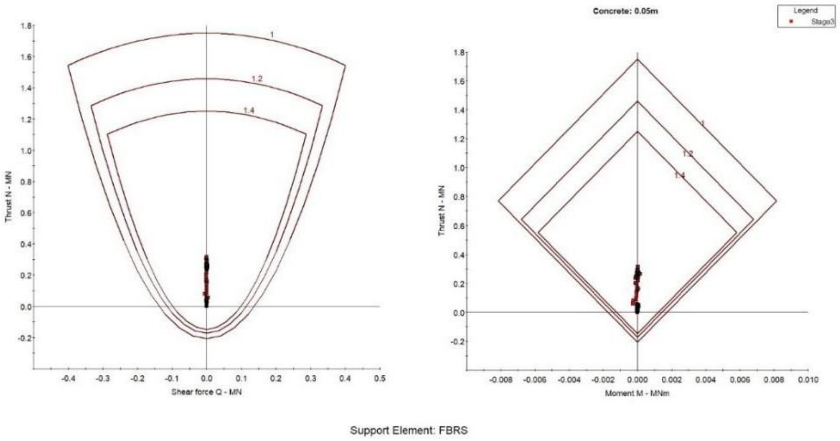

Figure 40: Support capacity plots of 50mm SFRS for Rock class-III

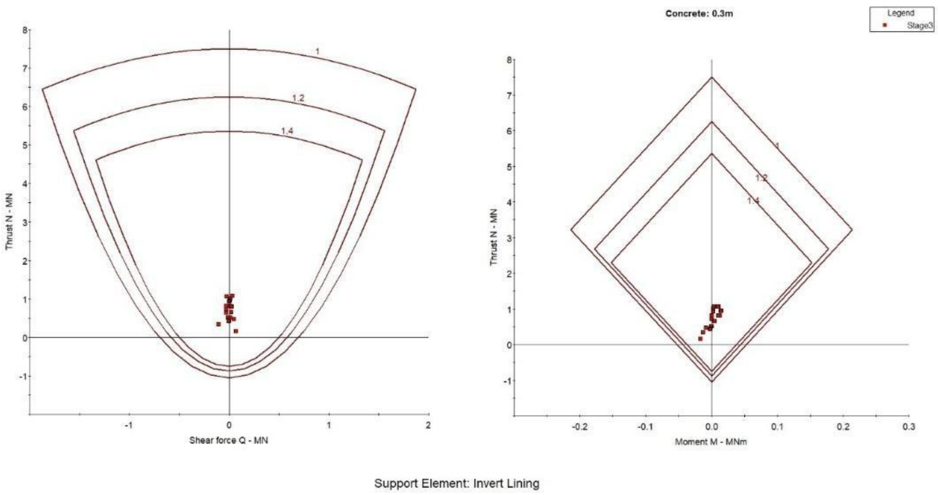

Figure 41: Support capacity plots of 30 cm M25 invert concrete lining for Rock class-III

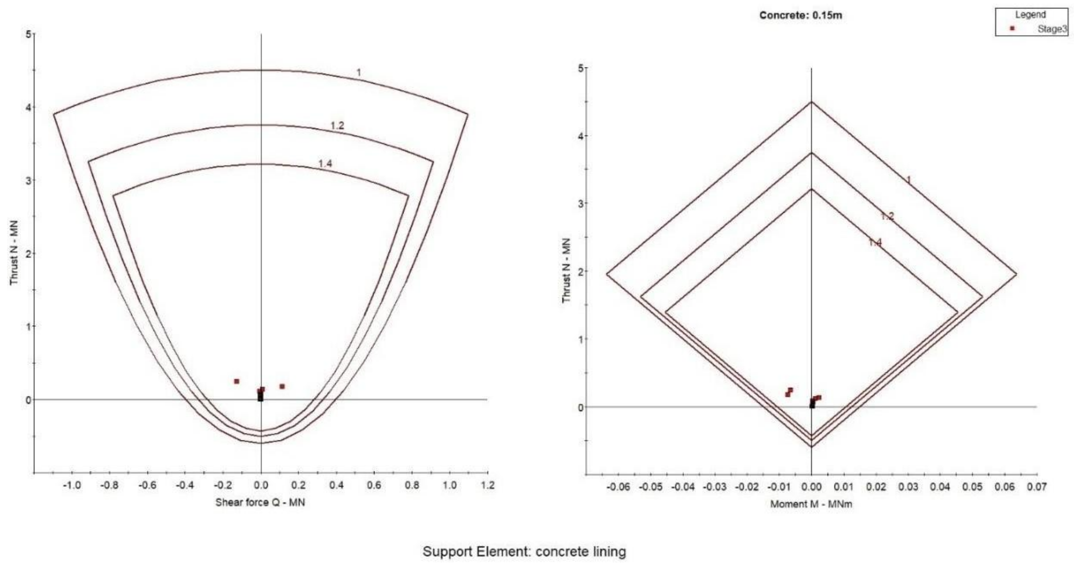

Figure 42: Support capacity plots of 15 cm M25 concrete lining on tunnel benching only for Rock class-III

The support capacity plots for the steel fibre reinforced shotcrete and concrete lining all resulted safe within safety factor of 1.4.

Analysis for the Rock class IV:

The numerical analysis for rock class IV is analyzed using following rock properties

Adopted Rock Properties:

Rock Type: Phyllitic Quartzite

Poisson’s ratio, v: 0.26

Unit weight, : 27.80 KN/m3 Modulus

of elasticity: 42910.00 MPa UCS:

105.96 MPa

Geological Strength Index, GSI: 45.00

Tectonic Stress: 5.00 MPa

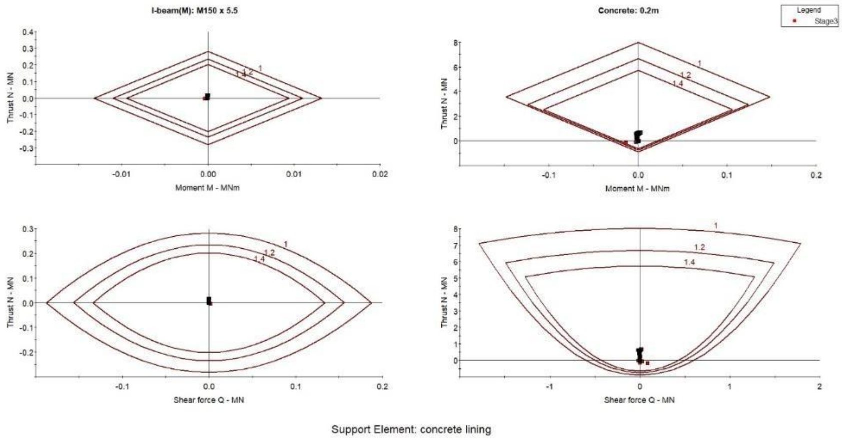

Figure 43: Support capacity plots of ISMB 150 Steel Ribs with 20 cm M25 concrete lining on the tunnel face for Rock class-IV

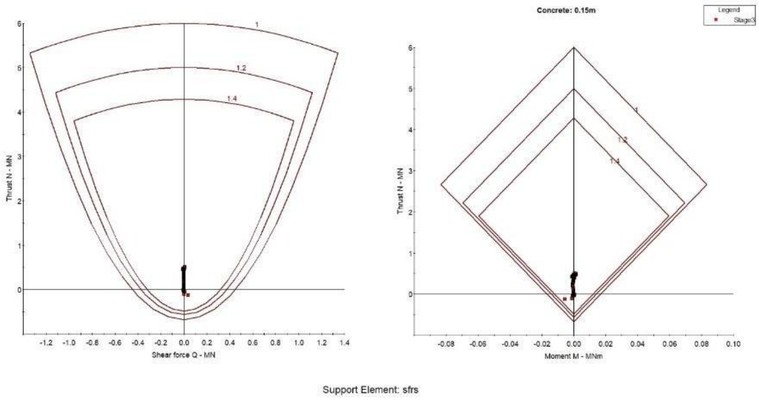

Figure 44: Support capacity plots of 15cm of SFRS lining of Rock class-IV

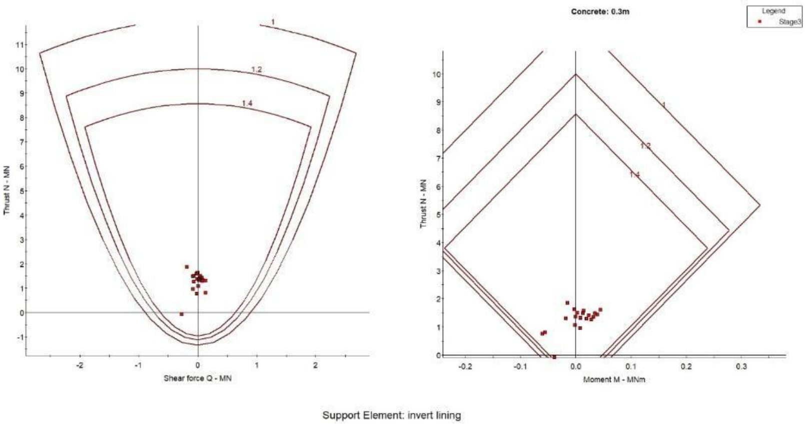

Figure 45: Support capacity plots of 30cm of M25 concrete for invert lining for Rock class-IV

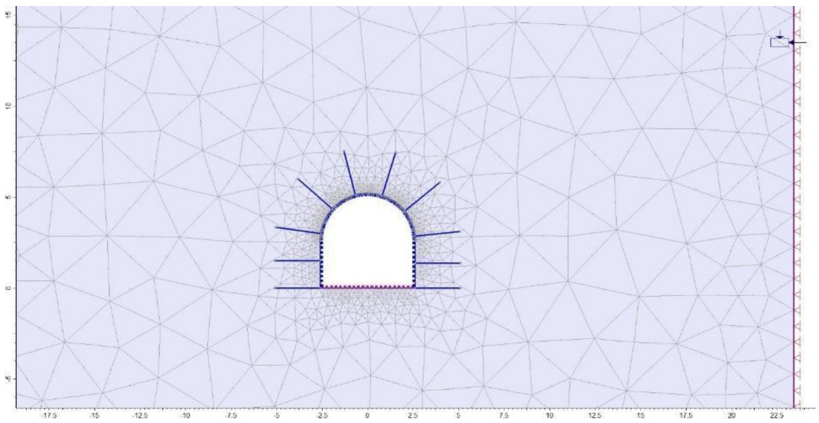

Figure 46: Support installation for Rock class-IV

Summary of recommended support:

Support recommendation is made on the basis of the empirical relations and the finite element analysis. The support recommendations are presented in table below:

Table 32: Rock Support Design Result Summary

Rock / Designation	**Rock Mass Classification**	**Recommended Support (Detail Geological Report, Hydro-consult Engineering, 2016)**	**Estimated Support as per Phase2**
Class III	0.5 < Q <1 38 < RMR < 44	Bolts in pattern 1.6 m spacing. 5cm fiber shotcrete at crown and walls. Invert Concrete Lining	Bolt pattern of 3m length and 25mm diameter at 2m interval Walls and Crown: 50 mm Steel Fibre Reinforced Shotcrete (M35) Invert: 30cm Reinforced M25 concrete.
Class IV	0.01 < Q < 0.1 23 < RMR < 38	Bolts in pattern 1.4 m spacing. Fiber shotcrete: Crown = 10cm, Walls = 5cm. Invert concrete lining.	Bolt pattern of 3m length and 25mm diameter at 1.5m interval Walls and Crown: 150mm Ribs Reinforced Shotcrete (M35, Rib-ISMB 150*75) with 20cm M25 concrete lining. Invert: 30cm Reinforced Concrete, M25

Section Details:

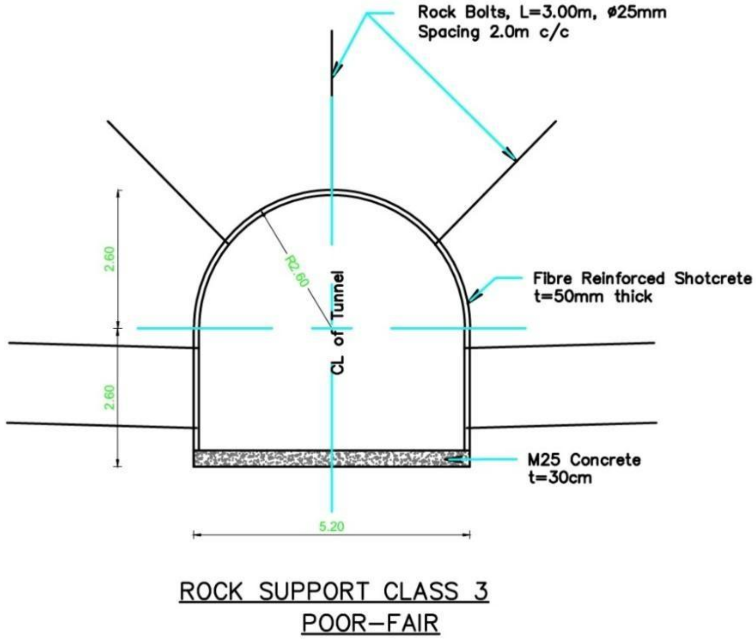

Figure 47: Tunnel section of Class III rock support

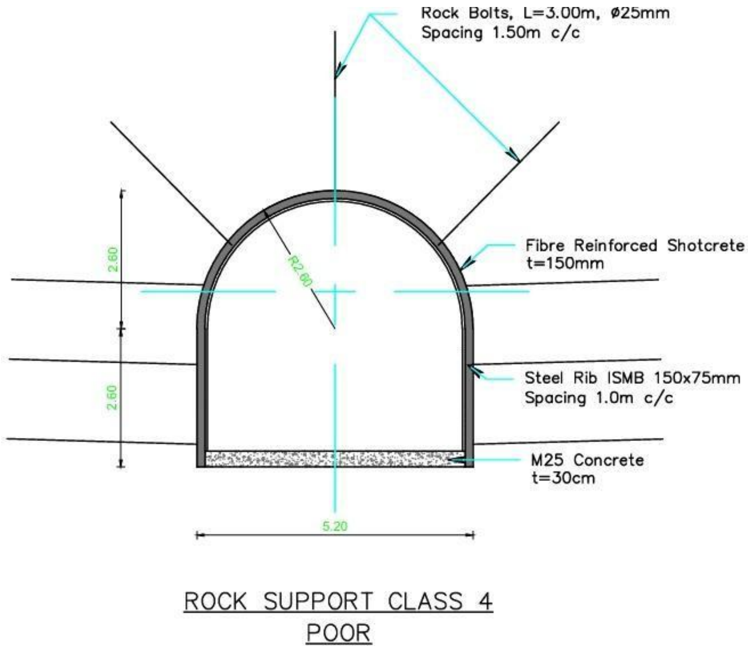

Figure 48: Tunnel section of Class IV rock support

9. ECONOMIC ANALYSIS

9.1. Cost and Quantity Calculation

Cost and quantity calculation is an important part in carrying out the economic analysis of any project. It provides a clear understanding of the financial requirements and resource allocations needed for the project activities implementation. In this context, the cost and quantity estimation report serves as a crucial document and the basis to derive the total project cost estimate.

The estimation process begins with the identification of quantities required for various aspects of the project, which are obtained from designs and drawings. The unit rates for different construction activities are determined based on district rates and government norms, ensuring accuracy and reliability in the estimation process.

Rate analysis is conducted to estimate the quantities of different items for each work, utilizing government norms and district rates. This analysis considers factors such as labor, materials, and equipment required for construction activities.

The cost and quantity calculation includes components such as:

Calculation Procedure: The designed structures are further broken down into specific construction portions. Quantities for each portion are calculated based on drawings prepared from design and map layouts based on the type of quantity calculation. Namely, excavations, concrete quantity, formworks and reinforcements are considered through their respective procedure. The calculated quantities are used to determine the cost of civil works by summing the products of quantities and unit costs of the procedures.

Civil Works Estimate: This includes the estimation and calculation of manpower costs, construction materials cost and equipment cost. Manpower costs are determined by subdividing the labor force into categories such as unskilled, semi-skilled, and skilled workers, with unit rates sourced from district rates. Cost for construction materials, such as cement, aggregates and reinforcement steel, are considered from the quantity calculation and the rates are derived based on the material, in terms of sourcing from local market or potential procurement considering transportation costs to the project area as per the requirements.

9.1.1. Calculation for Weir, Undersluice and Intake Construction

Rate analysis for the headworks site was carried out from which it was summarized that:

- The rate for excavation work was Rs. 383.26 per cubic meter.
- The rate for concrete work was Rs. 12638 per cubic meter.
- The rate for formwork work was Rs. 826.21 per square meter.

- The rate for reinforcement work was Rs.121.7 per kilogram.

From quantity calculation based on the design, it was obtained that:

- The excavation required for Undersluice and Weir was 22113.53 cubic meter.
- The excavation required for Intake was 355.32 cubic meter.

Similarly,

- The concrete quantity required for Undersluice and Weir construction was 25192.2 cubic meter.
- The concrete quantity required for Intake construction was 6870.4 cubic meter.

Therefore, the total cost for the construction of the designed weir, Undersluice and Intake for head works was calculated to be NRs. 1121377231.63.

Table 33: Cost for construction of designed weir , undersluice and intake

S.N.	Particular	Total Quantity	Rate (Rs.)	Cost (Rs.)	Remarks
1	Excavation	22468.85	383.26	8611409.53	
2	Concreting	49077.60	12638	620242708.80	
3	Formwork	11628.35	826.21	9607461.11	
4	Reinforcement	3852591.60	121.7	468860397.72	
Total Cost of Gravel Trap				6865414.97	
Total Cost Approach Canal				7189839.49	
Total Cost				1121377231.63	
The total cost is One Arab Twelve Crore Thirteen Lakh Seventy-Seven Thousand Two Hundred Thirty-One Rupees and Sixty-Three Paisa Only.					

9.1.2. Calculation for Headrace Tunnel

Summary of Rate Analysis for Tunnel Cost Estimation:

Based on the detailed rate analysis, the estimated construction cost for tunnel support was evaluated for two rock mass classifications (Class III and Class IV) covering a 1,525-meter-long tunnel. The results are summarized as follows:

- For **Rock Mass Class III** (Poor to Fair, 50% tunnel coverage), with **50mm thick fiber-reinforced shotcrete**, the **total estimated cost** was **Rs. 388.52 million**.
  Major cost components included:
  - Rock excavation and disposal: Rs. 227.69 million
  - Pattern bolts: Rs. 19.25 million
  - Shotcrete: Rs. 3.85 million
  - M25 concrete works: Rs. 37.77 million
  - Concrete lining: Rs. 88.60 million
  - Water control and probe drilling: Rs. 3.77 million

- For **Rock Mass Class IV** (Poor, 50% tunnel coverage), with **150mm thick fiber-reinforced shotcrete** and additional steel rib supports, the **total estimated cost** was **Rs.504.11 million**.
  The cost drivers included:
  - Rock excavation and disposal: Rs. 227.69 million
  - Shotcrete (thicker layer): Rs. 30.83 million
  - Steel rib support: Rs. 86.30 million
  - M25 concrete works: Rs. 39.20 million
  - Concrete lining: Rs. 88.28 million
  - Pattern bolts, formwork, water control, and probe drilling: Combined ~Rs. 31.81 million
- The **total tunnel support cost** combining both classes (Class III and Class IV) was **Rs. 892.63 million**.

9.2. Project Financial Analysis

Financial analysis deals with project cost estimates as well as project revenue estimates. The expenses as well as generated revenue estimated here are from the assumption of values after the completion of project's development. The financial analysis of the project focuses on deciding if project is financially sound and viable or not.

Project Cost, Expected Revenue, Present values, Internal Rate of Return, Payback Period and B/C Ratio have been calculated based on financial projection and are as follows:

9.2.1. Project Cost

The total cost of the project is estimated to be **NRs. 5280155536** (In Words: Five billion, two hundred eighty million, one hundred fifty-five thousand, five hundred thirty-six rupees only). The construction period of the project is estimated to be 3 years with the economic life of project to be 30 years.

9.2.2. Expected Annual Revenue

The energy shall be sold to NEA at the rate NRs. 4.8 per kWh during the wet months and NRs.8.40 per kWh during the dry months. The total revenue is **NRs. 764995739** (In Words: Seven hundred sixty-four million, nine hundred ninety-five thousand, seven hundred thirty-nine rupees only.). Overall efficiency of plant is considered to be 89.3%.

9.2.3. Depreciation

Depreciation is simply an accounting tool, a way of spreading the cost of the equipment over its usable life. In the project, the deprecation of civil components of 3%, hydro mechanical components of 20 %, and electro mechanical components of 20 % and transmission line of 5 % is estimated to determine the opening and closing balance for the end of the year.

9.2.4. Payback Period

The payback period is the time required to recover the initial cost of an investment. It is the number of years it would take to get back the initial investment made for a project. For the project, the payback period for 8 years is estimated under which the revenue generated was **NRs. 940848266** (In Words: Nine hundred forty million, eight hundred forty-eight thousand, two hundred sixty-six rupees only.).

9.2.5. Net Present Value

Net present value (NPV) refers to the difference between the value of cash now and the value of cash at a future date. It is a method used to determine the current value of all future cash flows generated by a project. NPV in project management is used to determine whether the anticipated financial gains of a project will outweigh the present-day investment meaning the project is a worthwhile undertaking. In this project, the net present value for 30 years project period is estimated to be **NRs. 8,146,482,458** . In words it is:

Eight billion, one hundred forty-six million, four hundred eighty-two thousand, four hundred fifty-eight rupees only

A project is determined to be financially viable if it results in positive Net Present Value (NPV). Since the NPV of this project is positive, it is financially attractive and sound.

9.2.6. Internal Rate of Return (IRR)

The internal rate of return rule is a guideline for evaluating whether to proceed with a project or investment. The IRR rule states that if the IRR on a project or investment is greater than the minimum required rate of return, then the project or investment can be pursued. The IRR Rule helps companies decide whether or not to proceed with a project. In the project, the IRR of the project is estimated to be 13% for project life of 30 years.

10. CONCLUSION

The hydropower potential is found to be 150 GWh, generating 23.5 MW of electricity in peak load. The successful completion of this project marks a comprehensive achievement in the field of hydropower engineering. Through detailed hydrological analysis—including flow duration curves, flood frequency assessment, and power potential estimation—this project lays the groundwork for effective water resource utilization. All major hydraulic and structural components such as the Weir, intake, desander, headrace tunnel, surge tank, penstock, and powerhouse have been designed in accordance with established engineering standards, ensuring both functionality and safety.

Structural and stability analyses were performed using advanced tools like AutoCAD, SAP2000, and Phase 2.0, allowing for precise modeling under various loading conditions. The finalized project layout and alignment were selected through rigorous technical evaluation of alternatives. Additionally, the project includes accurate quantity and cost estimation, along with a preliminary economic feasibility study using key indicators such as the Benefit-Cost Ratio (B/C) and Internal Rate of Return (IRR).

The calculation of expected energy generation and potential revenue offers a realistic insight into the project's viability. All findings, analyses, and designs are compiled into a comprehensive final report, ready for academic submission and professional presentation.

Ultimately, this project not only demonstrates a complete hydropower design workflow but also significantly enhances the technical skills of the team in areas such as civil design, structural modeling, and economic evaluation—ensuring preparedness for future challenges in the hydropower sector.

11. REFERENCE

- Khadka, S. S., & Shrestha, M. (2021). Challenges of run-of-river hydropower projects in Himalayan regions: A case study of Seti Khola HEP. *Renewable Energy Journal*, *45*(3), 112125.
- Pandey, B., & Thapa, U. J. (2020). *Hydropower engineering in Nepal: Design and implementation* (2nd ed.). Kathmandu University Press.
- Gurung, R. (2022). *Sediment management strategies for Seti Khola Hydropower Project* [Master's thesis, Kathmandu University].
- Shrestha, S., & Prakash, M. (2021, November). *Seismic risk assessment of Seti Khola HEP*. Proceedings of the International Conference on Hydropower Technology, Kathmandu, Nepal.
- Government of Nepal, Ministry of Energy, Water Resources and Irrigation, Department of Electricity Development. (2018). *Guidelines for study of hydropower projects.*
- Government of Nepal, Ministry of Energy, Water Resources and Irrigation, Department of Electricity Development. (2018). *Powerhouse design guidelines 2018.*
- Bureau of Indian Standards. (1984). IS 11130:1984 – Criteria for structural design of barrages and weirs [WRD 22: River Training and Diversion Works].
- Bureau of Indian Standards. (1986). *IS 11625: Criteria for hydraulic design of penstocks* [WRD 14: Water Conductor Systems].
- Bureau of Indian Standards. (1993). *IS 12800 (Part 1): Guidelines for selection of turbines, preliminary dimensioning and layout of surface hydroelectric powerhouses.* New Delhi, India.
- Baral, S. (2016). *Fundamentals of hydropower engineering*. Kathmandu: National Book Center Pvt. Ltd.
- Garg, S. K. (2017). *Irrigation engineering and hydraulic structures*. New Delhi: Khanna Publishers.