KATHMANDU UNIVERSITY

SCHOOL OF ENGINEERING

DEPARTMENT OF CIVIL ENGINEERING

INTERNSHIP REPORT ON

"KABELI-A HYDROPOWER PROJECT (36.7 MW)"

SUBMITTED BY:

SHRIJA SHRESTHA (027917-20)

SARITA SAPKOTA (028084-20)

DHARAMBIR YADAV (028097-20)

RAJESH YADAV (028098-20)

INTERNAL SUPERVISOR:

ASST.PROF. SANTOSH CHAUDHARY

DEPARTMENT OF CIVIL ENGINEERING

EXTERNAL SUPERVISORS:

ER. BASANT CHAUDHARY

SENIOR SURVEYOR LAKPA SHERPA

ER. GOPAL THAPA

7th, APRIL, 2025

Preface

This report was compiled based on a 5-week internship at site at Kabeli -A Hydroelectric Plant in Taplejung and a 2- week internship at office at Butwal power company in Kathmandu. Our report is informed by the knowledge acquired and literature reviewed during these internship activities.

Our time spent at the site was an invaluable experience, allowing us to witness firsthand the construction activities we have been studying for the past four years during our Bachelor's degree. Working alongside the KAHEP engineering team provided us with insights into the practical aspects of our field. We are grateful for the opportunity to gain firsthand experience working on a project with an engineering team.

Through our internship, we gained a comprehensive understanding of the roles and responsibilities of the client, consultancy, and contractor, as well as their collaborative efforts in project implementation.

We extend our gratitude to both the host organizations and Kathmandu University for facilitating a systematic internship process and providing guidance on the necessary considerations during this period.

Abstract

This report presents our internship experience at Kabeli-A Hydropower Project, undertaken as part of our academic requirements at Kathmandu University through the Department of Civil Engineering. Our team of four had the opportunity to gain hands-on experience in hydropower development, exploring various aspects of project planning, construction, and operation. During our internship, we engaged in site visits, technical discussions, and practical learning, which enhanced our understanding of civil engineering applications in the hydropower sector. We extend our sincere gratitude to Kabeli Energy Limited and Butwal Power company for providing us with this invaluable opportunity to learn and grow in a real-world engineering environment.

Acknowledgement

We would like to express our heartfelt gratitude to Kabeli Energy Limited (KEL) and Butwal Power company (BPC) for providing us with the opportunity to intern at Kabeli-A Hydropower Project. This experience has been instrumental in enhancing our practical knowledge and understanding of hydropower engineering.

We extend our sincere appreciation to Mr. Sanjeeb Baral, Managing Director of API Energy Limited, for his support and encouragement throughout our internship. His insights and leadership have been truly inspiring.

Our deepest thanks go to our supervisor, Mr. Basant Chaudhary sir, Mr. Lakpa Sherpa & Gopal Thapa for his continuous guidance and mentorship during our time at the project site. His expertise and support greatly contributed to our learning experience.

We are also immensely grateful to Er. Abhinash Kayastha for his invaluable guidance and assistance in resolving technical challenges. His patience and willingness to help us navigate complex engineering concepts played a crucial role in our professional development.

Lastly, we would like to thank our faculty at Kathmandu University and the Department of Civil Engineering for facilitating this internship opportunity, as well as our fellow interns for their teamwork and collaboration.

Declaration

This report is the outcome of a purely academic internship performed under our site supervision, consultation with the seniors, Visual observation, and field data observation. We shall not be liable for any damage arising of the use of information form this report.

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1. Introduction

1.1 Background

Kabeli-A Hydropower Project is a run-of-river hydropower project located in the Kabeli River, a tributary of the Tamor River in eastern Nepal. Initially, the project was initiated with financial backing from the World Bank to promote sustainable hydropower development in Nepal. However, due to various financial and operational challenges, the World Bank withdrew its funding, leading to changes in project ownership and management.

Currently, the project is being developed under a joint venture between Butwal Power Company (BPC) and Kabeli Energy Limited (KEL), with a 60-40 stake distribution. The project aims to contribute to Nepal's energy sector by generating clean and renewable electricity, supporting the national grid, and reducing dependency on imported fossil fuels. Designed as a peaking run-of-river project, Kabeli A Hydropower includes critical components such as a diversion weir, intake structure, headrace tunnel, surge shaft, penstock, and a powerhouse.

This report documents our internship experience at Kabeli A Hydropower, where we gained practical insights into the construction, operation, and management of a large-scale hydropower project. Our internship, facilitated by the Department of Civil Engineering at Kathmandu University, provided us with hands-on exposure to real-world engineering challenges, site management, and technical problem-solving. Through this report, we aim to share our learning outcomes and experiences from this invaluable opportunity.

Figure 1 : Components of Hydropower

1.1.1 Objectives

- Gain practical exposure to hydropower construction and operations.
- Understand project management and coordination in hydropower development.
- Enhance problem-solving and technical skills through site experience.
- Familiarize with key hydropower infrastructure components.

1.1.2 Scopes

- To participate in hands-on site activities at Kabeli-A Hydropower for practical exposure.
- To assist in daily construction work under the guidance of supervisors and engineers.
- To observe, document, and report operational activities for structured learning.
- To apply theoretical knowledge to real-world engineering challenges on-site.

1.2 System overview

1.2.1 Project description

The Kabeli-A Hydroelectric Project (KAHEP) is a run-of-river hydropower project located in the Panchthar and Taplejung districts of eastern Nepal. It is situated on the Kabeli River, a major tributary of the Tamor River within the Koshi River Basin. The project area lies between longitudes 87°42′E to 87°55′E and latitudes 27°16′N to 27°22′N, at an elevation ranging from 400 to 1,200 meters above sea level. The project is accessible via roads from Phidim, the district headquarters of Panchthar, and is approximately 800 km east of Kathmandu. The nearest major town is Birtamod, an important trade hub in eastern Nepal.

Geologically, the KAHEP site is located in the Lesser Himalayan Zone, characterized by complex rock formations, including metasedimentary rocks such as phyllites, quartzites, schists, and gneisses. The region has experienced significant tectonic activity, with the presence of major thrust faults like the Main Central Thrust (MCT), making detailed geological and geotechnical investigations critical for project stability. The rock mass in the tunnel alignment varies in strength, requiring appropriate tunnel support measures. Additionally, sediment load in the Kabeli River is relatively high, necessitating an efficient desilting system to prevent turbine erosion.

The design of KAHEP consists of a weir- intake with a settling basin that diverts water from the Kabeli River into a 4.5 km-long headrace tunnel. The water then flows into a surge shaft, which stabilizes pressure fluctuations before being conveyed through the penstock pipe to the underground powerhouse. The powerhouse is equipped with two Francis turbines, each optimized for high efficiency under varying hydraulic conditions. The project operates with a gross head of approximately 116 meters and a design discharge of 37.73 cubic meters per second (m^3/s). The generated power is stepped up to 132 kV for transmission to the national grid.

The KAHEP is being developed by Kabeli Energy Ltd. (KEL), with the involvement of Butwal Power Company (BPC), under a public-private partnership model. The project aims to enhance Nepal’s energy security by adding 37.6 MW of clean, renewable electricity to the national grid. In addition to energy generation, KAHEP is expected to drive local economic development by creating jobs, improving infrastructure, and fostering regional growth. The project incorporates environmental mitigation measures to maintain ecological balance, ensuring continuous downstream water flow and minimizing impacts on aquatic life. By reducing dependence on imported electricity and fossil fuels, KAHEP plays a vital role in Nepal’s sustainable energy development.

1.2.2 Project Location

The Kabeli-A Hydroelectric Project is located on the left bank of the Kabeli River, a snow-fed river originating from the Kanchenjunga Himalaya Range. The project area lies between latitudes 27° 13' 41" N to 27° 17' 32" N and longitudes 87° 40' 55" E to 87° 45' 50" E. The tunnel portal is situated approximately 3.5 km upstream of Kabeli Bazar, while the powerhouse is proposed on the left bank of the Tamor River. The head pond site is at an elevation of 577.45 meters above sea level. The project is accessible via the all-season Mechi Highway, a 228 km road connecting Charali, located 4 km east of Birtamod Bazaar in Jhapa District, to Taplejung. The highway passes through major locations such as Phikkal, Ilam, Ranke, Phidim, and Gopetar before reaching Kabeli Bazar at 202 km north. The Kabeli River, with a steep gradient, has a catchment area of 820 km² at the intake site. The project utilizes a 15 km long loop of the Kabeli River formed with the Tamor River, with a design discharge of 37.23 m³/sec at 40% exceedance flow. A location map is attached for reference.

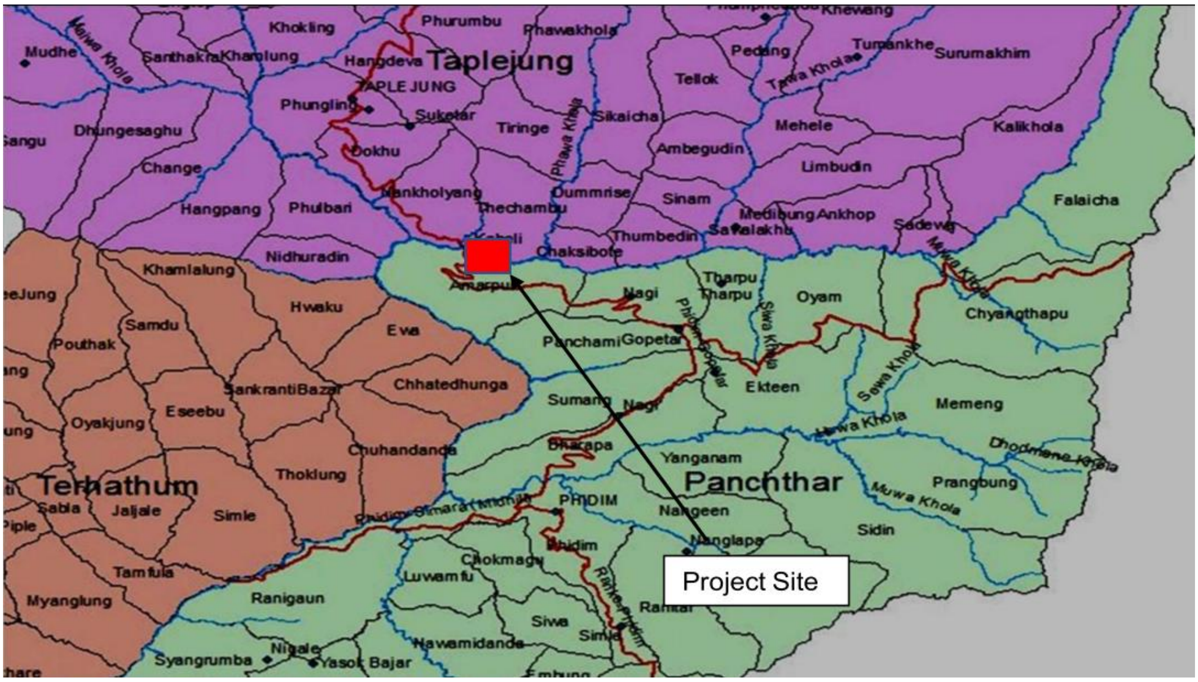

Figure 2 : Project Location

1.2.3 Silent features

The Kabeli-A Hydroelectric Project is a run-of-river (ROR) type with a proposed installed capacity of 37.60 MW and a design discharge of 37.23 m^3/sec. The project taps 35.14 m^3/sec of discharge from the tailrace of the upstream project, along with an additional 2.09 m^3/sec from Phawa Khola by constructing a siphon across the Kabeli River. The combined discharge is collected at the headpond and diverted through the headrace pipe, which connects to the headrace tunnel. From there, water is conveyed through a 1,492 m long headrace canal, a 4,657 m long headrace tunnel, and a 177 m long penstock pipe to reach the powerhouse. The surface powerhouse will be constructed on the left bank of the Tamor River, near Pinase village. The project's salient features are detailed in Table 1

Table 1:Silent Features

$S.N.$	$Items$	$Description\ (Present\ Study)$
1	$Project\ Name$	$Kabeli - A\ Hydroelectric\ Project$
2	$Location$	$Hilihang\ RM\ (Ward\ 1, 2, 4)\ in\ Panchthar\ Pathivara\ and$
2.1	$Project\ Boundaries$	$East$: 87° 45' 50'' E, $West$: 87° 40' 55'' E, $North$: 27° 17' 32'' N, $South$: 27° 13' 41'' N
3	$of\ Development$	$Run - of - River\ (Cascade\ RoR)$
4	$Hydrology\ at\ Intake$	$Catchment\ Area: 713.90\ km^2, 100 - Year\ Flood: 1020\ m^3/s, Mean\ Monthly\ Flow: 47.078\ m^3/s$
5	$Pipe\ from\ Phawa\ Kho$	$Surface, Mild\ Steel, Length: 200\ m, Internal\ Diameter:$ $- 8\ mm$
6	$Tailwater\ Taping\ Can$	$RCC\ Pressurized\ Box\ Canal, Size: 4.2\ m\ x\ 3.6\ m, Length:$
7	$Approach\ Tunnel$	$Inverted\ D - shaped, Concrete\ Lined, Size: 68\ m\ x\ 6.33\ m\ x\ 4\ m$
8	$Additional\ Intake$	$Side\ Intake, Size: 3.0\ m\ x\ 1.5\ m, Gate: Vertical\ Fixed\ Wheel$
9	$Additional\ Intake\ App$	$RCC\ Box\ Culvert, Length: 265\ m, Dimensions: 2.7\ m\ x\ 3.$
10	$Settling\ Basin$	$Simple\ Rectangular, Number\ of\ Chambers: 1, Dimensio$
11	$Headrace\ Canal\ (Conv$	$RCC\ Box\ Culvert, Size: 4.25\ m\ x\ 4.25\ m, Slope: 1{:}700, Le$

12	*Head Pond*	*Rectangular Concrete Lined, Length*: 55 *m*, *Width*: 8.0 *m*
13	*Headrace Canal (Heac*	*RCC Pressurized Box Culvert, Size*: 4.2 *m x* 4.2 *m*, *Lengtl*
14	*Headrace Tunnel*	*Inverted D* *− shaped, Shotcrete Lined, Concrete Lined, Length*: 4657
15	*Surge Shaft*	*Underground and Exposed to Surface, Internal Diamet* − 12.9 *m*, *Height*: 55 *m*
16	*Penstock*	*Material*: *Mild Steel, Length*: 254 *m before Trifurcatio*
17	*Powerhouse*	*Semi Surface, Size*: 58.80 *m x* 19.40 *m x* 28.50 *m*, *Machir*
18	*Tailrace*	*Design Tailwater Level*: 458.0 *masl, Length*: 123 *m*, *Cro* *− section*: 5.5 *m x* 3.0 *m*
19	*Turbine*	*Horizontal Axis Francis, Units*: 3, *Rated Speed*: 600 *rpm*
20	*Generator*	*Salient Pole Rotor Synchronous Generator, Units*: 3, *Vol*
21	*Transformer*	*Three Phase, Oil Immersed, Outdoor Core , Units*: 3, *Rat*
22	*Power and Energy Ou*	*Installed Capacity*: 37.60 *MW*, *Gross Head*: 120.50 *m*, *Es*
23	*Transmission Line*	*Voltage*: 132 *kV*, *Length*: *Loop − in Loop* *− out with Kabeli Corridor Transmission Line*
24	*Access Road*	*To Headworks*: 7.4 *km*, *To Powerhouse*: 15 *km from Me*
25	*Project Cost*	*Total Cost*: *NRs*. 7520 *million*, *Per MW Cost*: *NRs*. 200 *m*
26	*Financial Analysis*	*NPV*: *NRs*. 10,290,000, *B* /*C Ratio*: 1.19, *ROE*: 15.55%, *IRR*: 12.75%
27	*Construction Period*	30 *months*

2. Literature survey and technology

2.1 literature survey

2.1.1 Site geology

The geological conditions of the Kabeli-A Hydroelectric Project area are shaped by the Lesser Himalayan Crystalline to Meta-sedimentary rock sequences, which represent the Taplejung Window. The headworks area is primarily composed of granite, while the settling basin and headrace tunnel consist of a mix of granite, gneisses, schists, phyllites, and quartzites. The surge shaft and powerhouse areas are made up of phyllite, schist, and quartzite. The general orientation of the foliation in the region is 30-40° towards the north. The dam site will be founded on blocky granites, with three distinct joint sets. The uniaxial compressive strength of the granite is between 150 to 230 kg/cm². The dam is likely to be founded in the river channel deposits, which are over 20 meters thick. The rock mass quality at the intake area has been assessed as good (RMR), and the Q value is considered fair. Both the left and right bank slopes at the dam site are relatively stable, though potential instability may arise if excavation is not properly supported. The approach tunnel and settling basins are located in a medium-strong coarse granite zone, where the rock mass quality is classified as fair to good (RMR) and poor to fair (Q value). The headrace tunnel is expected to pass mainly through granite rocks, with the downstream alignment passing through gneiss, schist, quartzite, and phyllite. The tunnel is oblique to the major discontinuities, which is favorable for tunneling works. The surge shaft portal and surge shaft location consist of phyllite with intercalated thin quartzite, with strong to moderately strong bedrock reported. The rock mass quality of the surge shaft has been classified as poor to very poor in terms of RMR and Q value. The powerhouse is located on almost flat alluvial deposits on the right bank of Piple Khola, where the top 20 meters consist of alluvium materials, including pebbles and boulders of gneiss, phyllite, schist, quartzite, and granite, along with grey silty sand and light brown clayey silt. The tailrace canal passes through the active channel of Piple Khola, with moderately compact alluvial materials derived from the Tamor River, including boulders and cobbles of granite, gneiss, schist, phyllite, and quartzite. The construction camps and support facilities are primarily located in alluvial deposits consisting of boulders, pebbles, and cobbles of these rock types, within a matrix of clay and silt/sand. The land use in the area is predominantly agricultural.

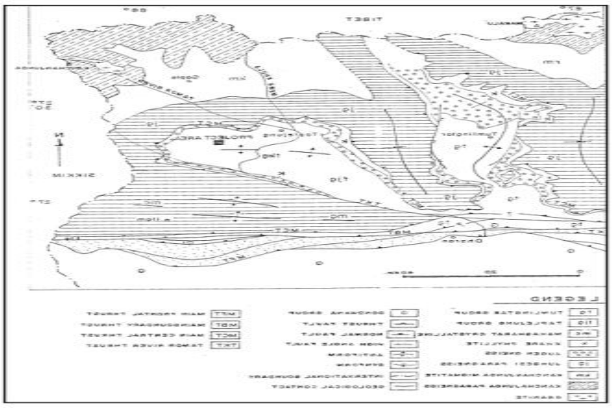

Figure 3 : Regional Geological Map of Eastern Nepal

2.1.2 Hydrology

The hydrological study for the Kabeli-A Hydroelectric Project (KAHEP) is a critical component of the project's design and operational planning. It focuses on the discharge available from two key upstream projects: the Kabeli B1 Hydroelectric Project and the Phawa Khola Small Hydropower Project. These projects provide important data for assessing the flow conditions in the Kabeli River basin and ensuring that the KAHEP is designed with accurate and reliable water flow estimates.

For the Kabeli B1 HEP, the design discharge is 35.14 m^3/s, which forms the basis for evaluating the monthly flow variations that are available to KAHEP. This data is crucial in understanding the volume of water that can be diverted to the KAHEP intake, ensuring its efficient operation. The monthly flow data from Kabeli B1 provides a direct estimate of the water resources available in the region.

The hydrological analysis for the Phawa Khola Small Hydropower Project is more complex, as there is no direct gauging data for this site. To address this, several estimation methods were employed to generate long-term flow data. The Catchment Area method was used as a first approach. This method involved using data from a nearby gauging station to extrapolate the flow at the intake site of Phawa Khola, providing an estimated average annual discharge of 5.71 m^3/s. This estimation was important in filling the data gap for Phawa Khola, ensuring that the hydrological study covered the entire catchment area.

Additional methods were used to cross-validate the flow estimates and refine the analysis. The Regional Regression method, which relied on data from nearby gauging stations, was applied to provide another set of flow estimates. This method takes into account regional hydrological patterns and allows for a more nuanced understanding of the water resources in the area. In addition, the Hydest method, which was developed by Nepal’s Water and Energy Commission, was also applied. This method uses regional data and specific hydrological models to predict monthly flow, and it was crucial for further refining the flow estimates for the catchment area.

Since there is no gauging station available on the Phewa Khola, a hydrologically similar river Hinwa khola was selected to estimate its discharge. This approach is commonly used to determine river discharge in the absence of direct measurement data, relying on regional similarity and comparative analysis.

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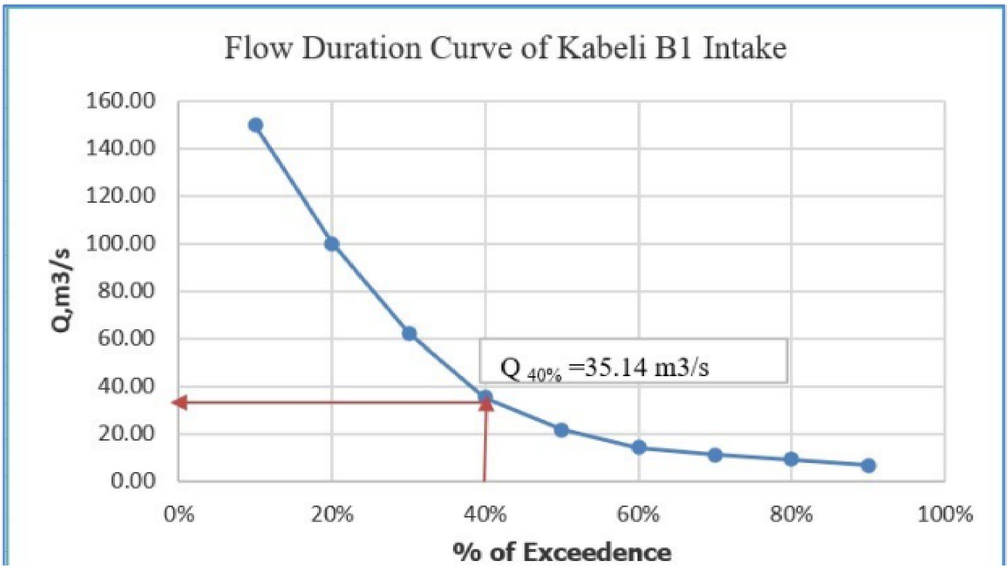


Figure 4 : Flow Duration Curve

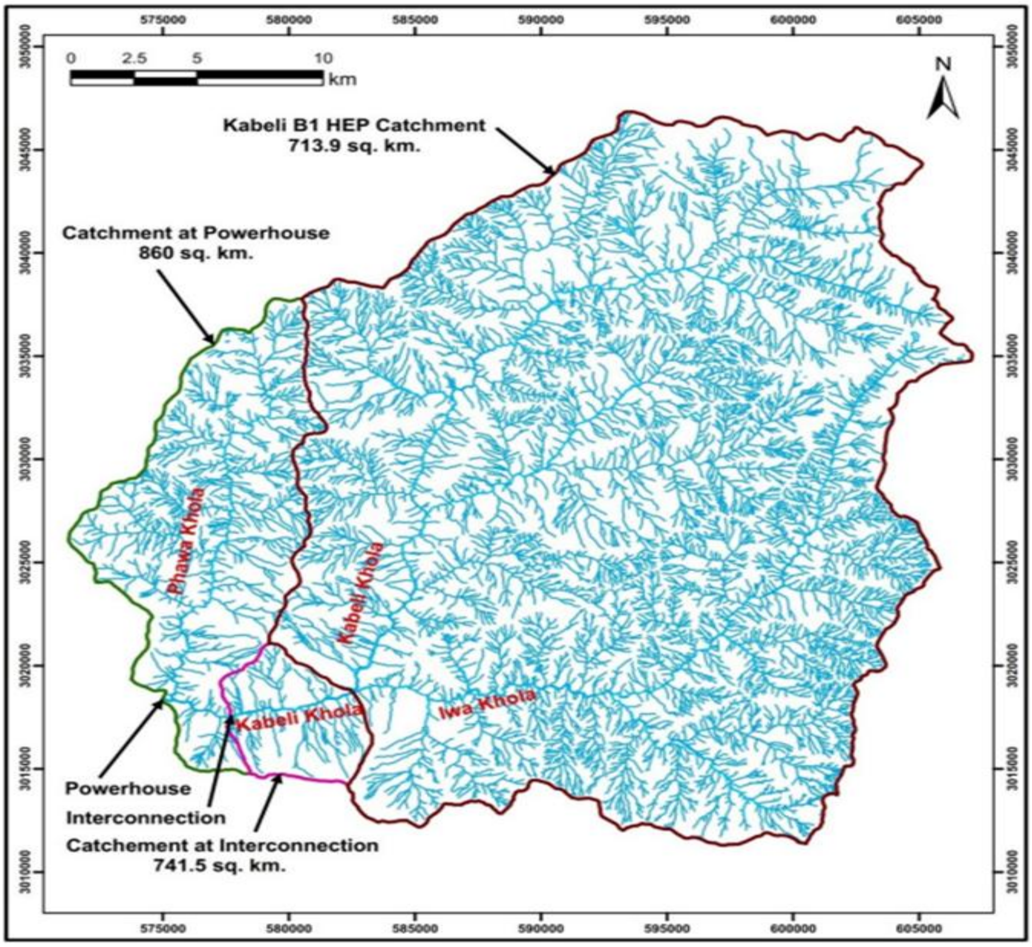


Figure 5 : Catchment Area

Table 2: Generated Mean Monthly Discharge of Phawa Khola

Year	**Jan**	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**	**Oct**	**Nov**	
1974	1.75	1.47	4.71	9.66	27.99	21.91	2.49	-	-	-	-	
1975	1.64	1.22	0.74	0.92	1.53	10.37	13.5	16.36	9.66	3.84	2.32	**Dec**
1976	1.65	1.41	0.99	1.36	20.47	21.28	16	9.57	-	-	-	-
1979	1.12	0.8	1.31	2.73	5.06	5.53	8.65	6.76	2.74	1.35	1.03	-
1984	1.4	1.11	6.77	15.91	11.62	11.35	5.97	3.42	1.83	-	-	-
1985	1.32	1.15	0.99	0.95	2.74	5.42	17.79	17.43	11.71	7.83	4.94	-
1986	1.99	1.37	1.14	1.22	1.9	7.47	10.28	11.27	11.89	8.03	3.93	-
1987	1.48	1.14	1.14	1.31	1.43	4.52	7.72	11.71	14.75	4.48	2.52	2.98
1988	1.52	1.07	1.4	0.82	1.9	1.88	9.57	20.92	12.43	7.7	3.57	2.41
1989	1.88	1.36	1.03	2.24	-	-	-	-	-	-	-	-
1990	1.7	4.88	12.7	13.77	10.82	7.76	3.84	1.95	-	-	-	2.29
1991	1.94	1.21	1.15	1.32	2.98	5.95	11.09	18.33	11.27	4.59	2.52	-
1992	1.28	1.13	0.88	-	-	-	-	-	-	-	-	-
1993	1.71	3.77	7.5	12.87	19.13	13.68	7.46	2.98	2.02	-	-	1.49
1994	1.34	1.09	0.97	1.04	2.06	18.15	14.75	6.04	3.67	3.07	-	-
1995	2.21	1.88	1.66	4.41	16.63	19.31	12.52	8.87	-	-	-	-

1996	1.95	17.26	17.61	13.77	-	-	-	-	-	-	-	-
1997	1.67	3.09	7.08	9.12	17.79	13.41	5.22	2.95	1.9	-	-	-
1998	1.13	0.67	0.5	0.64	12.87	10.73	9.3	6.22	3.33	2.57	-	-
1999	2.24	1.76	1.23	1.27	5.58	9.3	13.95	12.61	10.28	9.66	3.38	-
2000	1.94	1.49	1.22	1.39	8.94	11.8	10.01	7.65	5.59	3.53	2.26	-
2001	1.64	1.4	1.06	1.54	12.61	14.22	12.25	8.68	-	-	-	-
2002	1.39	1.67	9.92	13.86	14.31	8.78	4.87	3.59	2.56	-	-	-
2003	1.76	1.55	2.28	6.12	8.57	9.57	9.12	6.9	5.02	2.86	-	-
2004	1.93	1.21	0.76	1.25	3.11	10.37	9.75	10.91	8.8	4.67	2.55	-
2005	1.43	0.57	0.46	0.66	1.14	3.67	14.13	11.62	11.09	9.39	-	-
2006	2.35	1.68	2.88	10.64	13.86	11.98	13.05	8.74	-	-	-	-
Avg	1.72	1.32	1.17	1.16	2.95	7.07	12.68	14.93	12.23	7.28	3.63	-
												-

Table 3: Generated Mean Monthly Discharge of Hinwa Khola

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1974	2.9	1.96	1.64	2.38	5.27	10.8	33.3	31.3	24.5	17	6.75	2.78
1975	1.83	1.36	0.83	1.03	1.71	11.6	28.9	15.1	18.3	10.8	4.3	2.6
1976	1.85	1.58	1.11	1.52	6.59	16.6	22.9	23.8	17.9	10.7	7.84	6.06
1979	1.25	0.89	1.47	3.05	5.66	6.18	9.67	7.56	3.06	1.51	1.15	-
1984	1.57	1.24	2.07	7.95	7.57	17.8	13	12.7	6.68	3.83	2.05	-
1985	1.48	1.29	1.11	1.06	3.07	6.06	19.9	19.5	13.1	8.76	5.52	3.33
1986	2.23	1.53	1.28	1.37	2.13	8.36	11.5	12.6	13.3	8.98	4.39	2.7
1987	1.66	1.27	1.27	1.46	1.6	5.05	8.63	13.1	16.5	11.5	5.01	2.82
1988	1.7	1.2	1.57	0.92	2.12	2.1	10.7	23.4	13.9	8.61	3.99	2.56
1989	2.1	1.52	2.01	1.15	2.51	-	-	-	-	-	-	-
1990	1.9	1.96	2.17	5.46	12	14.2	15.4	12.1	8.68	4.3	2.18	-
1991	2.17	1.35	1.29	1.48	3.33	6.65	12.4	20.5	12.6	5.13	2.82	1.67
1992	1.43	1.26	0.98	-	-	-	-	-	-	-	-	-
1993	1.91	2.53	2.02	2.69	4.22	8.39	14.4	21.4	15.3	8.34	3.33	2.26
1994	1.5	1.22	1.08	1.16	2.3	13.2	21.4	20.3	16.5	6.75	4.11	3.43
1995	2.47	2.1	1.86	2.1	4.93	14.8	18.6	21.6	14	9.92	6.46	3.86
1996	2.94	2.17	2.18	1.69	8.07	12.2	19.3	19.7	15.4	12	6.32	3.95
1997	2.82	2.55	1.87	2.04	3.46	7.92	10.2	19.9	15	5.84	3.3	2.13
1998	1.26	0.75	0.56	0.72	11.1	15.7	14.4	12	10.4	6.96	3.72	2.87
1999	2.5	1.97	1.38	1.42	6.24	10.4	15.6	14.1	11.5	10.8	7.65	3.78
2000	2.17	1.67	1.36	1.56	9.62	10	13.2	11.2	8.56	6.25	3.95	2.53
2001	1.83	1.57	1.18	1.72	14.5	19.7	14.1	15.9	13.7	9.71	6.25	4.37
2002	3.23	2.14	1.56	1.87	7.89	11.1	15.5	16	9.82	5.45	4.02	2.86
2003	1.97	1.73	2.55	2.19	6.84	6.84	9.59	10.7	10.2	7.72	5.61	3.2
2004	2.16	1.35	0.85	1.4	3.48	13.7	11.6	10.9	12.2	9.84	5.22	2.85
2005	1.6	0.64	0.52	0.74	1.28	4.1	15.8	13	12.4	10.5	7.87	5.26

2006	2.63	1.88	2.2	2.37	3.22	11.9	15.5	13.4	14.6	9.77	7.87	5.4
Avg	2.1	1.6	1.42	1.61	5.07	10.1	15.82	16.7	13.68	8.79	5.04	3.15

Table 4:Combined Discharge for KAHEP

Month	**Available Total Turbine Discharge (KAHEP m³/s)**	**Available Total Turbine Discharge (PKSHPP m³/s)**	**Combined Discharge (m³/s)**
Baisakh	14.86	1.29	16.15
Jestha	23.22	2.09	25.31
Asad	35.14	2.09	37.23
Shrawan	35.14	2.09	37.23
Bhadra	35.14	2.09	37.23
Ashoj	35.14	2.09	37.23
Kartik	34.12	2.09	36.12
Mangsir	19.26	2.09	21.35
Paush	13.4	2.09	15.46
Magh	10.27	1.64	11.91
Falgun	9.17	1.27	10.44
Chaitra	9.49	0.89	10.38

2.2 Topographic Survey and Mapping

This section of the report details the methodology, outputs, and data collected for the topographical survey of the Kabeli-A Hydroelectric Project. The survey began with a reconnaissance study using existing topographical maps, followed by a more detailed survey conducted in Magh 2078. The main objective was to define the available head more accurately and prepare topographical maps of the project area to assist in the placement of key components, including the Headworks/Head Pond, Headrace pipe/canal, Headrace tunnel, Surge Tank, Powerhouse, and Tailrace. The survey covered the entire area from headwork to tailrace, identifying all physical and manmade features. Permanent control points were marked on stable boulders with enamel paint, and a closed traverse survey was conducted around the proposed sites for headworks, waterways, and the powerhouse. The resulting topographical maps of the headwork's site, tunnel alignment, powerhouse site, and tailrace were prepared at appropriate scales, with all surveyed data plotted in Auto-CAD drawings.

2.2.1 Methodology

Detailed topographical survey of the project area was carried out by the survey team led by a senior surveyor. Standard total stations with least count of 1mm were used to maintain the required accuracy. SOKKIA and PENTAX total stations were used to carry out the survey work. Required data to determine the locations coordinates and levels were taken directly in the field.

Survey works as per the scope of works includes the following:

- Desk study
- Reconnaissance survey
- Monumentation of Control Points
- Control Traversing
- Horizontal and Vertical Control
- Accuracy
- Data Processing

Desk Study

Before the field survey, a desk study was conducted using topographical maps (Scale 1:50,000) from the Survey Department of Nepal. Information related to the project area for survey activities was gathered, and available plans, profiles, and location maps from the identification study were reviewed.

Reconnaissance Survey

Following the desk study, a team of multi-disciplinary experts was deployed for field verification. The primary task was to identify ground control stations and determine areas requiring detailed topographical mapping.

Monumentation of Control Points

The survey teams established sufficient survey stations around the project components. Major control points were marked on boulders and rocks with enamel paint. Some points were further marked with iron pins or cross marks chiseled on the boulders for visibility.

Control Traversing

A basic ground control survey was conducted by performing a closed traverse from the headworks site to the powerhouse. Additional ground control stations were set up along the survey route, with all required data for determining coordinates and elevations collected in the field.

Horizontal and Vertical Controls

The horizontal control survey utilized SOKKIA and PENTAX total stations with precision down to 1mm. Surveys included the following:

- Mean angle and distance computations.
- Angular closure checks for the traverse.
- Azimuth checks between traverse points.
- Adjustment of angular misclosures.
- Calculation of ΔX and ΔY for plan metric closure.

Vertical control involved observing three sets of elevation readings and using the triangulation leveling method to determine elevations accurately. These control points, spanning from the headworks to the powerhouse site, were critical for precise survey results.

Accuracy

High-precision instruments with a least count of 1mm were employed throughout the survey, ensuring minimal closing errors. All closing errors were adjusted according to standard survey protocols, meeting acceptable accuracy limits.

Data Processing

Survey data was processed both in the field and in Kathmandu. The data, including coordinates and elevations for each station, was computed using UTM coordinates. After thorough verification, MS Excel and SW-DTM software were used to prepare contour maps, plans, profiles, and cross-sections. The final topographic maps were produced in AutoCAD format.

Detail Topographic Survey

The topographical survey covered areas from the headworks to the powerhouse site, with ground control points established both permanently and temporarily. Features like riverbanks, flood levels, landslides, and cultivated lands were surveyed using spot surveying methods. Inaccessible areas were surveyed from multiple known points to ensure accuracy in both horizontal and vertical angles

Mapping

Detailed topographical mapping was carried out for various project components, including the headworks, headrace tunnel, penstock alignment, surge shaft, powerhouse, and tailrace area. The survey results, along with coordinates and elevations for ground control stations, are provided in the appendices and shown in the general contour map.

River Cross Section

Cross-sections of the Kabeli River were surveyed at intervals of 50 meters to calculate the rating curves for the headworks and powerhouse/tailrace sites.

2.2.2 Hydrological Investigations

Collection of Available Meteorological and Hydrological Data, Meteorological data from stations near the Intake area were collected. The relevant stations and their annual precipitation records are listed in Table-5.

Table 5: Meteorological Station

Index No.	Station Name	Elevation (masl)	Annual Precipitation (mm)
1301	Num	1497	4098
1303	Chainpur	1329	1394
1317	Chepuwa	2590	2631
1401	Olangchunggola	3119	1663
1403	Lungthung	1780	2232
1404	Tapethok	1383	2561
1405	Taplejung	1732	1897
1413	Kamachin	4242	1352
1414	Nup	4000	972
1419	Phidim	1205	1369

The average precipitation in the catchment area was calculated to be 2083 mm using the Thiessen Polygon method.

2.2.3 Climatological Records

The KAHEP area lies in the subtropical climate zone, experiencing hot and humid weather from June to September, and cold climate from November to January. The mean monthly temperature varies from 0°C in January to 25°C in June. Extreme temperatures can exceed 30°C in summer and fall below 0°C in winter. The average humidity in Taplejung ranges from 36% to 77% annually.

Table 6: Final Coordinates of Control Points

BM Station	Easting (m)	Northing (m)	Elevation (m)
5002	573428.319	3018220.879	588.294
6001	573372.25	3018159.441	623.743
6002	573275.554	3018051.754	679.991
6003	572493.715	3017579.626	860.777
6004	572328.348	3017344.908	951.107
6005	571920.642	3017048.244	1081.604
6007	571392.106	3016473.322	1300.101
6008	571321.374	3015773.328	1107.095
6009	571411.6	3014969.432	871.095
6010	572444.606	3013660.742	597.235
6011	572634.66	3013465.148	504.719
6012	573119.43	3013455.082	624.734

2.2.5 Seismicity

The Kabeli-A Hydroelectric Project area lies within a seismically active zone due to its proximity to key tectonic features of the Himalayas. The region experiences seismic activity primarily due to the convergence of the Indian and Eurasian tectonic plates along the Main Himalayan Thrust (MHT). While the active fault systems near the project site have not been extensively reported, the general seismicity of the area is influenced by the surrounding structural features such as the Main Central Thrust (MCT), Main Boundary Thrust (MBT), and Himalayan Frontal Fault (HFF).

Recent seismic data indicates that moderate earthquakes (magnitude 6 to 7) frequently occur along the MHT, particularly between the foothills and the Higher Himalayas, which includes the area near the Kabeli A site. Although large earthquakes (M>7.5) have not occurred directly in this vicinity, the region remains susceptible to seismic events due to its proximity to active fault zones.

Seismic hazard assessments indicate that the project area faces a potential risk of significant ground motion over the next 50 years. This is based on a seismic hazard map that shows a 10% probability of exceeding a certain peak horizontal acceleration at bedrock, underscoring the need for seismic considerations in the design and construction of the hydroelectric project

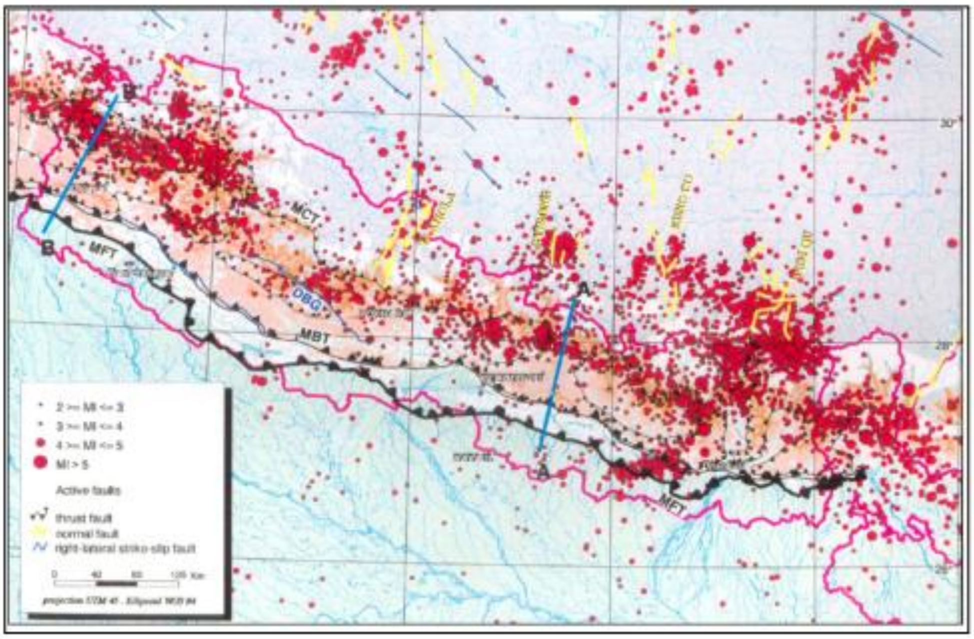


Figure 6 : Earthquake Epicentre Map of Nepal

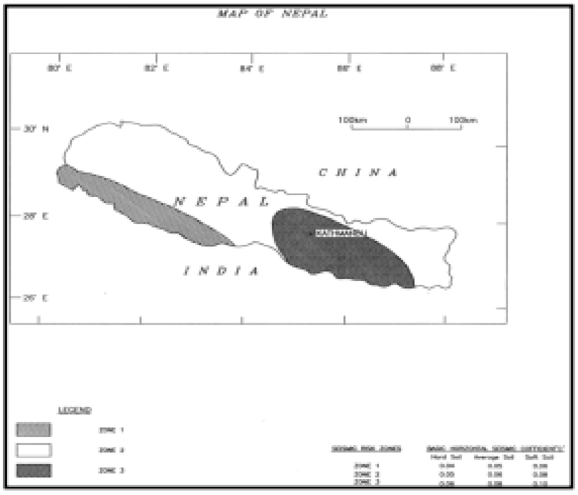


Figure 8:Seismic Risk

Figure 7: Seismic Hazard Map of Nepal

Table 7: Recurrence of Earthquake in Nepal

AVERAGE FREQUENCY OF EARTHQUAKES (DURING THE PERIOD 1994-2000)	M < 2 *	2 < M < 3	3 < M < 4	4 < M < 5	5 < M < 6	6 < M < 7
Nepal	4 per day	3 per day	1 per day	5 per month	6 per year	1 in 6 years
West Nepal	1 or 2 per day	1 per day	2 or 3 per week	2 per month	2 per year	1 in 6 years
East Nepal	2 or 3 per day	2 per day	4 or 5 per week	3 per month	4 per year	0 in 6 years

M : Local magnitude (an earthquake of magnitude greater than 4 or 5 can be felt).

2.3 Interconnected Chamber

Interconnection chamber: It's the most crucial and unique intake structure portion of Kabeli-A HEP (37.6 MW). As its name suggests, it’s a chamber where the water from two different sources is separated via a front intake gate located between additional intake gate walls and tailrace taping canal of Kabeli B1 cascade.

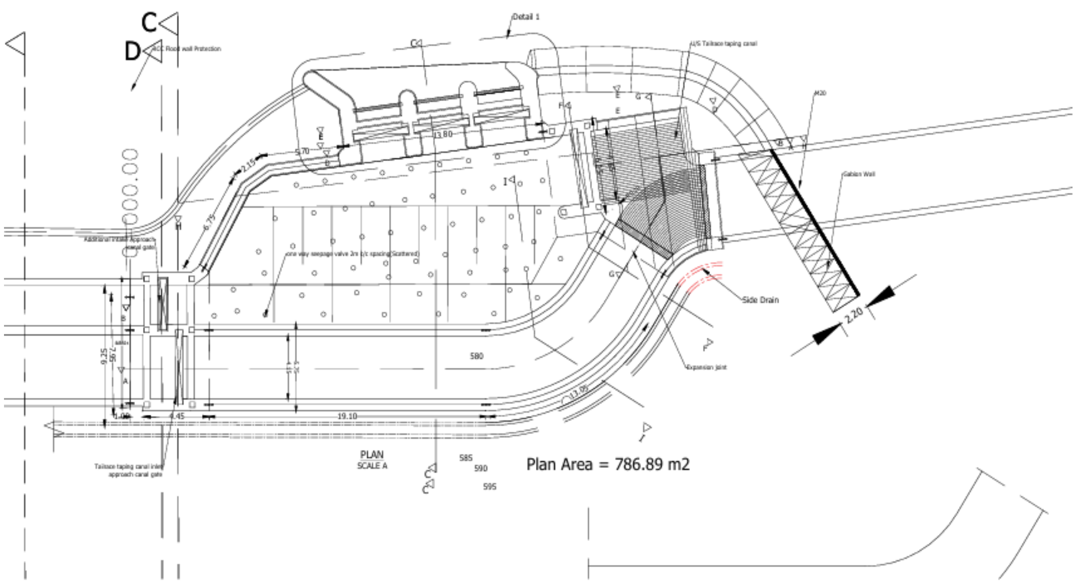


Figure 9 : Plan of Interconnected Chamber

At the hill side of this structure lies a taping canal which conveys the water coming right out of the tailrace of kabeli b1 cascade. And there are 3 gates at the riverside known as additional intake gates. The trash racks present in this place tends to block the small boulders and many other objects carried by the kabeli river. In a long run the chamber portion will be cleared with the help of long neck crane. The water flowing from kabeli enters the chamber portion through additional intake gates and is conveyed through small canal upto settling Basin.

By opening the Additional gate and frontal gate, we can directly discharge the water from the Kabeli-B1 cascade tailrace into the river. This process is carried out when the tunnel and headrace canal near the interconnected gate are undergoing maintenance. In the Kabeli-A Hydropower Project, the gate house serves two main functions: regulating the flow of water through gates and managing human shuttering operations. The system consists of three gate houses:

Additional Control Intake Gate House

This gate house controls three gates. It is operated when the Kabeli-B1 cascade plant is not in operation, and water needs to be drawn directly from the Kabeli River. It is also used during maintenance of the headrace canal and tunnel by redirecting the tailrace water back to the Kabeli River.

Approach Canal Gate House

This gate house controls two gates. It allows water to flow into the headrace canal for power generation. The flow comes from two sections: one larger section that carries tailrace water from Kabeli-B1, and a smaller section that carry the water directly drawn from the Kabeli River.

Front Intake Gate House

This gate house controls one gate. It regulates the flow of water coming from the tailrace of Kabeli-B1. Normally, the water passes through the larger section, which does not have a settling basin. However, during maintenance of any part of this larger section, the water is diverted through the smaller section, which passes through the settling basin to ensure a smooth operation of the project.

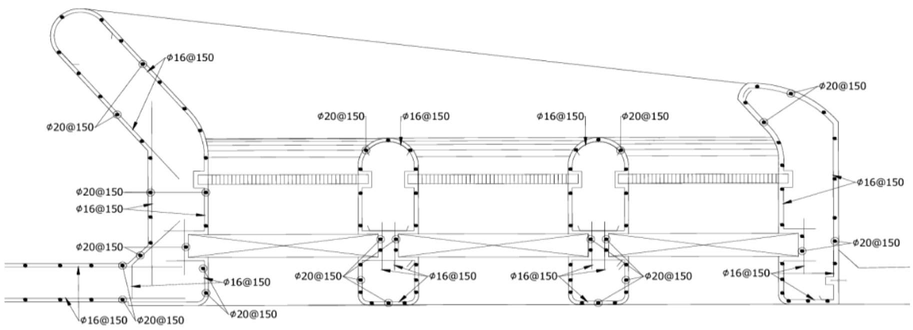


Figure 10 : Plan 1-1 of Additional Intake

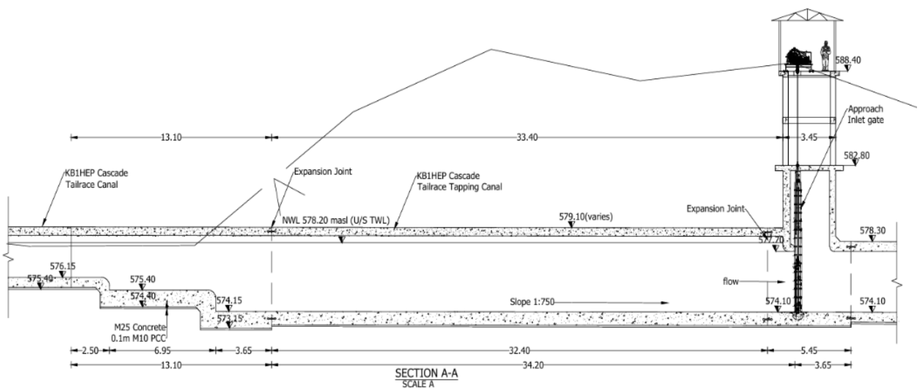


Figure 11 : Section A-A of Interconnected Chamber

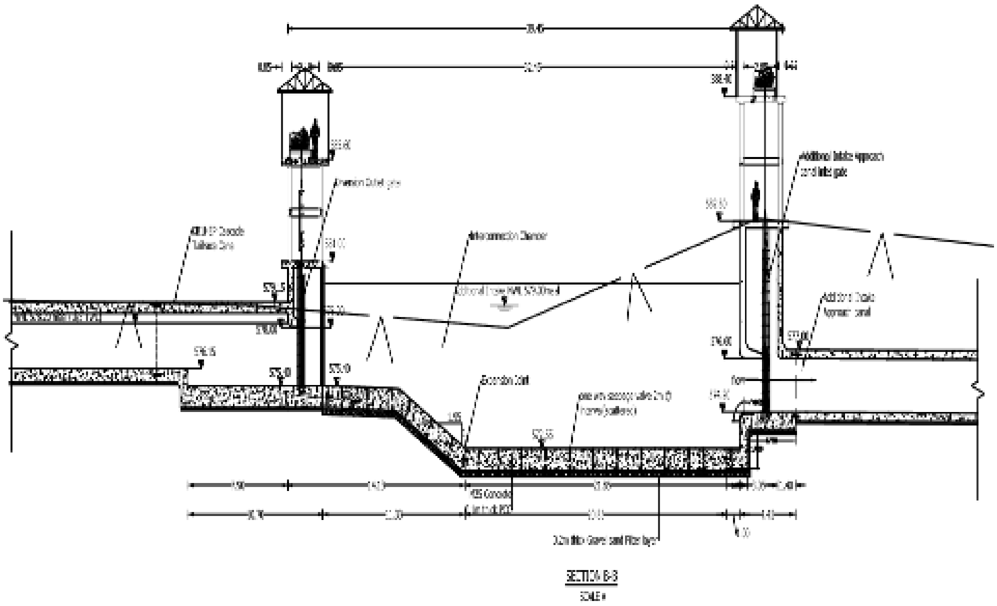


Figure 12: Section B-B of Interconnected Chamber

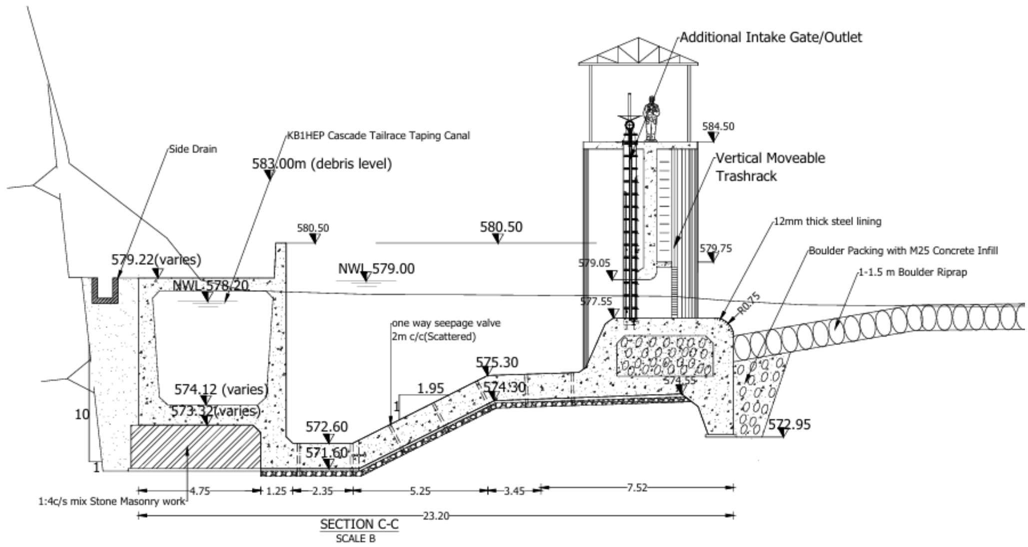


Figure 13 : Section C-C of Interconnected Chamber

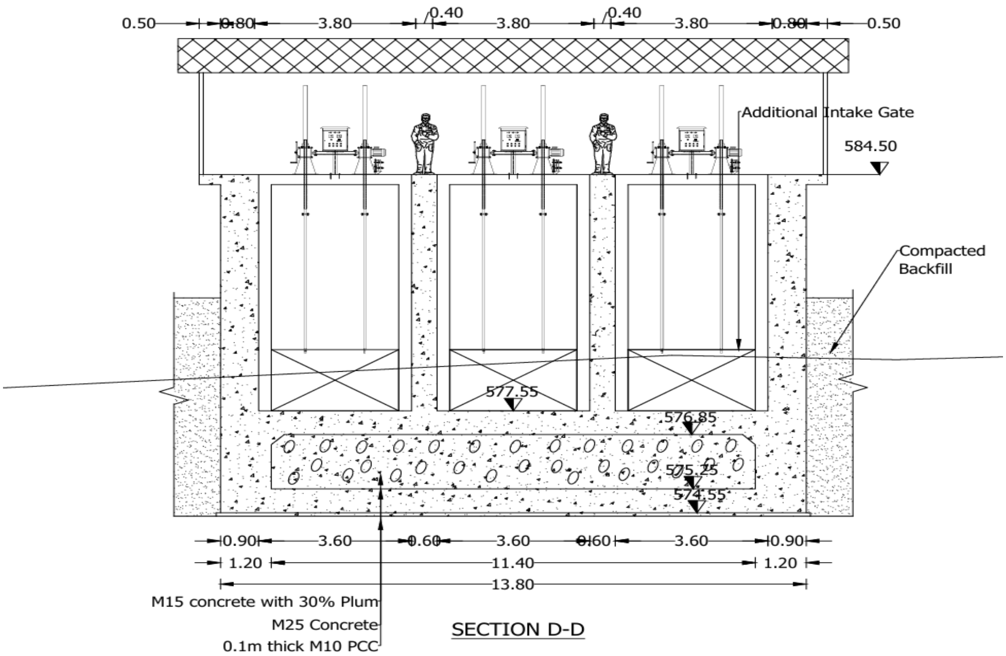


Figure 14 : Section D-D of Interconnected Chamber

2.4 Protection wall

The protection work around this structure is carried out using a cut-off wall to ensure safety against the river meandering process and the water pressure of the river having a width of 3.0 m and height of 6.8 m throughout the interconnected gate portion.

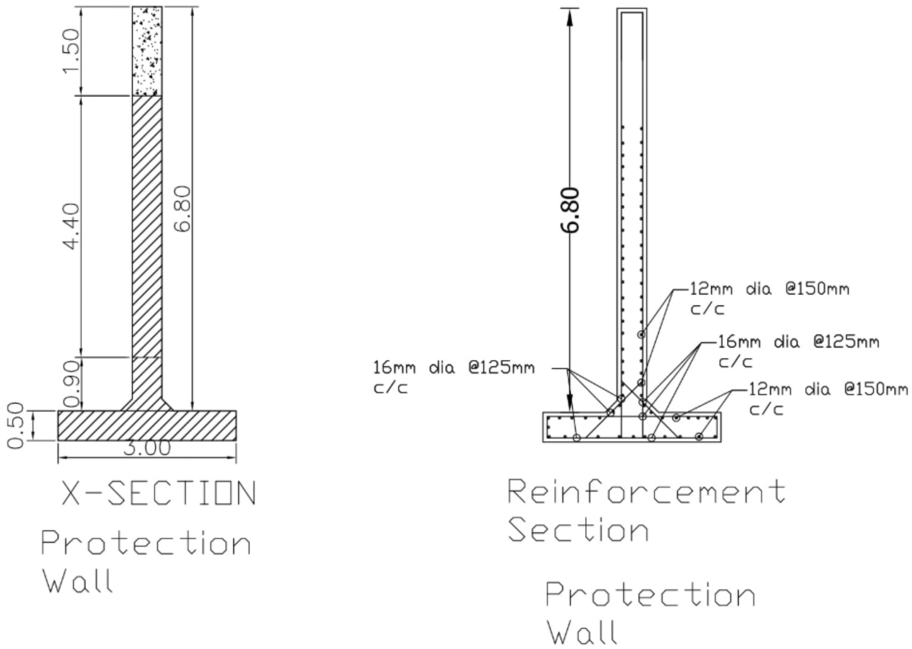


Figure 15 : Section of Protection wall

2.5 Desander

The Kabeli-A Hydropower Project incorporates a Hoover-type desander basin to effectively remove sediments from the water before it enters the intake system, ensuring smooth operation and reducing turbine wear. The project features one settling basin with a hopper, having a uniform section length of 55 meters and a total length of 80 meters (including the transition section), with a width of 13.5 meters and height of 10 meters. A mechanical flushing system is installed to periodically remove settled sediments. The approach canal transporting water from the intake to the settling basin consists of two channels, one measuring 270 meters in length with a cross-sectional area of 4.25 m × 3.6 m, and the other with a cross-sectional area of 2.7 m × 2.4 m. As water flows through the settling basin, heavier sediments such as sand, silt, and gravel settle at the bottom, while cleaner water continues toward the headrace pipe and tunnel for power generation. The mechanical flushing system ensures efficient removal of deposited sediments, preventing blockages and maintaining system efficiency. This desander basin plays a crucial role in protecting turbines, reducing maintenance costs, and ensuring the long-term operational efficiency of the hydropower plant.

The Kabeli-A Hydropower Project is a run-of-river (ROR) type project with a proposed installed capacity of 37.60 MW and a design discharge of 37.23 m^3/s. The project taps into the tailrace discharge from the upstream hydropower project, utilizing 35.14 m^3/s of water. Additionally, an extra 2.09 m^3/s of discharge is sourced from Phawa Khola by constructing a syphon across the Kabeli River. The combined discharge from both sources is collected at the head-pond, from where it is diverted into the headrace pipe and then connected to the headrace tunnel for power generation.

2.5.1 Working Principle

Water enters the settling basin at a controlled velocity, allowing heavier particles (sand, silt, and gravel) to settle at the bottom.

The mechanical flushing system efficiently removes settled sediments through a controlled discharge system.

The cleaner water is then directed towards the headrace pipe and tunnel for power generation.

Figure 16 : Desander Basin

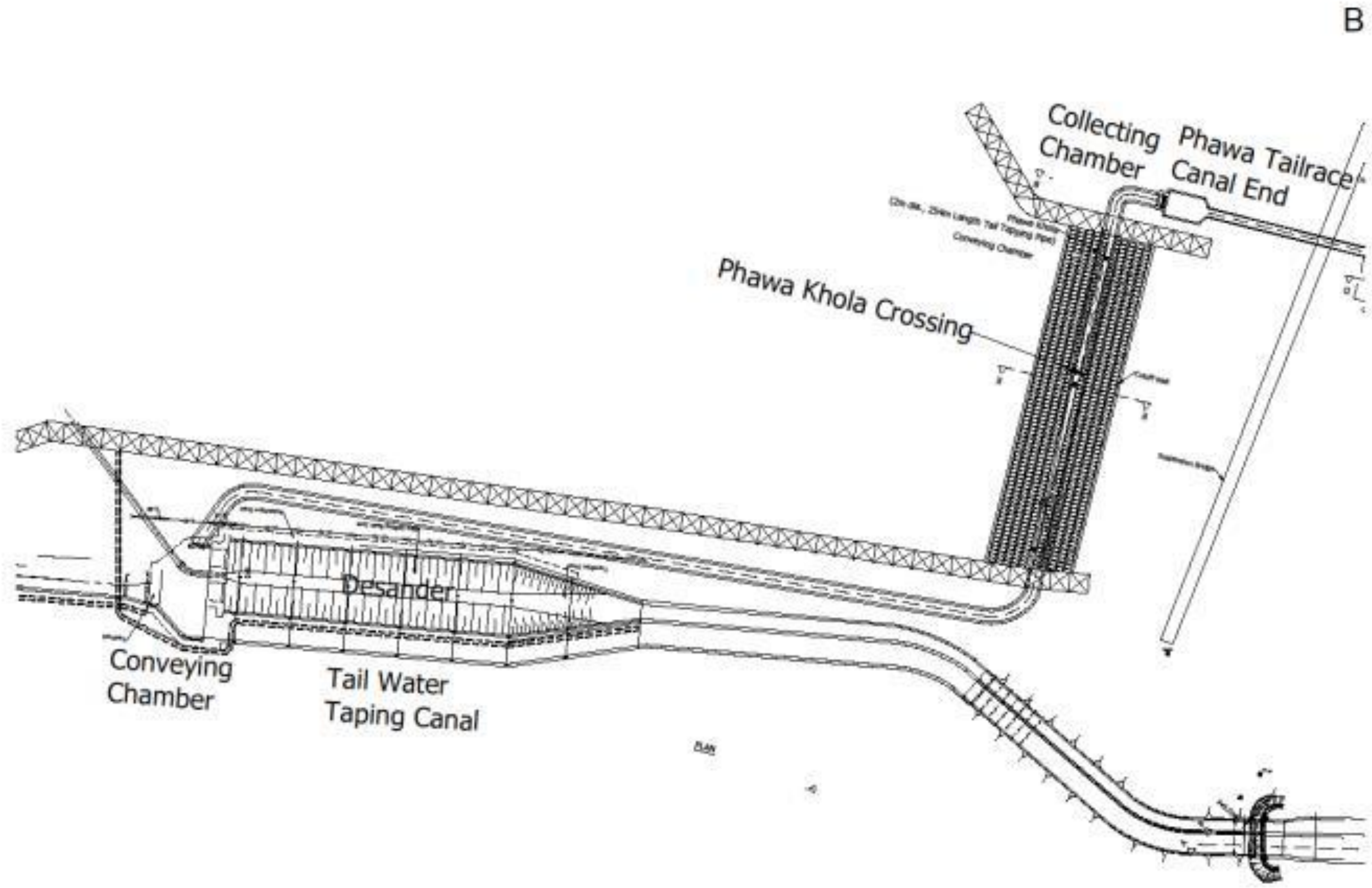


Figure 17 : Taping of Tailrace water

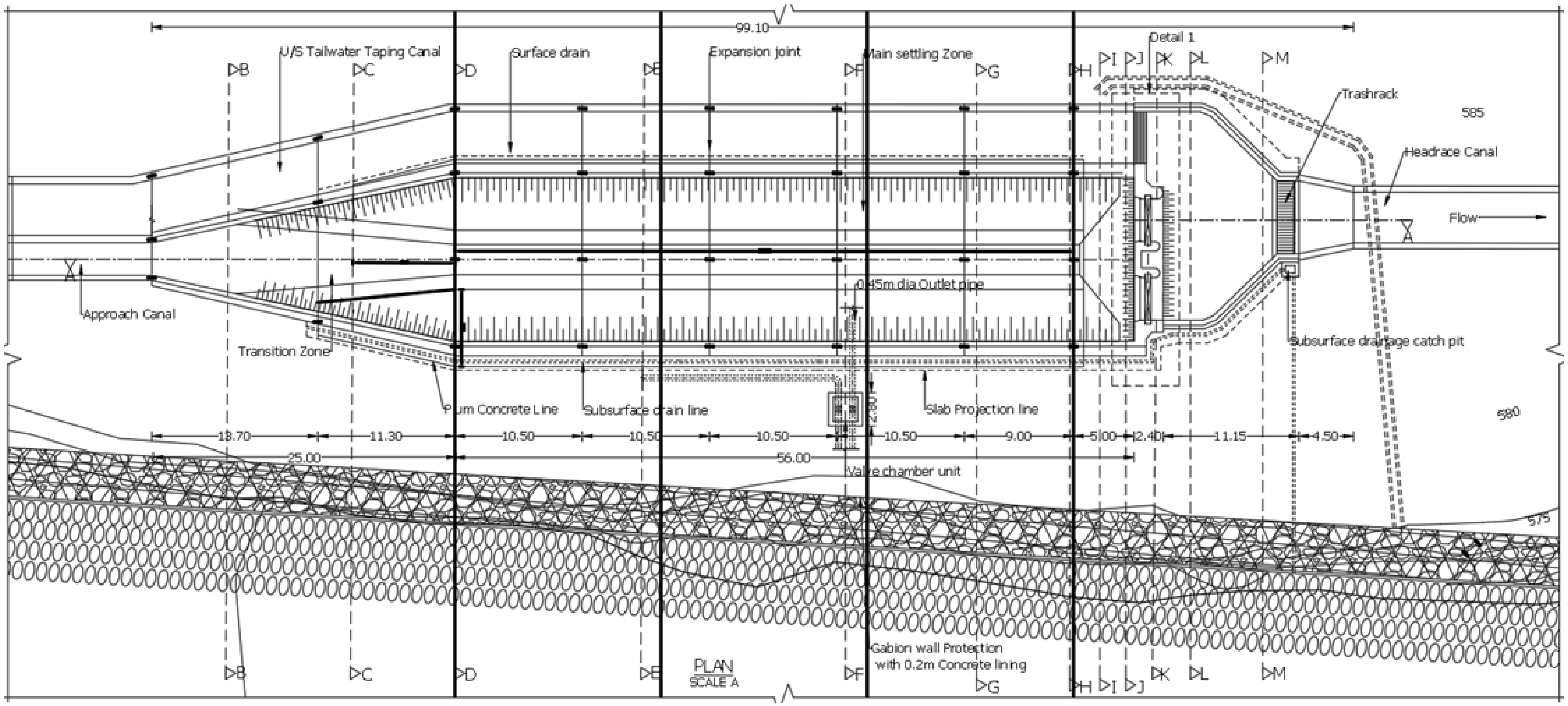


Figure 18:Tailrace water Taping

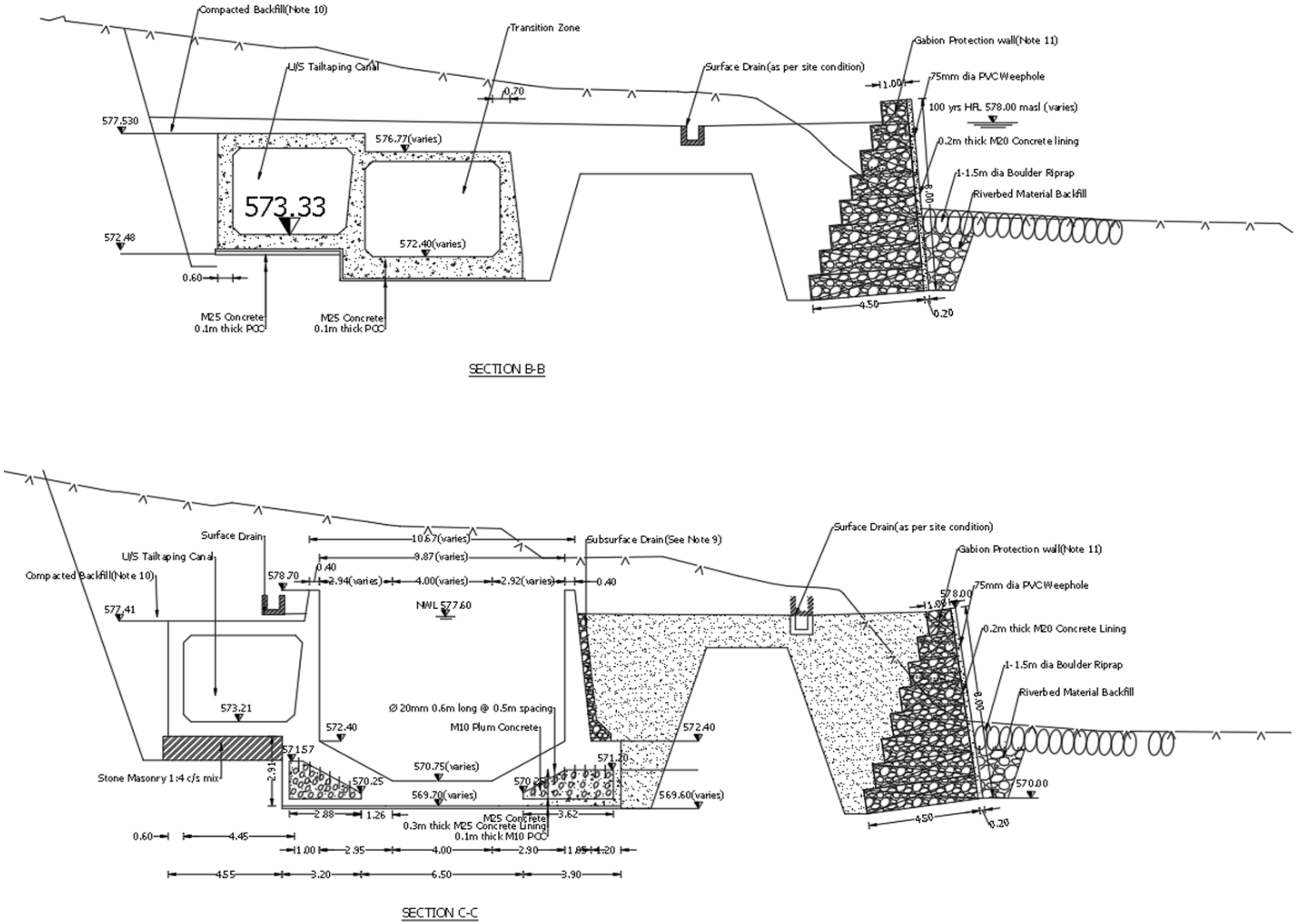


Figure 19 : Section B-B and C-C of Desander Basin

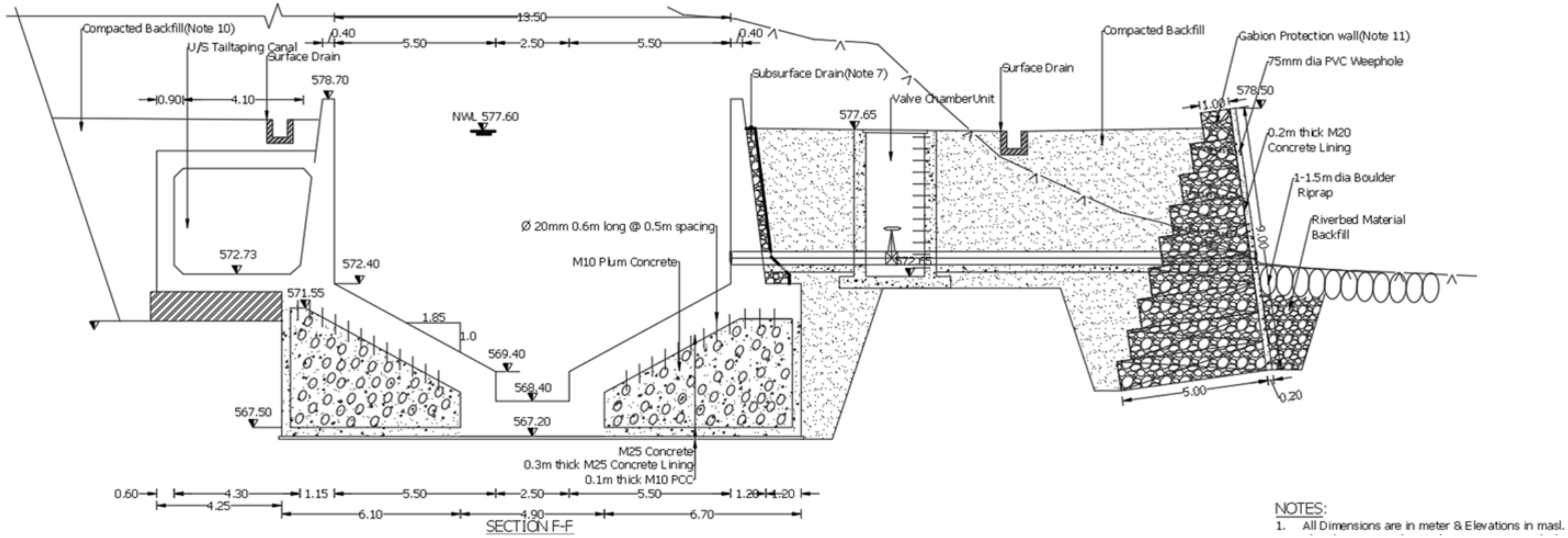


Figure 20 : Section F-F of Desander Basin

2.6 Approach Canal and Headrace Canal

2.6.1 Approach Canal (Canal after Interconnection Chamber)

The function of the approach canal is to convey water to the settling basin and the collection chamber. It regulates flow, and ensures a steady water supply for power generation.

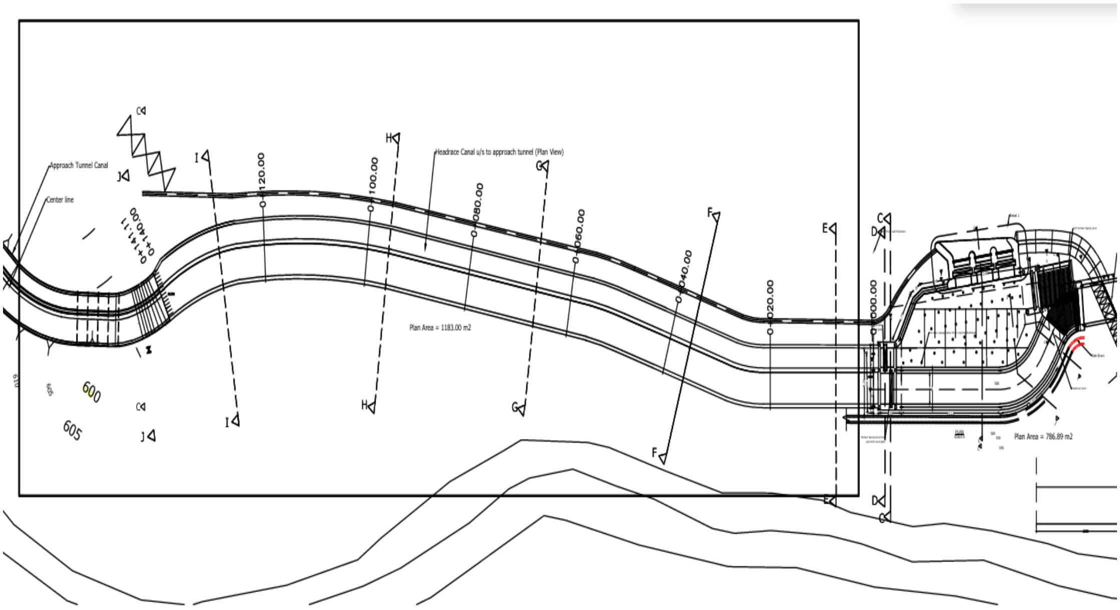


Figure 21: Plan of approach canal

The approach canal consists of two sections separated by a dividing wall. The larger section is designed to carry water from the tailrace of the Kabeli B1 Cascade Hydropower, while the smaller section is intended to draw water from the Kabeli River in case the cascade hydropower plant is not in operation.

The total width of the approach canal is 8.45 m, with a height of 4.6 m. The shear wall of the larger section has a thickness of 50 cm, whereas the shear wall of the smaller section has a thickness of 40 cm. The dividing wall between the two sections has a thickness of 60 cm. Additionally, the top slab of the larger section is 50 cm thick, while the top slab of the smaller section is 40 cm thick.

The canal will be completely buried by backfilling and provided with a side drain along its entire length. The opening of the larger section measures 4.25 m × 3.6 m, with a hunch portion of 0.4 m × 0.4 m, while the opening of the smaller section measures 3.7 m × 2.4 m, with a hunch portion of 0.3 m × 0.3 m.

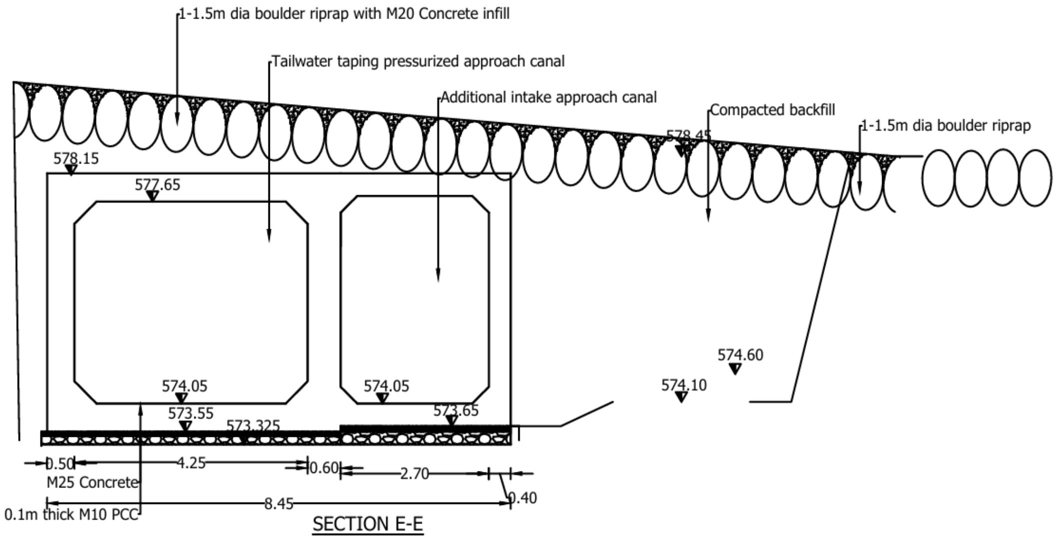


Figure 22 : Section of Approach canal

Figure 23 : Approach Canal

2.6.2 Headrace Canal (Canal after Settling Basin)

Water from the tailrace of Kabeli B1 Cascade Hydropower and the water drawn from the Kabeli River, one at a time use the common headrace canal to convey water to the headpond. If the Cascade Hydropower is not in operation, the water from the Kabeli River still passes through the settling basin. After this stage, both water sources merge into a common headrace canal once sediment has settled in the basin. However, the tailrace water flows directly without passing through the settling basin. The headrace canal is a symmetrical canal section of 5.2 m x 5.2 m outer to outer dimension. All the walls and slabs are of 50 cm thick and the rebars used are of 16mm main bars and 12mm distribution bars. A canal is of 20 m in length and it extends up to 1079.5 m. to reach headpond.

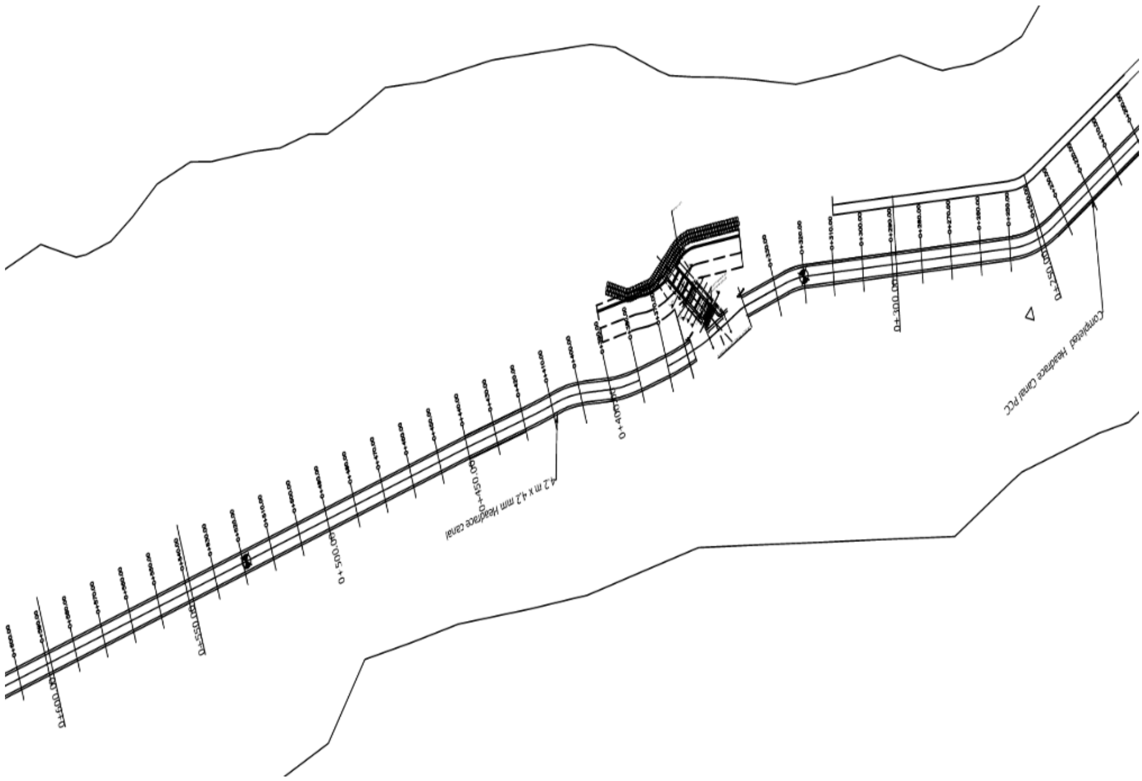


Figure 24 : Plan of approach canal after Settling Basin

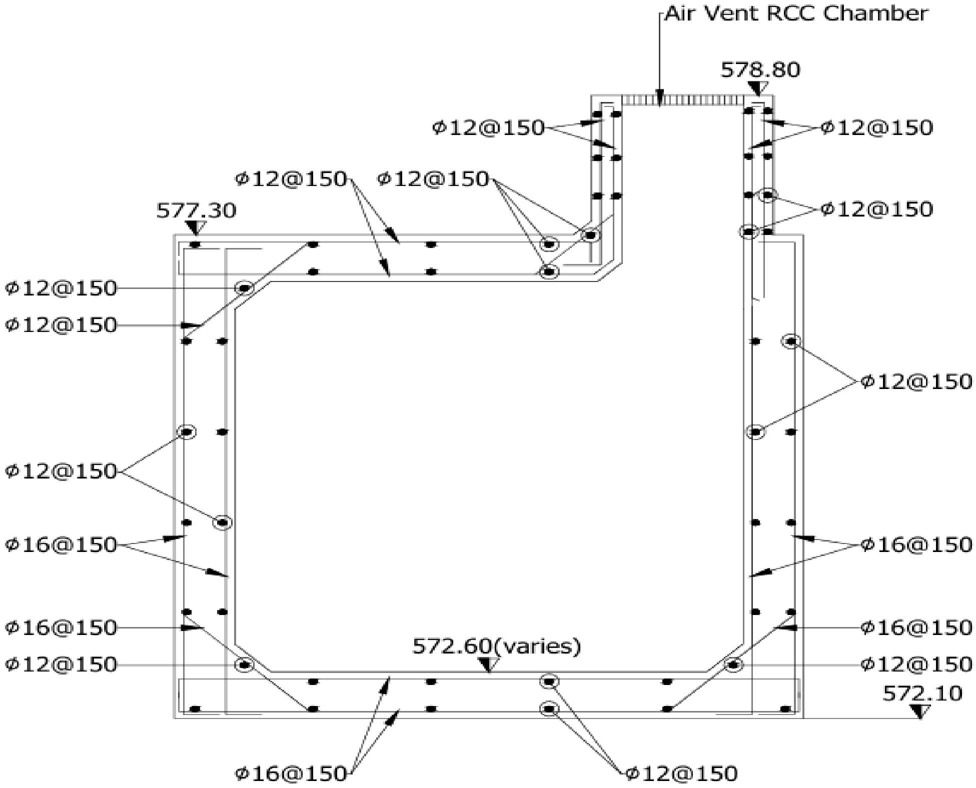


Figure 25 : Section of Approach Canal with Air Vent

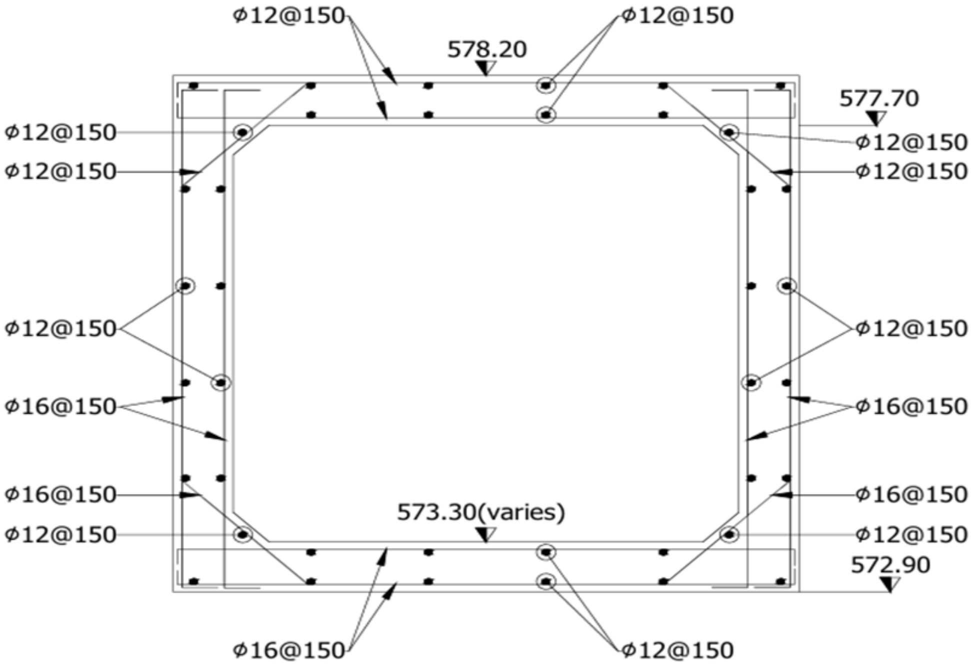


Figure 26 : Section B-B of Approach Canal

2.7 By pass Gate

The bypass gate in Kabeli Hydropower is designed primarily for easy maintenance. To enhance its stability, it is covered by cutoff walls on both the left and right sections. This measure was implemented following a recent flood event that caused significant damage at this location due to the direct impact of the river’s trajectory. The damage posed a high risk of canal collapse behind the bypass gate.

The main purpose of this design is to ensure the safety of the canal while facilitating maintenance of the siphon, canal, and forebay. Additionally, the absence of a breaking point in the long headrace canal, which has a length of 1080 meters, increases the risk of structural failure. Therefore, the cutoff walls and bypass gate serve as crucial protective measures.

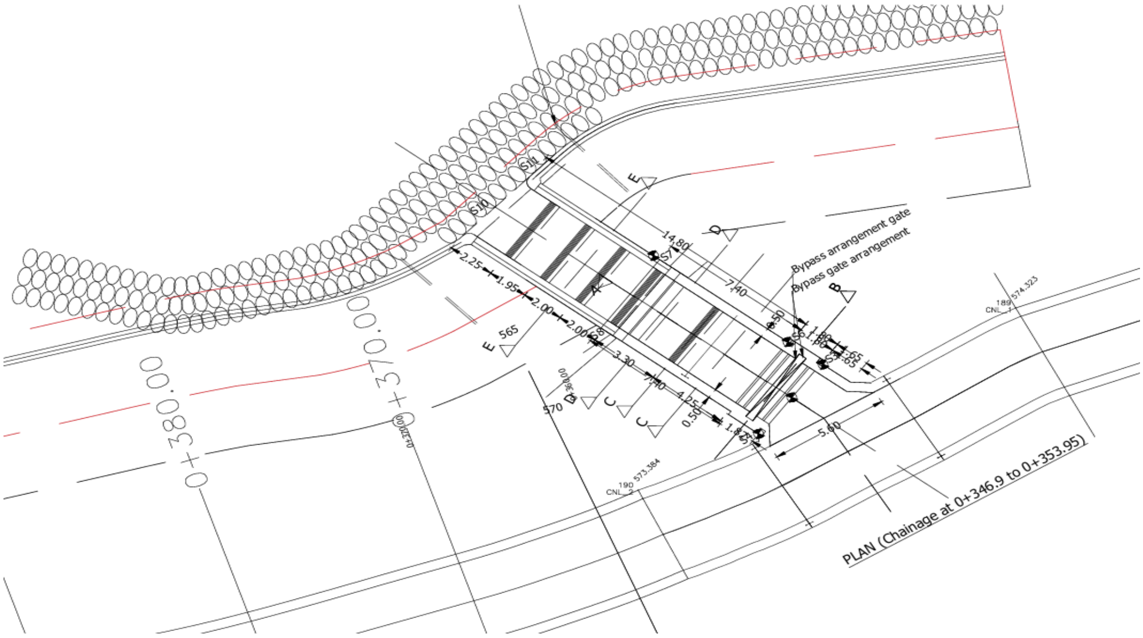


Figure 27 : Plan of By Pass Gate

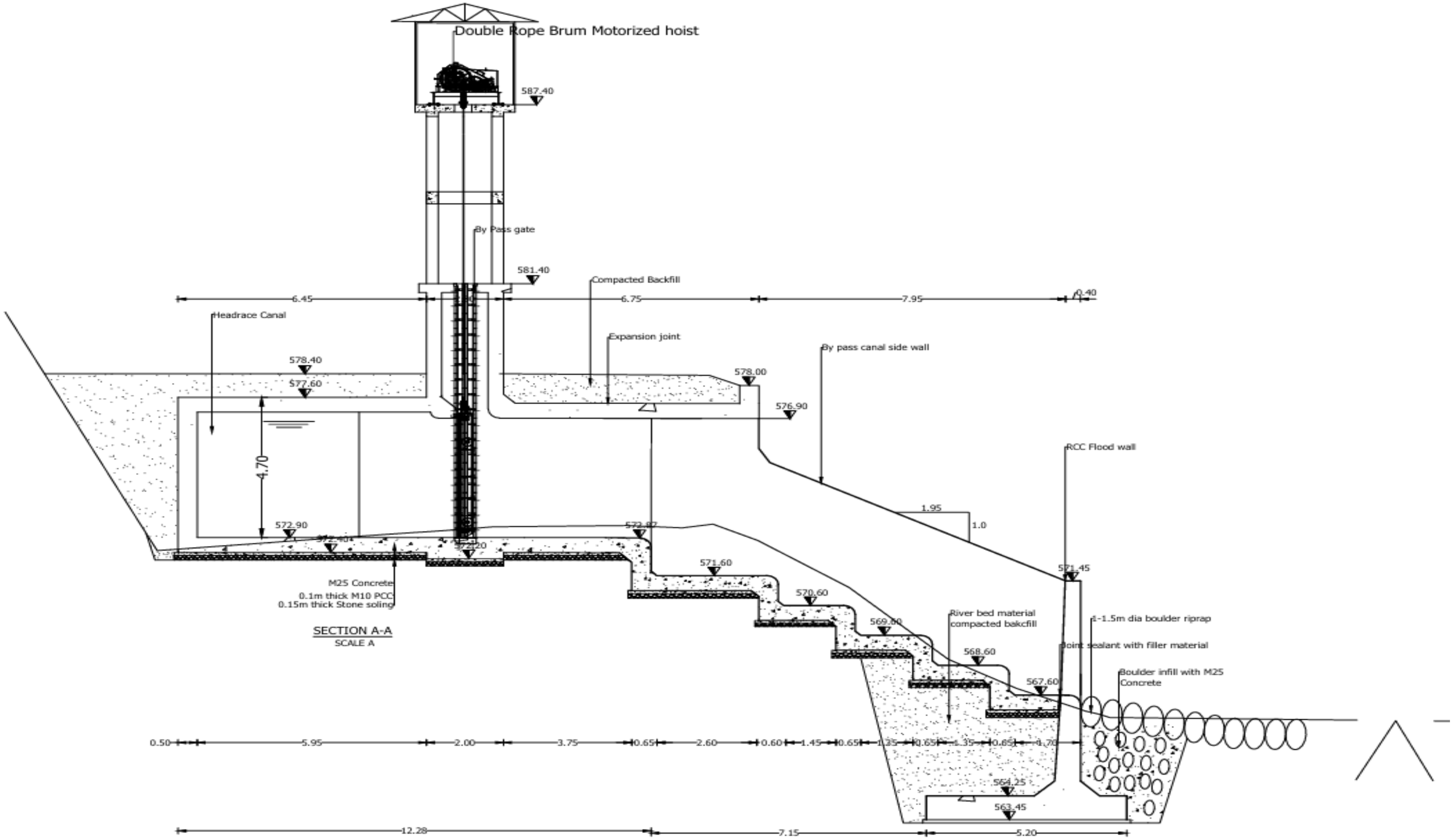


Figure 28:Section of By Pass Gate

2.8 Siphon

The function of the siphon with the length 50 m in the headrace canal of KAHEP is to protect it from the Khangrawa drainage (Kartikey Kholsi) during the rainy season. During heavy rainfall, the drainage experiences a significant increase in water flow, which can lead to the collapse and failure of the headrace canal if continue at the same level of the headrace canal. The length of siphon is 50 m.

To prevent such a situation, a siphon is constructed to allow the canal to pass beneath the drainage, ensuring its stability. Additionally, protective measures, such as plum concreting, are implemented to reinforce the structure and safeguard the canal from potential damage. The function of a siphon is to facilitate the crossing of obstacles while maintaining a continuous water flow and ensuring pressure flow mechanisms. Additionally, it plays a crucial role in enhancing structural safety by preventing potential failures in the system.

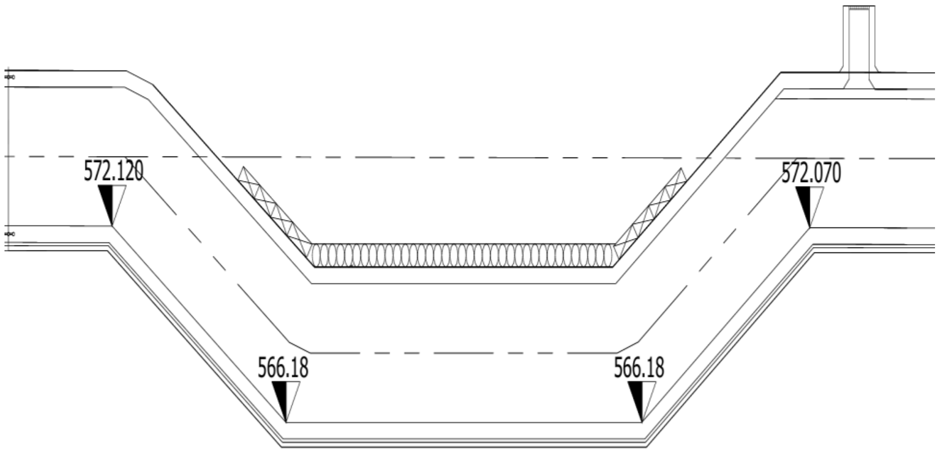


Figure 29 : L -Section of Siphon

2.9 Headpond and Spillway

The headpond is a crucial component of the water storage and regulation system, designed to temporarily store water before it is directed towards a hydraulic structure, such as a powerhouse or irrigation system. Since Kabeli-A is a run-of-river project, it does not have a large reservoir for storing water. Instead, the headpond plays a crucial role in short-term water storage and flow stabilization to ensure smooth operation.

By optimizing water management, the headpond increases the overall efficiency and reliability of the Kabeli-A Hydropower Project, contributing to Nepal's electricity generation with minimal environmental impact.

2.9.1 Role of the Head-pond in the Kabeli-A Project

Water Flow Regulation:

- The headpond ensures a continuous and controlled water supply to the penstock.
- Helps balance fluctuations in river discharge, especially during seasonal variations.

Sediment Management:

- Allows heavier sediments to settle before water enters the intake.
- Reduces turbine wear and increases the lifespan of hydropower equipment.

Enhancing Hydropower Efficiency:

- Maintains a stable water level, ensuring a steady water head (pressure) for power generation.
- Prevents sudden water shortages, which could affect power output.

Emergency Water Storage:

- Acts as a buffer during low flow conditions, allowing the plant to operate efficiently even when the river flow is reduced.

Figure 30 : Concrete Work at HeadPond Structure

2.9.2 Specifications

- Type: Rectangular Concrete Lined
- Number: One
- Length: 50 meters
- Width: 15 meters
- Height: 8.5 meters

2.9.3 Function and Importance

The rectangular concrete-lined headpond ensures efficient water flow regulation, minimizes seepage losses, and prevents erosion. Its 50-meter length and 15-meter width allow for an adequate volume of water storage, while the 8.5-meter height provides sufficient depth to maintain the required water head for downstream applications.

Being concrete-lined, the headpond is durable, resistant to environmental wear, and reduces maintenance costs compared to unlined or earthen alternatives. It plays a critical role in optimizing water management, improving hydraulic efficiency, and ensuring the smooth operation of associated infrastructure.

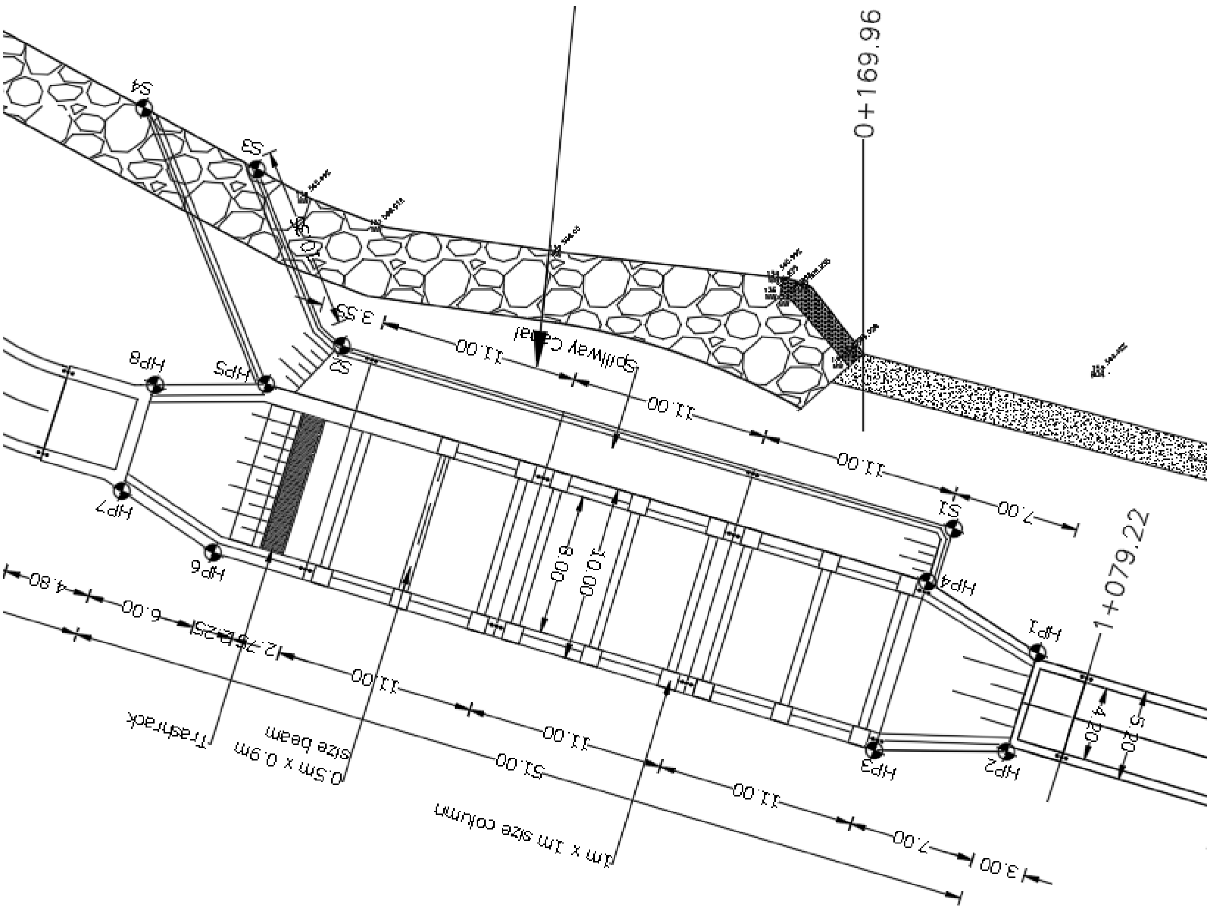


Figure 31: Plan of Head-Pond and Spillway

2.10 Surge Shaft and Valve Chamber

The surge shaft is an underground structure partially exposed to the surface, designed to regulate pressure fluctuations in the water conveyance system. It has an internal diameter of 12 meters and a total height of 54.6 meters. The bottom level of the surge shaft is at 540.95 meters above sea level (masl), while the top level reaches 595.50masl.

2.10.1 Hydraulic Parameters

- Minimum Down-surge Level: 558.17masl
- Static Surge Level: 575.50masl
- Upsurge Level: 591.50masl

2.10.2 Structural Configuration

The surge shaft is divided into three sections, each with varying diameters and concrete lining thicknesses based on hydraulic conditions:

Section 1-1 (Minimum Surge Area)

- Diameter: 12.0 m
- Concrete Lining: 0.7 m (M25 grade)

Section 2-2 (Static Surge Area)

- Diameter: 12.4 m
- Concrete Lining: 0.5 m

Section 3-3 (Upsurge Area)

- Diameter: 12.8 m
- Concrete Lining: 0.3 m

2.10.3 Support and Reinforcement

To ensure structural stability, the following reinforcement measures are implemented:

- Rock Bolting: 4-meter-long, 25 mm diameter rods installed at 1.7 m × 1.7 m spacing.
- Shotcrete Lining: A 10 cm thick Steel Fiber Reinforced (SFR) shotcrete layer is applied over the surface.

2.10.4 Geological and Geotechnical Conditions

The surge shaft is located in an area primarily composed of phyllite with intercalations of thin quartzite laminas, which are exposed a few meters downhill at a rock cliff. The proposed location was investigated using 2D Electrical Resistivity Tomography (ERT) and a 40-meter-deep borehole. Bedrock was encountered at a depth of 2.75 meters, with an average core recovery of 86% up to 25 meters. The Rock Quality Designation (RQD) was 33% up to 25 meters, while from 25 to 40 meters, more than 50% core losses were observed, resulting in an RQD of less than 10%. The rock lithology consists of strong to moderately strong, hard, light green to grey phyllite with micro folds and quartz veins up to 25 meters depth, transitioning into highly fractured light grey phyllite below this depth.

2.10.5 Hydrogeological Considerations

Packer tests conducted at 12 meters and 18 meters depth yielded Lugeon values of 11.44 and 10.48, respectively. However, excessive water loss prevented further packer tests in other sections, indicating highly fractured and weathered rock conditions.

2.10.6 Location Adjustment

Due to the high degree of weathering and fracturing at the initially proposed location, the surge shaft was relocated 25 meters northwest towards the hillside, where rock conditions are expected to be more stable. This adjustment ensures the structural integrity of the surge shaft, optimizing its efficiency in energy dissipation and pressure regulation within the hydraulic system.

Figure 32 : Surge Shaft

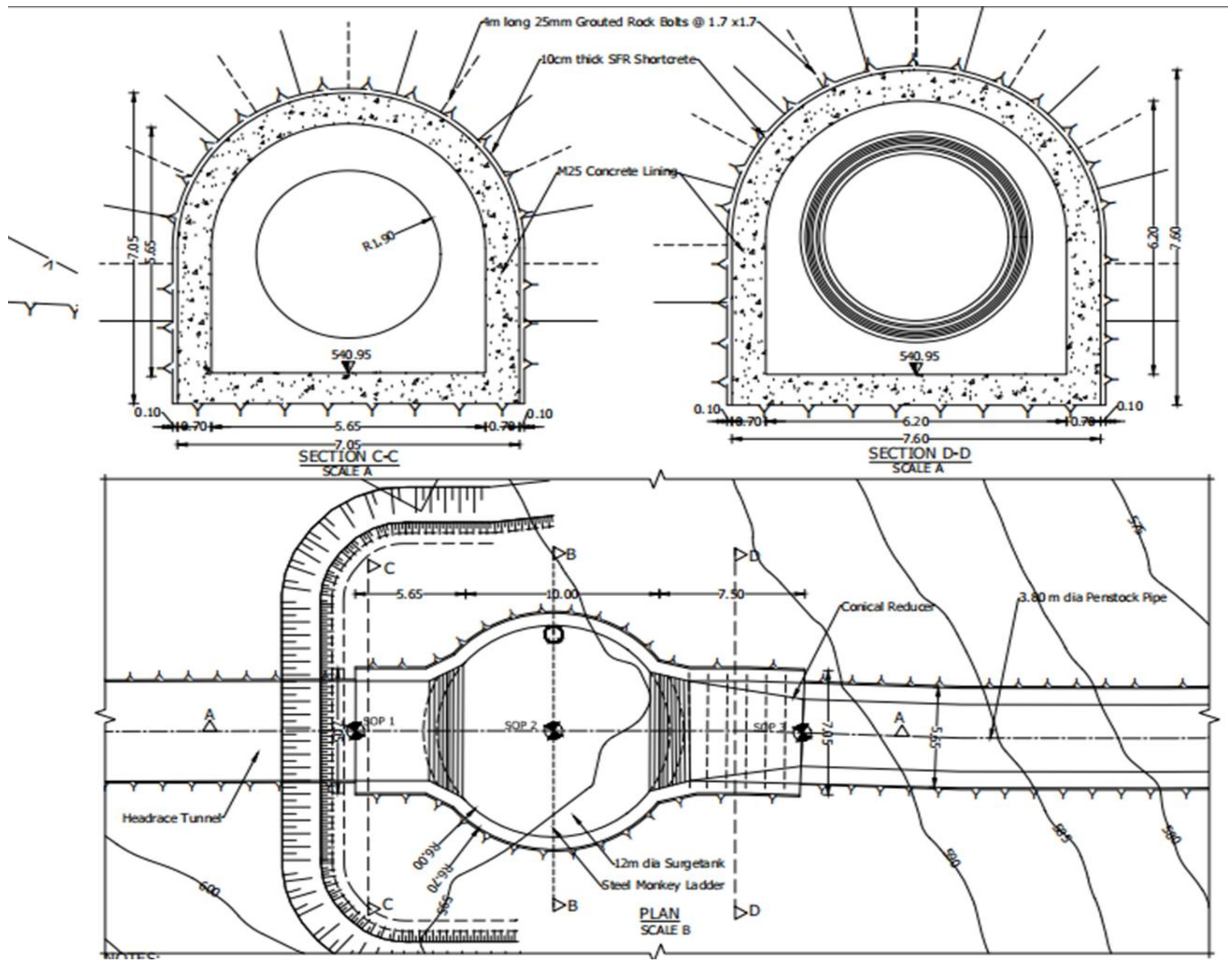


Figure 33 : Plan of Surge Shaft

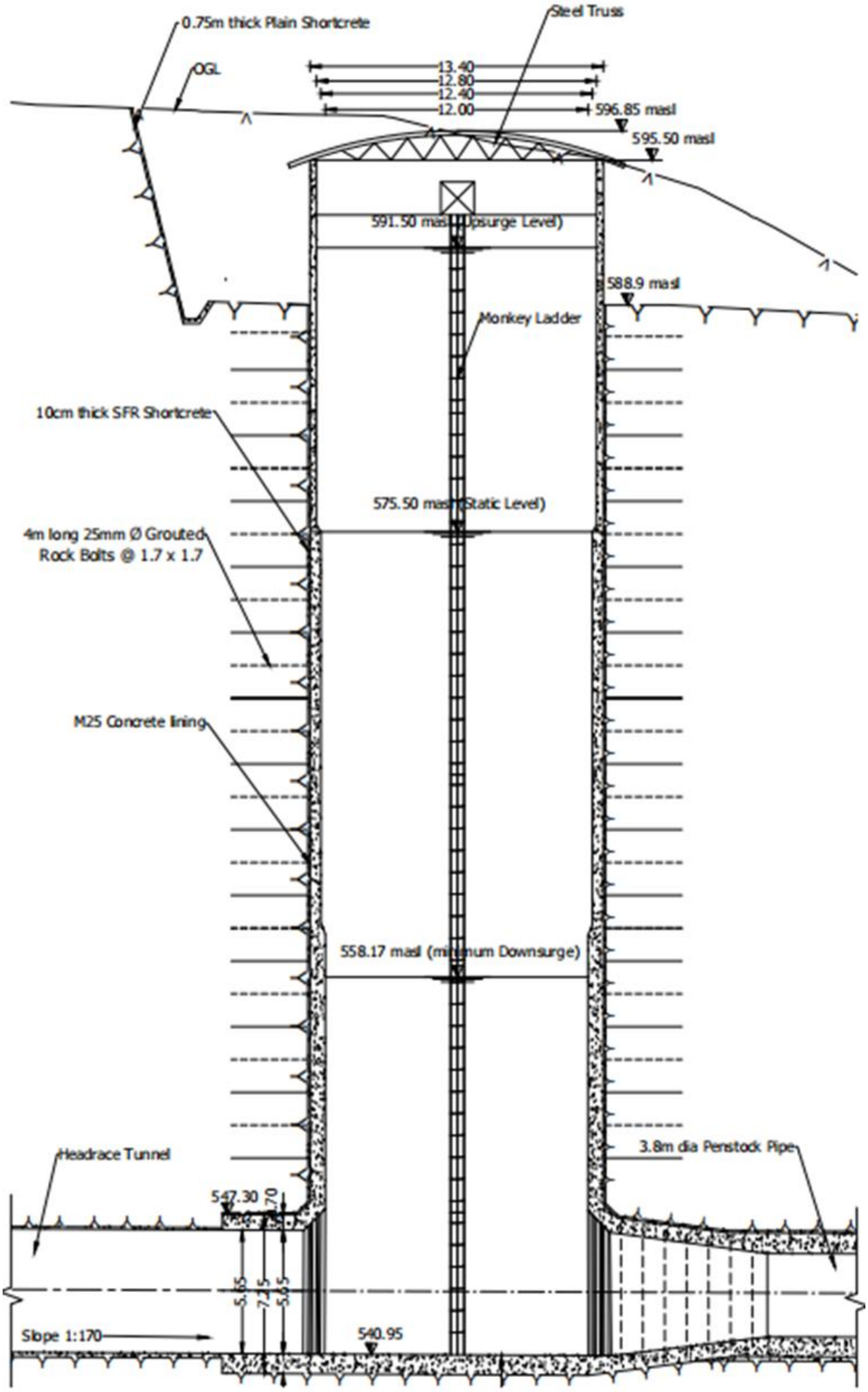


Figure 34 : Section A-A of Surge Shaft

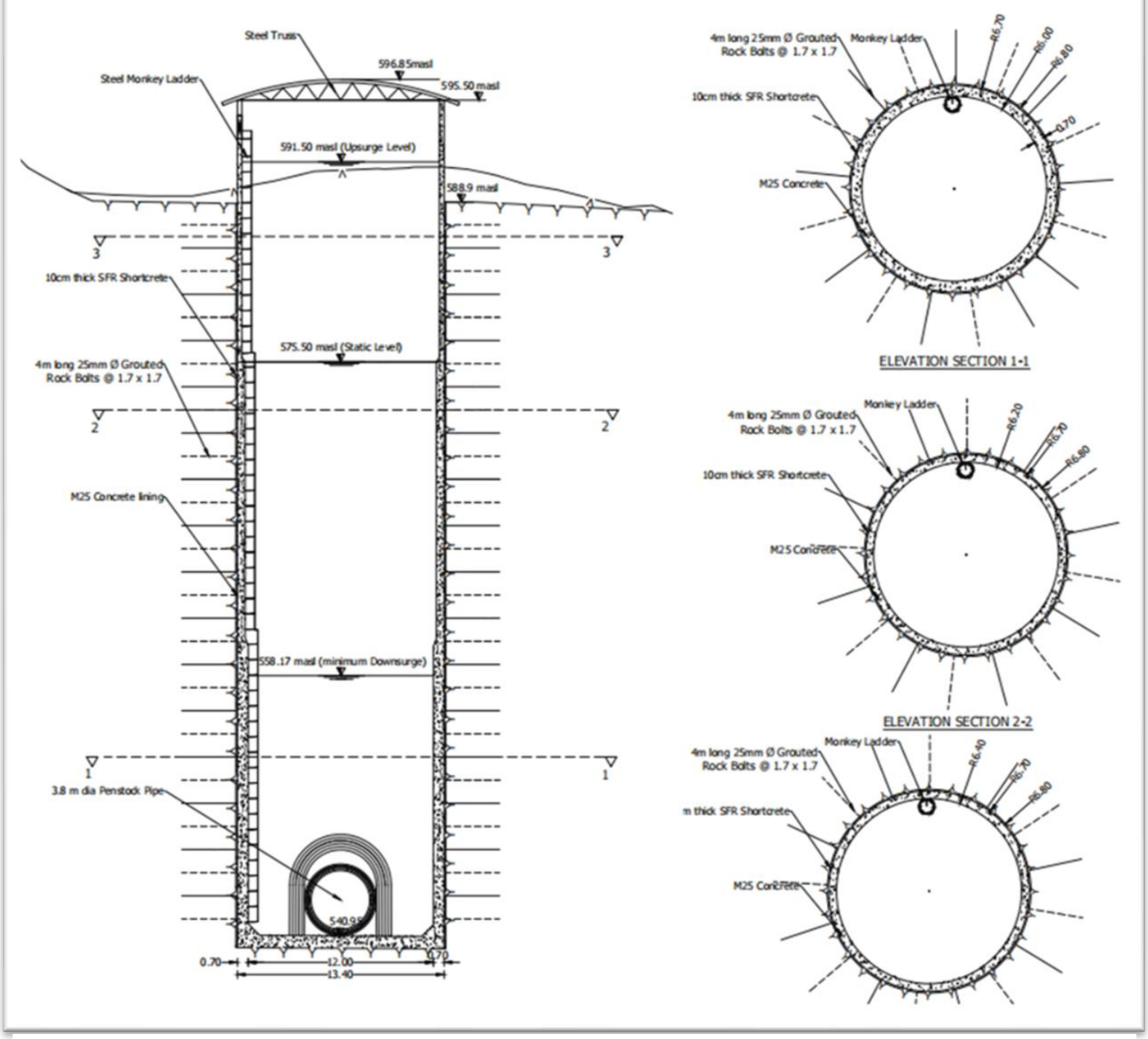


Figure 35 : Section B-B of Surge Shaft

2.11 Pressure Shaft

Penstock is pipe that convey the flow from the forebay or surge tank to turbine. penstock are designed to carry water to the turbine with the least possible loss of head consistent with the overall economy of project.

2.11.1 Pipe Specifications and Functions

The penstock system consists of seven different sections, each serving a specific function in water conveyance.

2.11.1.1 Large-Diameter Main Pipes (3.80 m)

Purpose: These pipes form the primary water transport route, connecting major stations.

Details:

- Transition to Station-1: The longest section (146.15 m) with a 12 mm thickness, weighing 164,874.12 kg.
- Station-1 to Station-2: A 35.86 m section, thicker at 16 mm, with a weight of 53,995.57 kg.
- Station-2 to Station-3: Slightly longer at 38.98 m, with a 20 mm thickness and 73,443.73 kg weight.
- Station-3 to Bifurcation-1 (Bif-1): The thickest in this category at 25 mm, but shorter at 26.8 m, weighing 63,201.27 kg.
  These pipes are critical as they handle the highest pressure and water volume.

2.11.1.2 Intermediate-Diameter Pipe (3.10 m)

- Bifurcation-1 to Bifurcation-2 (Bif-1 to Bif-2):
- Diameter: 3.10 m, length: 8.2 m, thickness: 20 mm.
- Weight: 12,618.80 kg.

Function: Distributes water from the main pipeline to branch pipelines, reducing pressure and ensuring smooth flow.

2.11.1.3 Medium-Sized Branch Pipes (2.20 m)

Branch Pipes:

- Diameter: 2.20 m, length: 45.18 m, thickness: 16 mm.
- Weight: 39,505.31 kg.

Function: These pipes branch off from the main line to distribute water to different sections of the hydro system.

2.11.1.4 Smallest Pipe (1.00 m)

PKSHP Tailrace Tapping Chamber to Headpond:

- Diameter: 1.00 m, length: 91.6 m, thickness: 8 mm.
- Weight: 18,216.52 kg.

Function: Used to regulate water flow from the tailrace chamber to the headpond, ensuring controlled water discharge.

2.11.2 Material Used

Most pipes are made of IS:2062 E350BR, a high-strength structural steel known for its durability and resistance to high pressure.

The smallest pipe (1.00 m) uses IS:2062 E250BR, a slightly lower-grade material suitable for lower-pressure applications.

2.11.3 Total Weight and Importance

The total weight of all penstock sections combined is 425,855.32 kg, demonstrating the massive scale of the hydro project. The weight variation among sections highlights the need for thicker pipes in high-pressure areas and lighter pipes in lower-pressure regions.

Table 8: Penstock Diameter and Weight

S. N.	Pipe Diameter (m)	Pipe Length (m)	Pipe Thickness (mm)	Material Type	Quantity (kg)	Remarks
1	3.80	146.15	12	IS:2062 E350BR	164,874.12	Transition to Station-1
2	3.80	35.86	16	IS:2062 E350BR	53,995.57	Station-1 to Station-2
3	3.80	38.98	20	IS:2062 E350BR	73,443.73	Station-2 to Station-3
4	3.80	26.8	25	IS:2062 E350BR	63,201.27	Station-3 to Bif-1
5	3.10	8.2	20	IS:2062 E350BR	12,618.80	Bif-1 to Bif-2
6	2.20	45.18	16	IS:2062 E350BR	39,505.31	Branch Pipes
7	1.00	91.6	8	IS:2062 E250BR	18,216.52	PKSHP Tailrace Tapping Chamber to Headpond
Total					**425,855.32**	

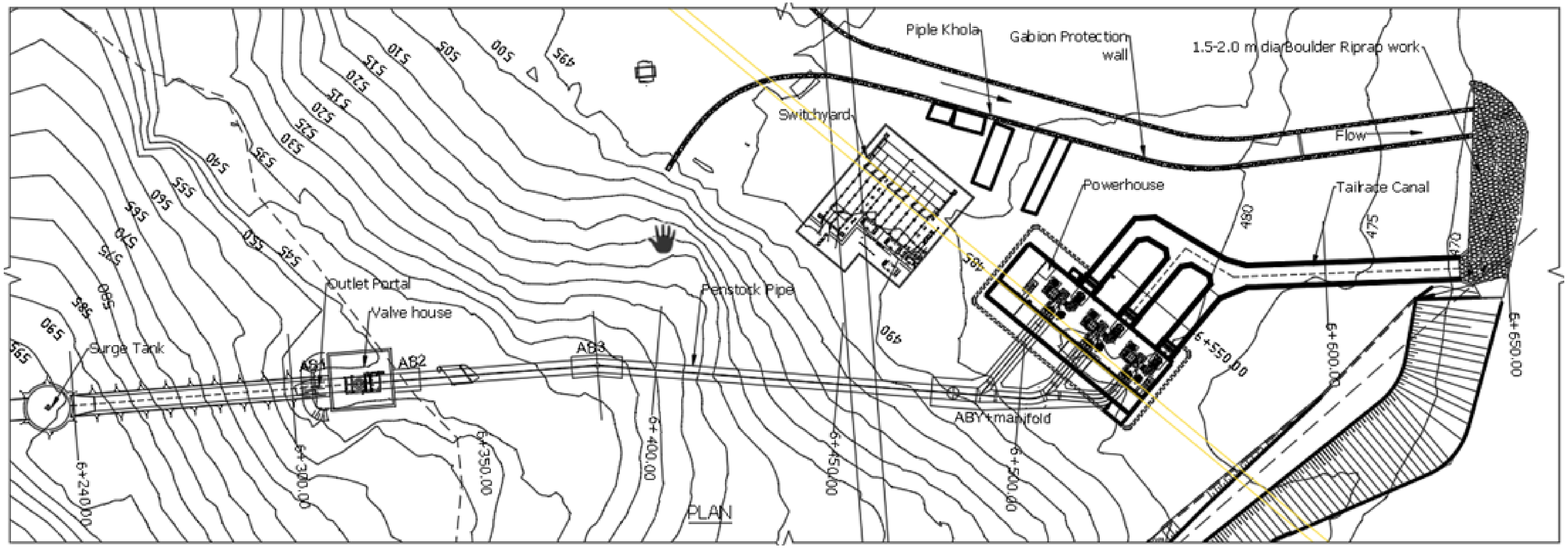


Figure 36 : Section B-B of Surge Shaft

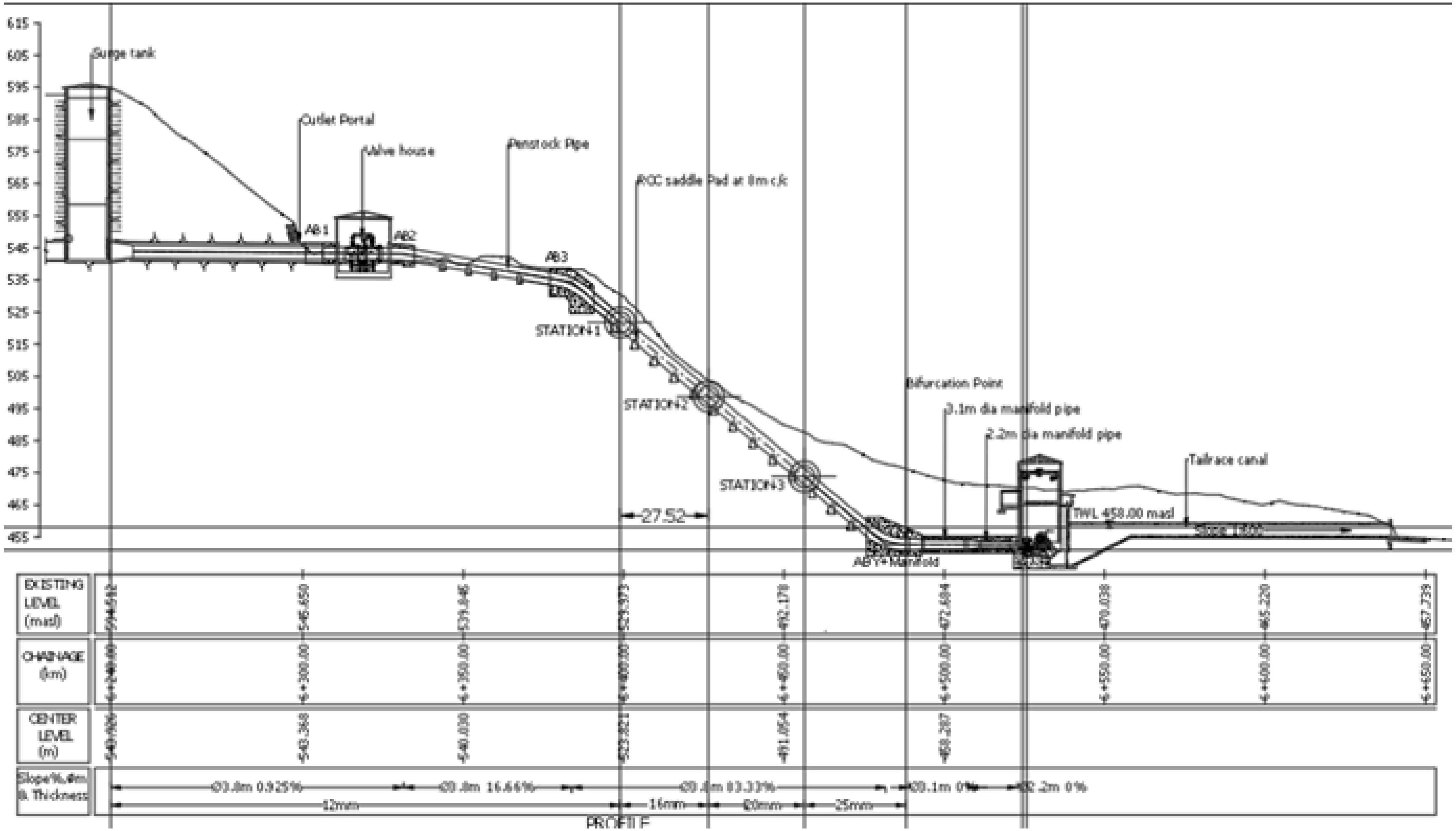


Figure 37 : Profile View of Penstock Pipe

Figure 38 : Fabrication of Penstock Pipe

Figure 39 : Erection of Bifurcation

2.11.4 Quality Assurance of Penstock Welds

2.11.4.1 Ultrasonic Testing (UT) of Penstock Pipe After Welding

Ultrasonic Testing (UT) is a non-destructive testing (NDT) method used to detect internal and external flaws in the welded joints of penstock pipes. Since penstocks operate under high pressure in hydroelectric power plants and water transmission systems, ensuring the quality of welds is crucial to prevent leaks and failures.

Steps of Ultrasonic Testing (UT) in Penstock Pipe Welding

1. Surface Preparation:

- ➢ The welded area is cleaned to remove dirt, rust, oil, or any surface contaminants.
- ➢ A smooth surface ensures better sound wave transmission.

2. Couplant Application:

- ➢ A coupling gel or liquid is applied to the test area to eliminate air gaps between the probe and the pipe surface, ensuring proper sound wave transmission.

3. Probe Placement and Sound Wave Transmission:

- ➢ A transducer (probe) generates high-frequency ultrasonic waves into the weld.
- ➢ These waves travel through the material and reflect back when they encounter a flaw or a boundary.

4. Signal Interpretation:

- ➢ The returning echoes are displayed on a screen as waveforms.
- ➢ Defects such as cracks, voids, and lack of fusion are identified based on signal variations.

5. Evaluation and Documentation:

- ➢ The detected defects are analyzed, and their size, location, and type are recorded.
- ➢ If defects exceed the permissible limit, the weld is repaired and re-tested.

Advantages of UT for Penstock Pipes

- Detects both surface and subsurface defects.
- Provides precise defect location and size estimation.
- Suitable for thick-walled penstock pipes.
- Non-destructive and does not damage the pipe.

Limitations

- Requires skilled operators to interpret results.
- Surface roughness or improper couplant application can affect accuracy.
- Not ideal for irregularly shaped or highly curved surfaces.

Ultrasonic Testing is a critical quality assurance method to ensure the strength and reliability of welded penstock pipes, reducing the risk of structural failure in hydropower plants and water distribution systems.

Figure 40 : UT Test

2.11.4.2 Die Penetration Test of Penstock Pipe After Welding

The die penetration test (DPT) is a non-destructive testing (NDT) method used to detect surface defects such as cracks, porosity, and incomplete fusion in welded joints of penstock pipes. Since penstocks operate under high pressure, ensuring the integrity of welds is crucial to prevent leaks or failures.

Steps of the Die Penetration Test

1. **Surface Preparation:**
   - The welded area is cleaned using a solvent to remove dirt, grease, and other contaminants.
   - Any surface irregularities like slag or spatter are removed to ensure accurate test results.
2. **Application of Penetrant:**
   - A liquid penetrant (usually red or fluorescent) is applied to the surface of the weld.
   - The penetrant is allowed to seep into any surface defects through capillary action.
3. **Dwell Time:**
   - The penetrant is left on the surface for a specified duration (typically 10-30 minutes) to allow it to enter any cracks or defects.
4. **Excess Penetrant Removal:**
   - The excess penetrant is carefully wiped off using a lint-free cloth and a cleaner without removing the penetrant trapped in defects.
5. **Developer Application:**
   - A developer (dry powder or liquid) is sprayed over the surface.
   - The developer draws out the penetrant from defects, making them visible as red (for visible dye) or glowing marks (for fluorescent dye under UV light).
6. **Inspection & Evaluation:**
   - The weld is examined under appropriate lighting conditions.
   - If defects are found, they are marked for repair and re-inspected after correction.
7. **Cleaning & Documentation:**
   - After inspection, the surface is cleaned again to remove the test materials.
   - Results are documented for quality control and safety records.

Advantages of DPT for Penstock Pipes

- Simple and cost-effective
- Detects fine surface cracks and porosity
- Does not require complex equipment
- Can be performed on various materials, including steel

Limitations

- Only detects surface defects (not subsurface flaws)
- Requires proper cleaning to avoid false results
- Ineffective on rough or porous surfaces

The **die penetration test** is an essential quality control measure in penstock pipe welding to ensure durability and prevent failures in hydroelectric power plants and water supply systems.

Figure 41 : Die Penetration Test

2.12 Approach Tunnel

A 69.5 m long, 4.01 m (finished) diameter concrete lined Inverted D shape headrace tunnel connecting approach canal to settling basin for conveying 37.73 cumec of water is proposed for the Project.

2.12.1 Design Criteria of Approach Tunnel

The Approach is excavated by Drill and Blast Method (DBM). Rock load is taken care by the proper provision of rock supports comprising the steel ribs with back fill concrete, consolidation grouting of surrounding rock mass, rock bolts and/or shotcrete as per the actual rock conditions After excavation and installation of support system, once the rock-mass is stabilized, lining is being provided. Tunnel lining is being done with the help of The analysis and design of rock support system in normal reaches, has been performed by the Design of rock support system by empirical method using Q values, as given by Barton.

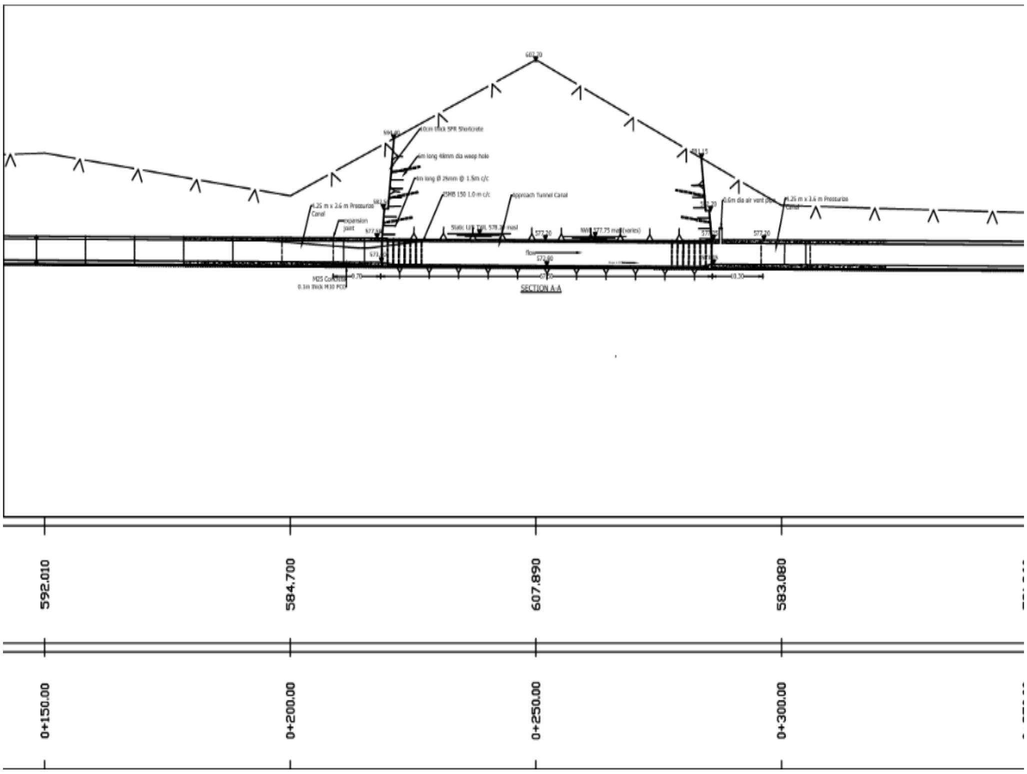


Figure 42 : L-Section of Approach Tunnel

2.12.2. Layout

The Approach Tunnel is located in the left bank of kabeli river. It is proposed to provide 7.15 m (W) x 5.25 m (H) excavated section of HRT having 0.4 m thick concrete lining at Invert, Walland crown,10-15 cm thick plain shotcrete and 50 cm thickness at dividing wall. The sectional details of Approach Tunnel excavated section for rock (Type -Iv) is given in figure below.

The tunnel section is divided into two parts by a dividing wall. The Larger section is designed for the tailrace water from Kabeli B Cascade hydropower, while the smaller section is meant for the water from the Kabeli River. When the Kabeli B Cascade is not operational, the water from the Kabeli River flows through an interconnected gate and passes through the settling basin. In such cases, the hydropower plant can operate at full capacity by drawing water solely from the Kabeli River.

Figure 43 : Approach Tunnel

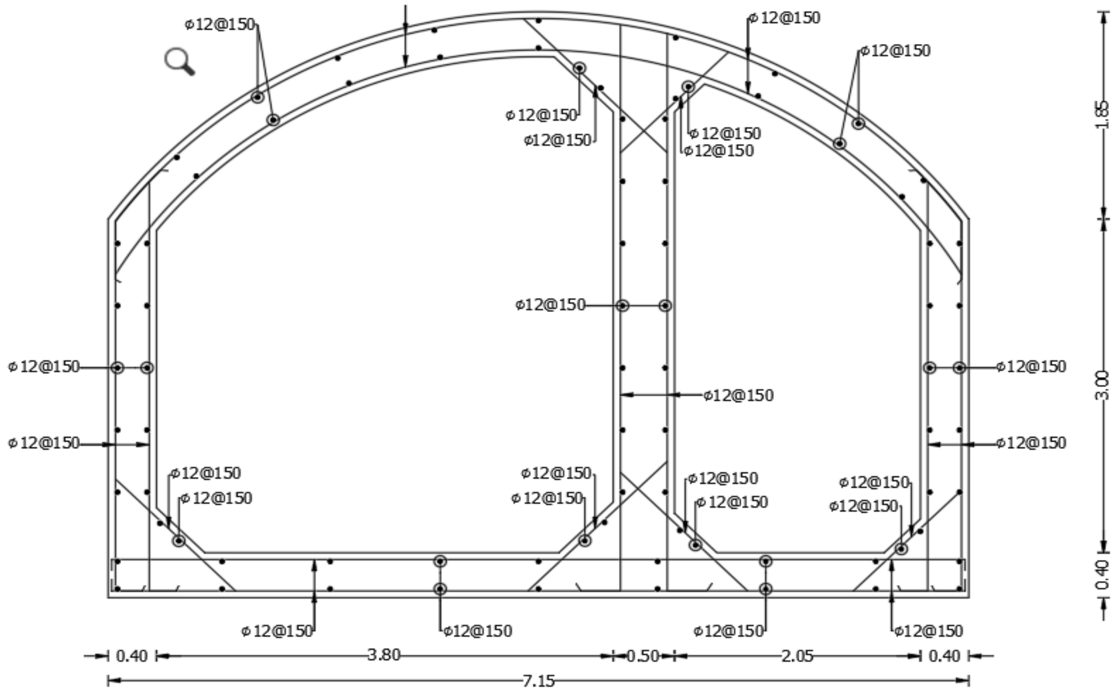


Figure 44:Typical Section of Approach Tunnel

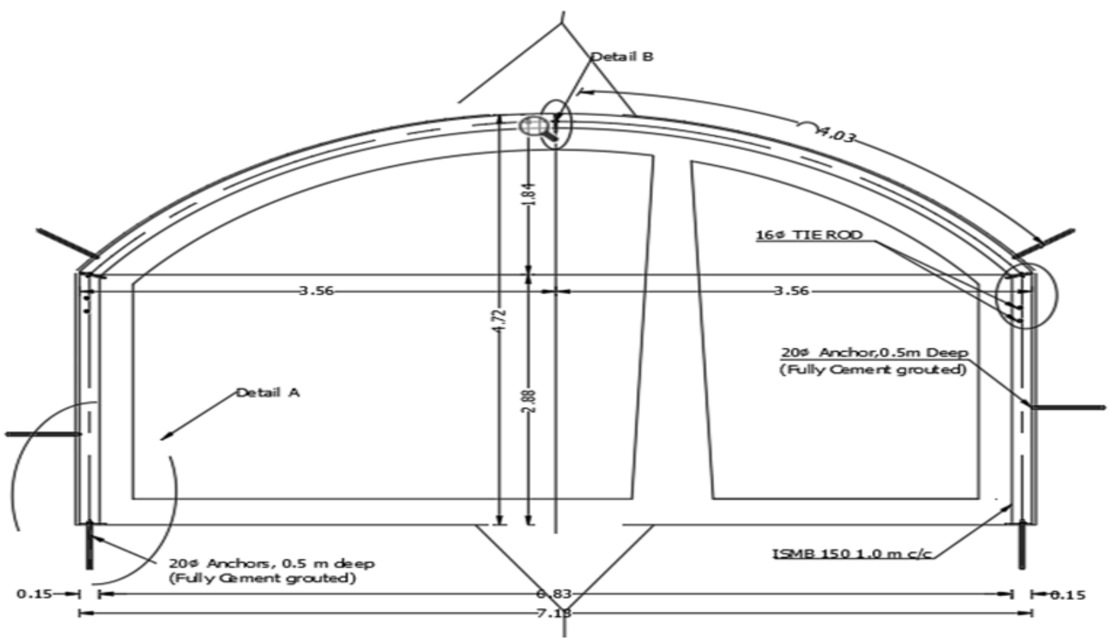


Figure 45 : Section of Concrete Lining Between Ribs (Rock Type-V)

2.12.3 Tunnel Cycle

Various stages of drill and blast cycle are listed below. The same cycle is repeated until the tunnel excavation is fully completed. The methods of securing and drilling may vary depending on the quality and condition of the rock.

- Drill pattern design
- Drilling
- Loading and Blasting
- Mucking
- Securing
- Geological Mapping

2.12.4 Rock support in Approach tunnel and excavated size

Table 9:Excavated size of HRT in different rock condition

S.N.	'Q' Value	Rock Category	Rock Support	
			Shotcrete	**Rock bolt, Ribs, Spiling, incline, and Concrete lining**
1	0.1-0.05	Type -V	15 cm thick plain Shotcrete	Rock Bolt :25 mm dia. rock bolts, 2.5 m Long Pattern Bolting @ 1.2 m c/c ISMB :150 * 75 mm Steel Ribs @ 1.5 m c/c Concrete Lining: 40 cm Thick at Invert, 40 cm at wall and Crown
3	0.1–0.5	Type -IV	10-15cm thick plain Shotcrete	Rock Bolt :25 mm dia. rock bolts, 2.5 m Long Pattern Bolting @ 1.5 m c/c Concrete Lining: 40cm Thick at Invert ,wall and crown

2.12.5 Excavation on Approach Tunnel

Table 10:Excavated size of HRT in different rock condition

Rock Type	**B**	**H**
Type -IV (Q = 0.1 -0.5)	7.15 m	5.25 m
Type – V (Q = 0.1-0.05)	7.15 m	5.25 m

2.12.6 Rock Bolting Procedure of Installation

Rock bolt, in tunneling and underground mining, steel rod inserted in a hole drilled into the roof or walls of a rock formation to provide support to the roof or sides of the cavity. Rock bolt reinforcement can be used in any excavation geometry, is simple and quick to apply, and is relatively inexpensive. The installation can be fully mechanized. The length of the bolts and their spacing can be varied, depending on the reinforcement requirements.

Following procedure shall be adopted for installation of rock bolt:

- Evaluate Rock Mass: Using techniques such as RMR, Q-system, or GSI, classify the rock's state and determine its stability and suitability for support. This requires a thorough geological study.
- Customize Support Strategy: To provide the best possible reinforcement and tunnel stability, design a rock bolting pattern that is specific to the quality of the rock. This involves modifying the length, spacing, and density of bolts.

Drilling: A hole with the required diameter and depth is drilled into the tunnel's wall or ceiling.

- Cleaning: To guarantee a strong binding, dust and debris are removed from the drilled hole.
- Inserting the Bolt: Using tools or by hand, the rock bolt is inserted into the hole.
- Cement Grouting: To improve the binding between the bolt and the rock and to keep it in place, grout is injected into the hole surrounding the bolt. The bolt shall be tensioned while the cement is still green in the hole and is not set.

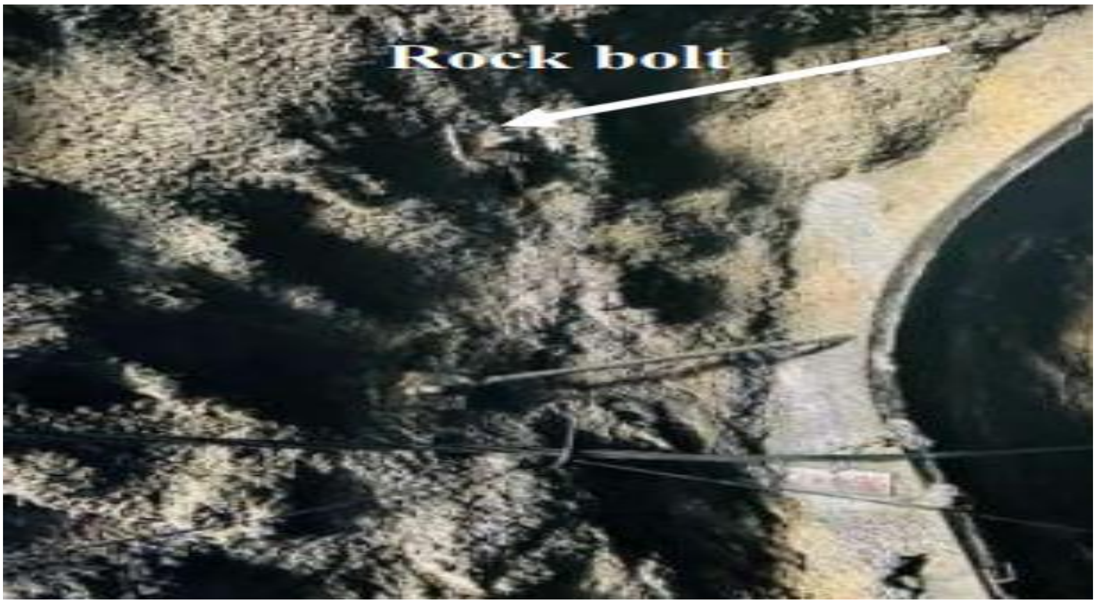


Figure 46 : Rock Bolting

2.13. Headrace Tunnel

A 4.327 km long, 2.83 m (finished) diameter concrete lined Inverted D shape headrace tunnel connecting forebay to penstock Shaft for conveying 37.73 cumec of water is proposed for the Project... The HRT is likely to pass through granite rock in the majority length of the tunnel mostly at the inlet. The downstream stretch of the alignment passes through gneiss, schist, quartzite and phyllite. Gradient of Headrace Tunnel is considered as downward 1 in 283 from start (Inlet Portion) to transition 1 , upward 1 in 168 from transition 1 to transition 2 , downward 1 in 1000 from transition 2 to transition 3 , upward 1 in 92 from transition 3 to transition 4 , downward 1 in 144 from transition 4 to end point of tunnel (Outlet Portion) Rock mass supporting system is proposed by Rock Bolt, Shotcrete and use of steel ribs. The entire length is proposed to be lined with concrete and subsequently contact/consolidation grouted.

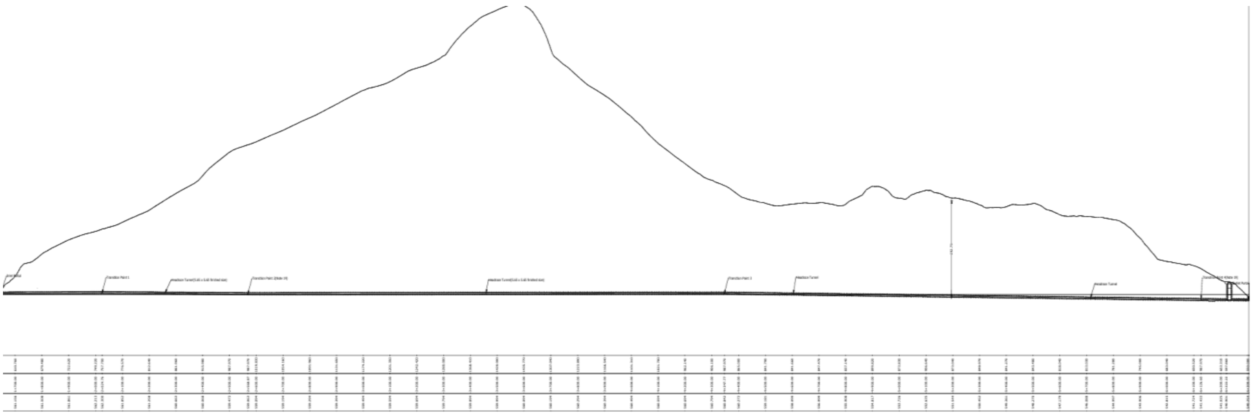


Figure 47 : L-Section of Headrace Tunnel

2.13.1Design Criteria of HRT

The Head Race Tunnel is excavated by Drill and Blast Method (DBM). Rock load is taken care by the proper provision of rock supports comprising the steel ribs with back fill concrete, consolidation grouting of surrounding rock mass, rock bolts and/or shotcrete as per the actual rock conditions. After excavation and installation of support system, once the rock-mass is stabilized, lining is being provided. Tunnel lining is being done with the help of gantry.

The analysis and design of rock support system in normal reaches, has been performed in the Following design of rock support system by empirical method using Q values, as given by Barton.

2.13.2. Layout

The Head Race Tunnel is located in the left bank of kabeli river. It is proposed to provide 5.75m (W) x 5.9m (H) excavated section of HRT having 0.2 m thick concrete lining at Invert only,5 cm thick plain shotcrete, 2.83 m vertical stem and 2.83m high crown for good rock. The sectional details of HRT excavated section for good rock (Type -I) is given in figure below.

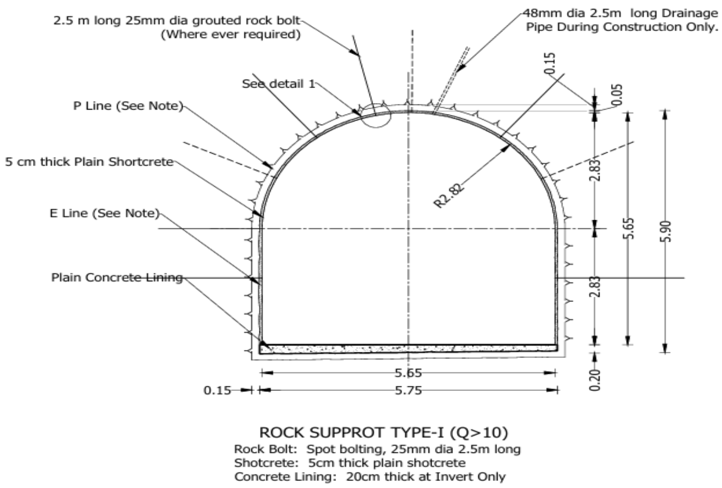


Figure 48 : Rock Type - I

It is proposed to construct a Head Race Tunnel (HRT) with an excavated cross-sectional dimension of 5.75 meters in width and 5.90 meters in height in Type-II (good quality) rock. The excavated tunnel profile comprises 2.83 meters high vertical sidewalls (stems) and a 2.83 meters high curved crown, forming a modified horseshoe-shaped section.

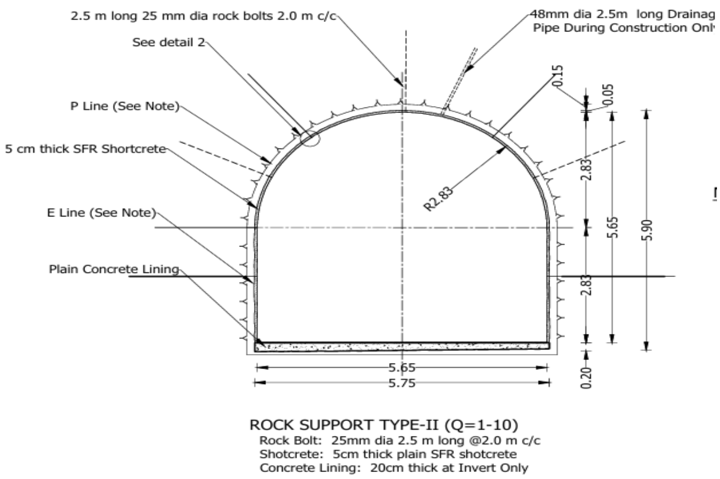


Figure 49 : Rock Type - II

It is proposed to provide 5.85m (W) x 5.9m (H) excavated section of HRT having, 20 cm thick concrete lining at Invert only,10 cm thick SFR shotcrete, 2.83 m vertical stem and 2.83m high crown. The sectional details of HRT excavated section for good rock (Type -III) is given in figure below.

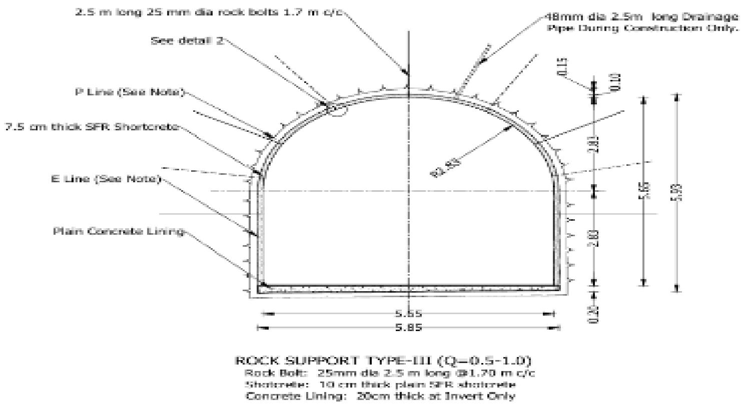


Figure 50 : Rock type - III

It is proposed to provide 6.15 m (W) x 6 m (H) excavated section of HRT having, 20 thick concrete lining at Invert only,10-15 cm thick SFR shotcrete, 2.83 m vertical stem and 2.83m high crown. The sectional details of HRT excavated section for good rock (Type -IV) is given in figure below.

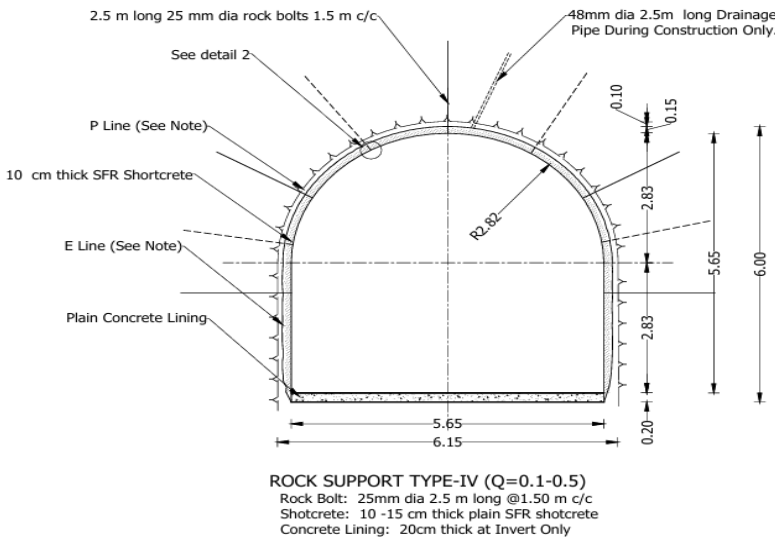


Figure 51 : Rock Type -IV

It is proposed to provide 6.35m (W) x 6.20m (H) excavated section of HRT having 20 cm thick concrete lining at Invert only and 20 cm at wall and crown ,15 cm thick SFR shotcrete , 2.83 m vertical stem and 2.83m high crown . The sectional details of HRT excavated section for good rock (Type -V) is given in figure below.

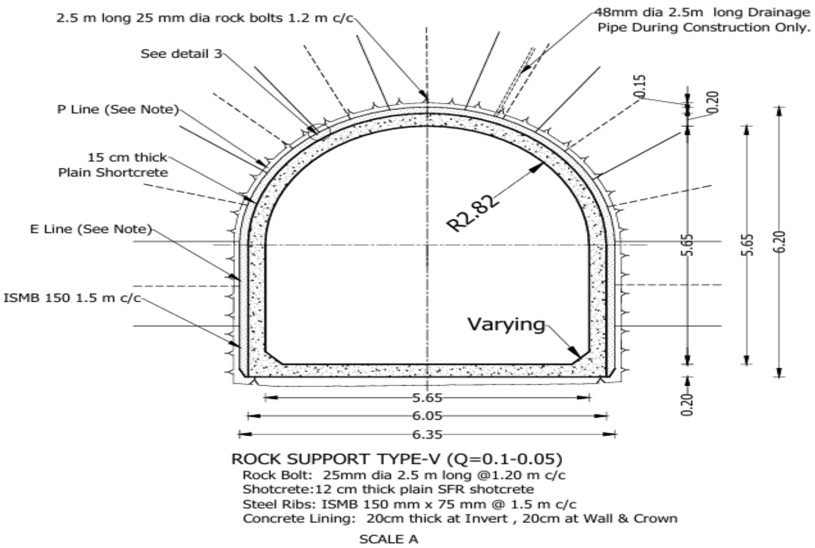


Figure 52 : Rock Type -V

It is proposed to provide 6.45m (W) x 6.40m (H) excavated section of HRT having 30 cm thick concrete lining at Invert, 25 cm at wall and crown, 2.83 m vertical stem and 2.83m high crown for Very Poor rock Type -VI. The sectional details of HRT excavated section for Very crown Poor rock is given in Figure below:

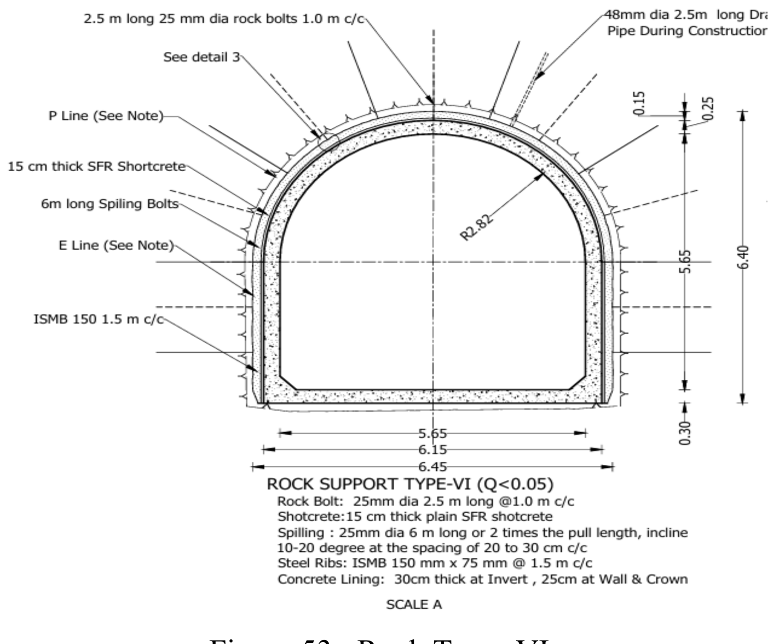
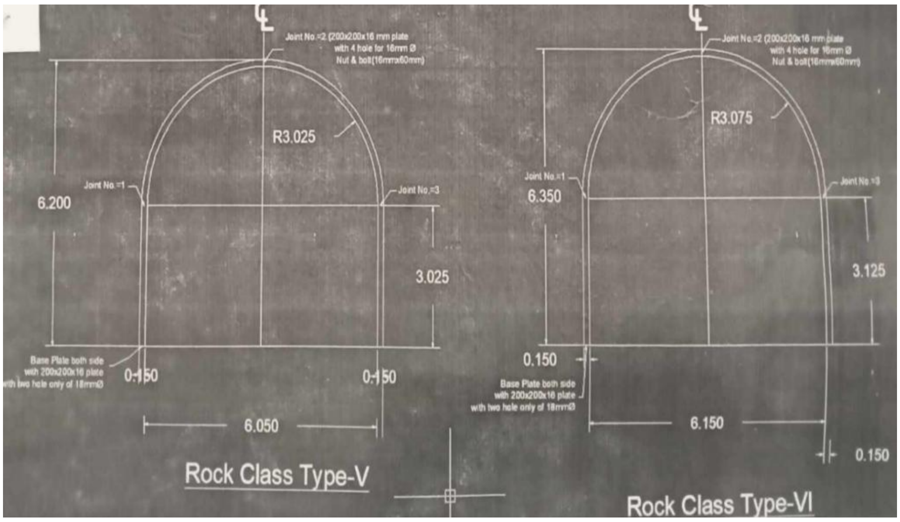


Figure 53 : Rock Type -VI



2.13.3 Tunnel Cycle

Various stages of drill and blast cycle are listed below. The same cycle is repeated until the tunnel excavation is fully completed. The methods of securing and drilling may vary depending on the quality and condition of the rock.

- Drill pattern design
- Drilling
- Loading and Blasting
- Ventilating
- Mucking
- Securing
- Geological Mapping

2.13.4 Rock support in Head Race Tunnel and excavated size

Table 11:Excavated size of HRT in different rock condition

S.N.	'Q' Value	Rock Category	Rock Support	
			Shotcrete	**Rock bolt, Ribs, Spiling, incline, and Concrete lining**
1	<0.05	Type -VI	15 cm thick plain Shotcrete	Rock Bolt : 25 mm dia. rock bolts, 2.5 m Long Pattern Bolting @ 1.0 m c/c, ISMB : 150 * 75mm Steel Ribs @ 1.5 m c/c Spiling : 25 mm dia 6 m long or 2 times the pull length Concrete Lining : 30cm Thick at Invert , 25cm at wall and Crown Incline : 10-20 degree at spacing of 20 to 30 cm c/c
2	0.1 - 0.05	Type -V	15 cm thick plain Shotcrete	Rock Bolt :25 mm dia. rock bolts, 2.5 m Long Pattern Bolting @ 1.2 m c/c ISMB :150 * 75 mm Steel Ribs @ 1.5 m c/c Concrete Lining : 20cm Thick at Invert , 20 cm at wall and Crown
3	0.1 – 0.5	Type -IV	10-16cm thick plain Shotcrete	Rock Bolt :25 mm dia. rock bolts, 2.5 m Long Pattern Bolting @ 1.5 m c/c Concrete Lining : 20cm Thick at Invert only
4	0.5 – 1	Type -III	10cm thick plain Shotcrete	Rock Bolt :25 mm dia. rock bolts, 2.5 m Long Pattern Bolting @ 1.7 m c/c Concrete Lining : 20cm Thick at Invert only
5	1 – 10	Type -II	5 cm thick Plain Shotcrete	Rock Bolt :25 mm dia. rock bolts, 2.5 m Long Pattern Bolting @ 2 m c/c Concrete Lining : 20cm Thick at Invert only
6	>10	Type -I	5 cm thick plain shotcrete	Rock Bolt : Spot bolting , 25 mm dia 2.5 m long Concrete Lining : 20cm Thick at Invert only

2.13.5 Excavation on HRT

Table 12:Excavated size of HRT in different rock condition

Rock Type	B	H
Type -I Good Rock (Q>10)	5.75 m	5.9 m
Type -II (Q = 1-10)	5.75 m	5.9 m
Type -III (Q = 0.5-1.0)	5.85 m	5.9 m
Type -IV (Q = 0.1 -0.5)	6.15 m	6.0 m
Type – V (Q = 0.1-0.05)	6.35 m	6.2 m
Type – VI (Q < 0.05)	6.45 m	6.40 m

2.13.6 Rock Bolting Procedure of Installation

Rock bolt, in tunneling and underground mining, steel rod inserted in a hole drilled into the roof or walls of a rock formation to provide support to the roof or sides of the cavity. Rock bolt reinforcement can be used in any excavation geometry, is simple and quick to apply, and is relatively inexpensive. The installation can be fully mechanized. The length of the bolts and their spacing can be varied, depending on the reinforcement requirements.

Following procedure shall be adopted for installation of rock bolt:

1) • Evaluate Rock Mass: Using techniques such as RMR, Q-system, or GSI, classify the rock's state and determine its stability and suitability for support. This requires a thorough geological study.

2) • Customize Support Strategy: To provide the best possible reinforcement and tunnel stability, design a rock bolting pattern that is specific to the quality of the rock. This involves modifying the length, spacing, and density of bolts.

a) • Drilling: A hole with the required diameter and depth is drilled into the tunnel's wall or ceiling.

b) • Cleaning: To guarantee a strong binding, dust and debris are removed from the drilled hole.

c) • Inserting the Bolt: Using tools or by hand, the rock bolt is inserted into the hole.

d) • Grouting: To improve the binding between the bolt and the rock and to keep it in place, grout is injected into the hole surrounding the bolt. The bolt shall be tensioned while the cement is still green in the hole and is not set.

2.13.7 Typical Blasting Pattern

A typical blasting pattern for an inlet tunnel involves a systematic drill hole layout, explosive charging, and a controlled blasting sequence.

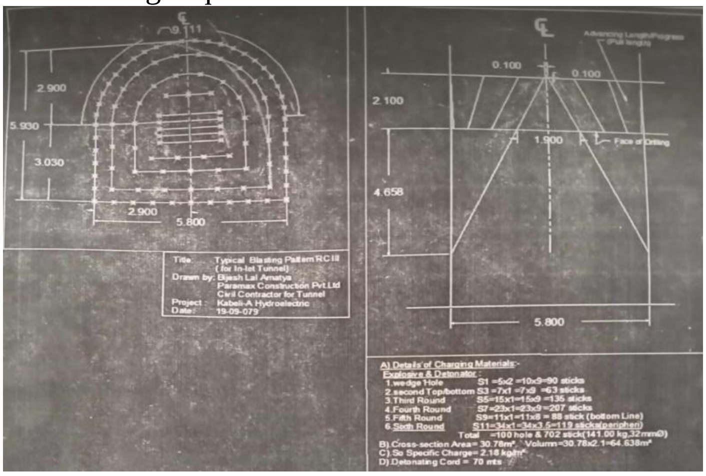


Figure 54 : Typical Blasting Pattern

2.13.8 Tunnel Failure at 1+703 section

A tunnel failure occurred at chainage 1+703 meters, where the tunnel invert level is approximately 557 meters above mean sea level (masl), and the existing ground level is around 1257masl. This indicates an overburden of approximately 700 meters. The collapse was primarily due to high overburden pressure, which induced localized stress concentrations in the surrounding rock mass. These excessive stresses likely exceeded the strength of the rock, resulting in a localized collapse of the tunnel section.

In the affected tunnel section at chainage 1+703 meters, approximately 25 meters in length has experienced failure. Within this zone, nearly 17 steel ribs have collapsed or been severely damaged. A backhoe loader was also present inside the failure zone at the time of the incident, posing additional risk and potential equipment damage.

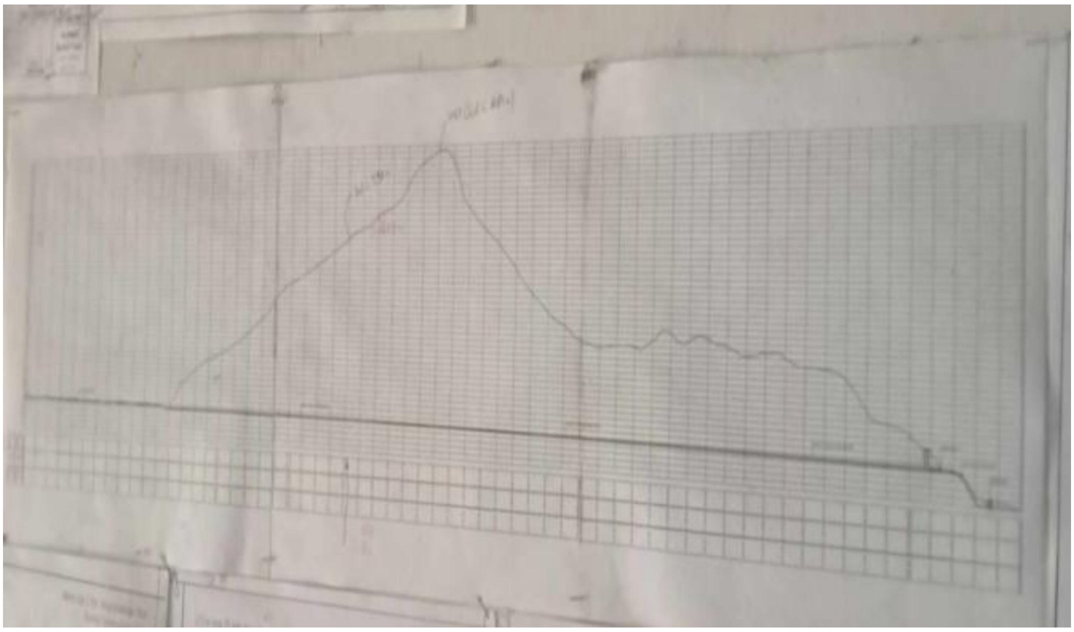


Figure 55 : L-Section of Tunnel

The primary support system in this section consisted of 3-meter-long rock bolts installed at 1.5-meter spacing. Despite these measures, the support system was unable to withstand the high overburden pressure and localized stress, leading to structural failure.

In the section where the tunnel failure occurred, both Class II and Class III rock types were encountered. Based on the geological conditions, the use of steel ribs was not mandatory. However, due to the presence of a high overburden—approximately 700 meters—ribs were installed as a precautionary safety measure.

Despite this, the overburden stress in the area was significantly high, leading to squeezing ground conditions. This caused several ribs to bend, and the tunnel crown also began to deform. To provide additional structural support, runners and C-sections were installed. However, even these elements were affected by the continuous stress and showed signs of bending.

Later, a small rockfall occurred from the tunnel crown, where further bending was observed. To support the shotcrete application in the deformed crown area, a wire mesh arrangement was provided. However, due to ongoing stress and deformation, the crown bending increased, making it difficult to carry out tunnel works safely. In particular, the clearance was reduced to the point where passing construction equipment like loaders became extremely difficult.

To address this issue, a decision was made to perform crown scrapping and remove the deformed portion. This operation was carried out using a backhoe. Unfortunately, during the scrapping, a sudden rock failure occurred, resulting in the collapse of approximately 25 meters of tunnel length.

To address the failure problem, wire mesh was first installed as an initial support measure immediately after the collapse. The reinforcement work is currently ongoing in the failed section to restore stability and ensure safe working conditions.

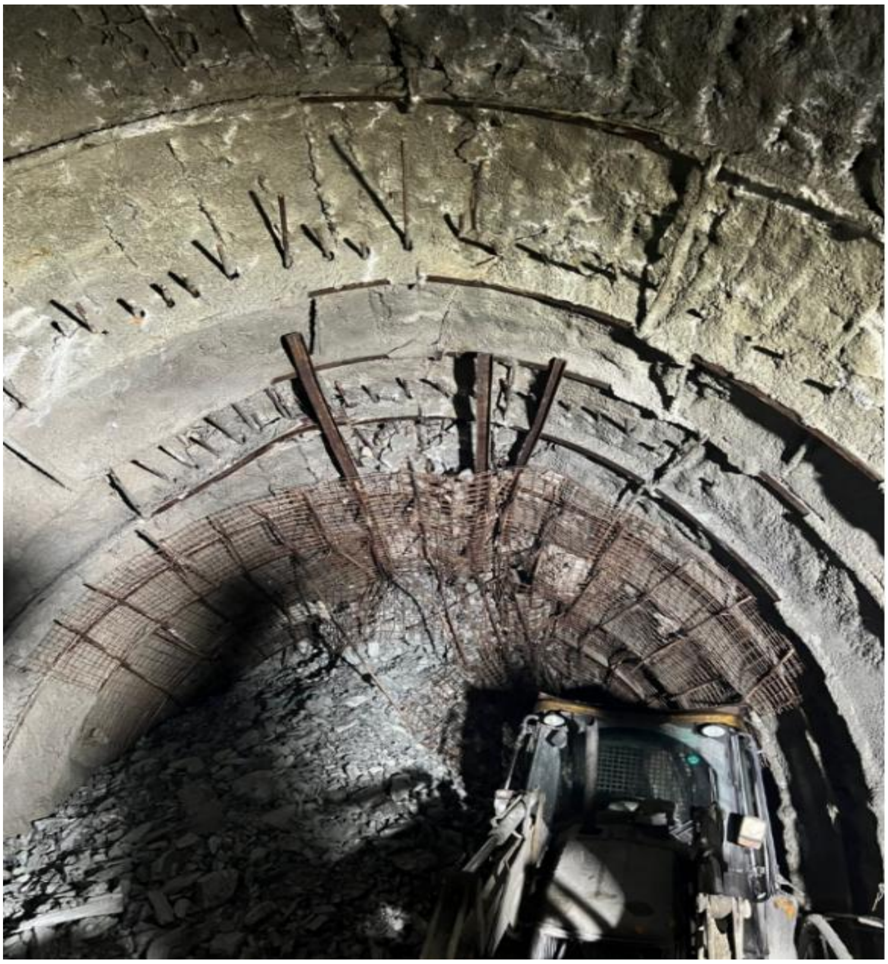


Figure 56 : Failure section of Tunnel

2.14 Power House

The semi-underground powerhouse of the Kabeli-A Hydropower Project will be constructed on the left bank of the Tamor River at Pinasi village of Amarpur VDC, just upstream of the confluence of Piple Khola with the Tamor River and along the bank of Piple Khola. The powerhouse is designed as a 60.91-meter-long, 13.90-meter-wide, and 12.15-meter-high structure, providing sufficient space to accommodate the turbine floor, generator floor, machine hall, service bay, control room, and other utility spaces. It will house three units of vertical axis Francis turbines, ensuring efficient power generation. Additionally, the powerhouse will be equipped with essential operational systems, including a drainage and dewatering system, cooling water system, compressed air system, oil handling system, ventilation system, fire protection system, elevator, and land workshop equipment. These facilities will support the seamless operation and maintenance of the power plant, enhancing its overall efficiency and reliability.

Figure 57 : Power House

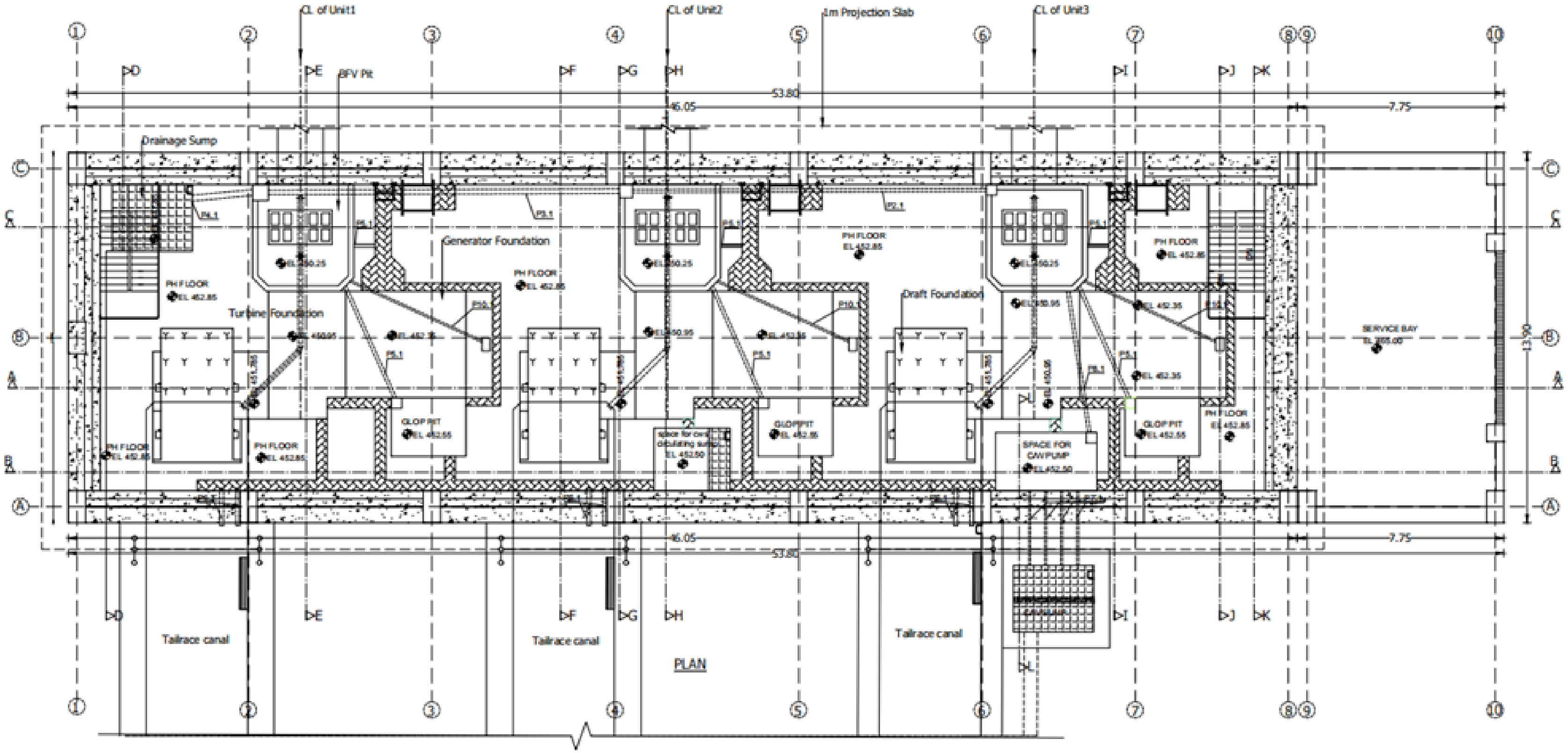


Figure 58 : Plan of Power House

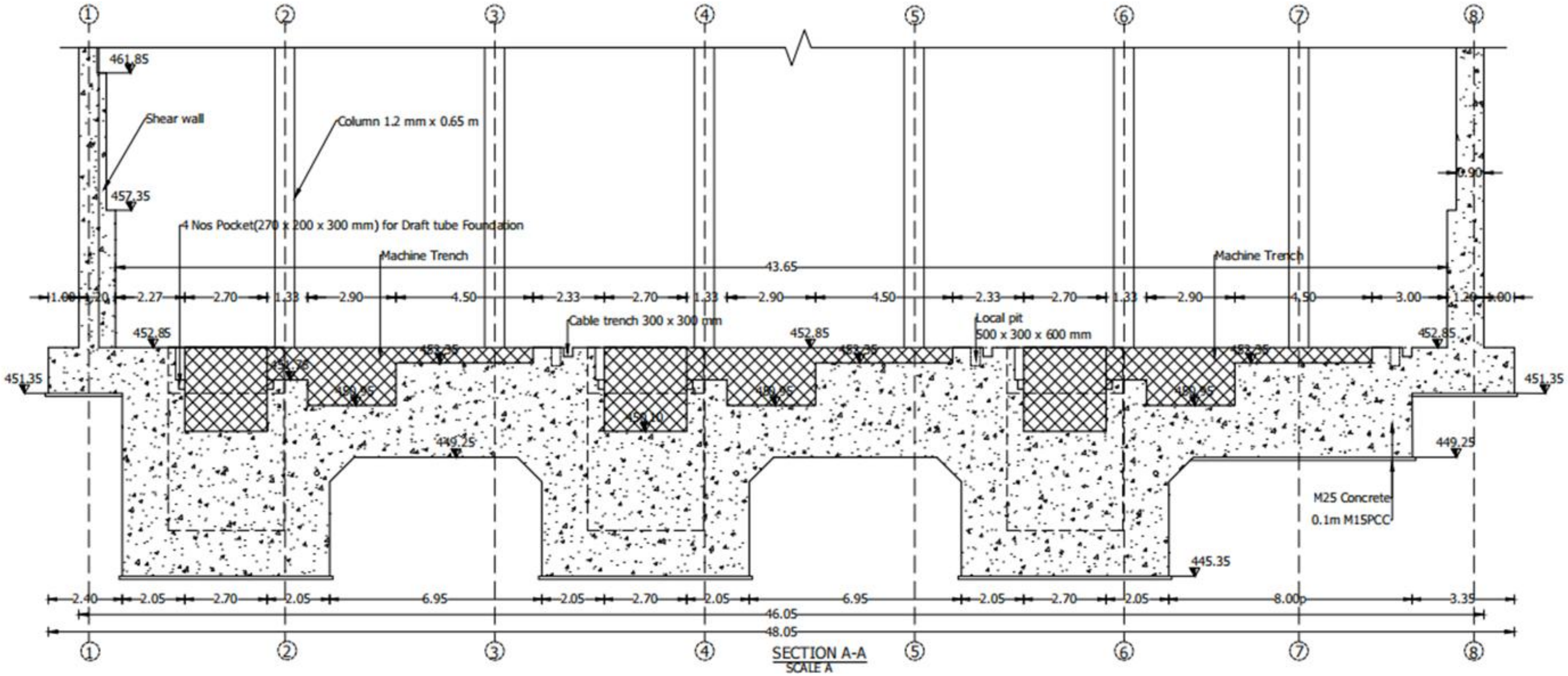


Figure 59 : Section A-A of Powerhouse

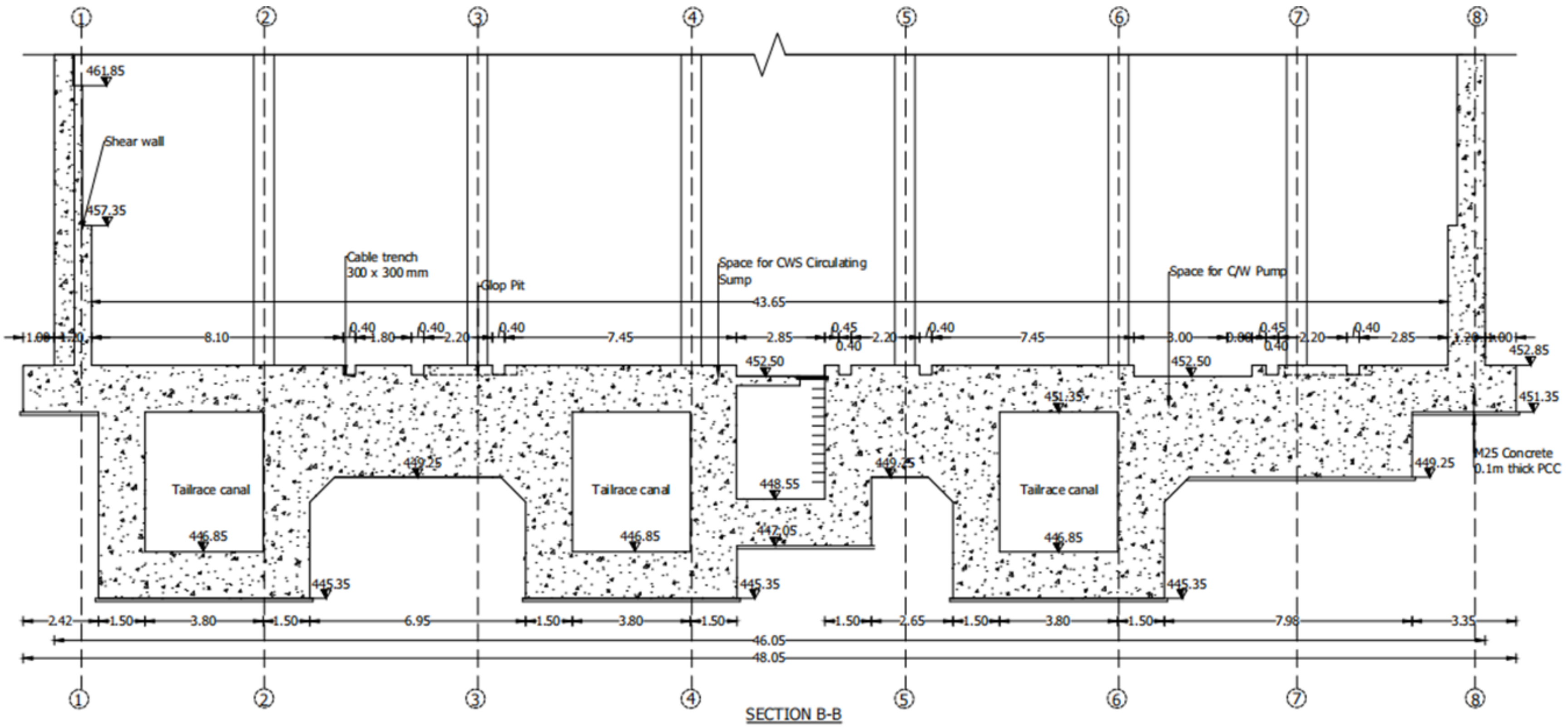


Figure 60 : Section B-B Of Powerhouse

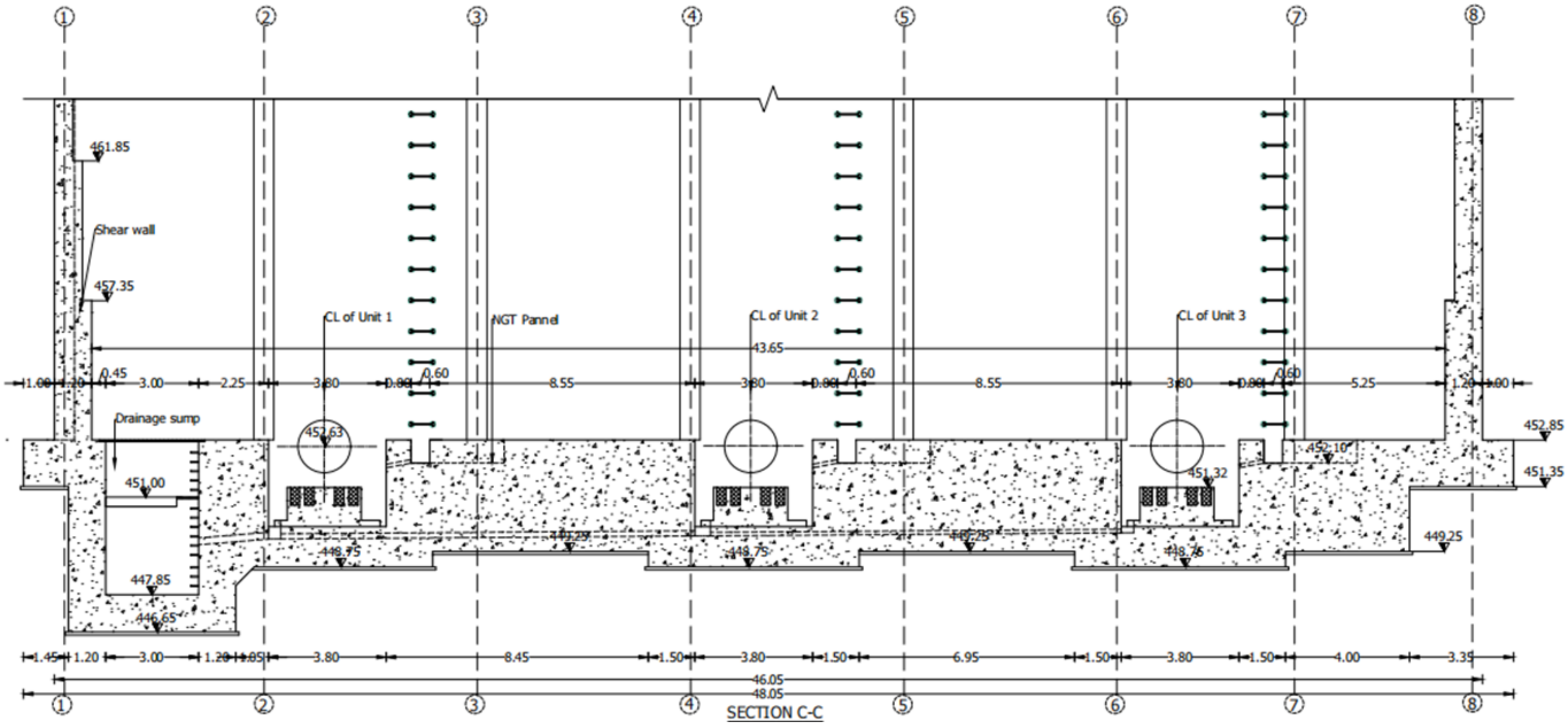


Figure 61 : Section C-C Of Powerhouse

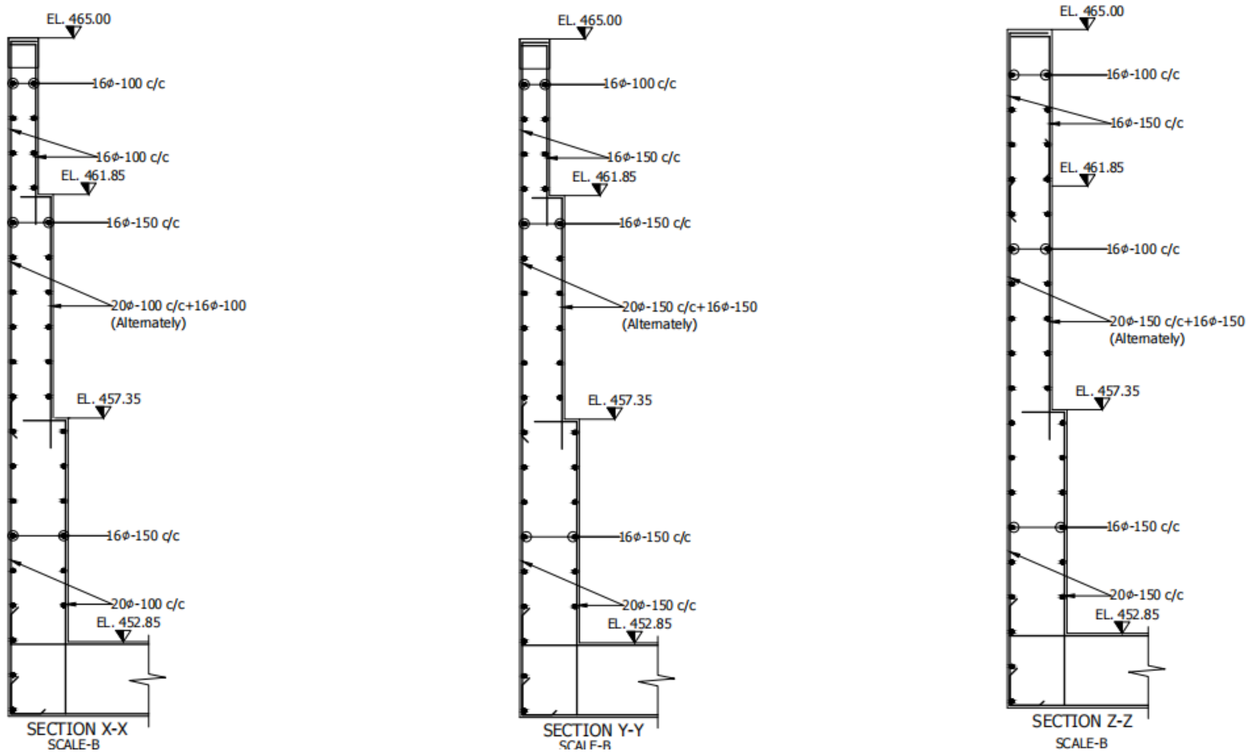


Figure 62 : Figure Section Elevation of Shear Wall

2.15 Draft Tube and Tailrace Canal

The tailrace system of the Kabeli-A Hydropower Project is a crucial component designed to ensure the smooth discharge of water after power generation. The powerhouse contains three separate tailrace units, each measuring 17.60 meters in length, which ultimately merge into a single tailrace channel with a total length of 74 meters, a width of 6.7 meters, and a height of 4.4 meters. The draft tube, featuring a circular inlet and a rectangular outlet, facilitates the proper flow of water into the tailrace by reducing velocity and improving hydraulic efficiency. The tailrace channel is designed as a non-pressure ‘closed conduit channel’ flow, ensuring controlled water conveyance to the Tamor River. A bed slope of 1 in 1500 is provided to maintain a stable flow depth at full discharge. At the end of each tailrace unit, flap gates are installed to protect the turbines and draft tubes from grit and debris that could enter due to backwater effects when the power plant is shut down. These gates also enable maintenance by isolating specific sections of the tailrace as needed. The tailrace system plays a vital role in maintaining turbine efficiency, preventing structural damage, and ensuring sustainable hydropower operations.

2.15.1 Tailrace Structure and Function

Tailrace Units:

- The powerhouse consists of three individual tailrace units of 17.60 meters each.
- These units handle the outflow of water from the turbines and lead it into the combined tailrace.
- The tailrace maintains water pondage between RL 452.18 msl and 446.85 msl to regulate the water level.

2.15.2 Draft Tube & Francis Turbine Efficiency:

- The draft tube of the Francis turbine is submerged in tailwater at the turbine exit.
- It helps in reducing the velocity of discharged water, converting kinetic energy into pressure energy, thereby improving turbine efficiency.

2.15.3 Heat Exchanger System:

- A heat exchanger is installed within the pondage water to assist in cooling and maintaining optimal operating conditions.
- Final Tailrace Section:
- The three tailrace units merge into a single tailrace with a total length of 74 meters.
- The final tailrace structure ensures smooth water discharge into the river.

2.15.4 Backflow Prevention:

- A gate is installed at the end of each unit of the tailrace to prevent backflow of water into the powerhouse during fluctuations in river levels or extreme conditions.

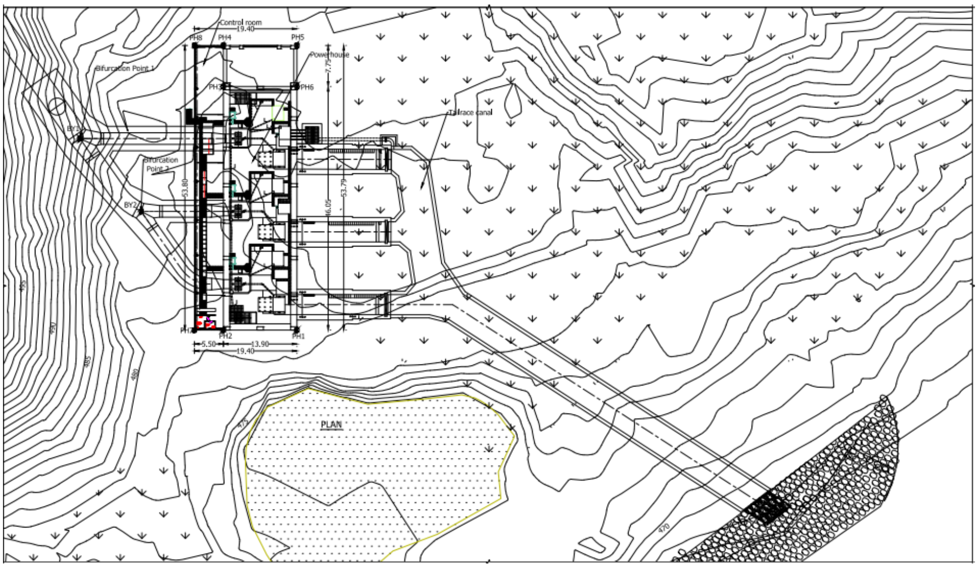


Figure 63 : Plan of Tailrace

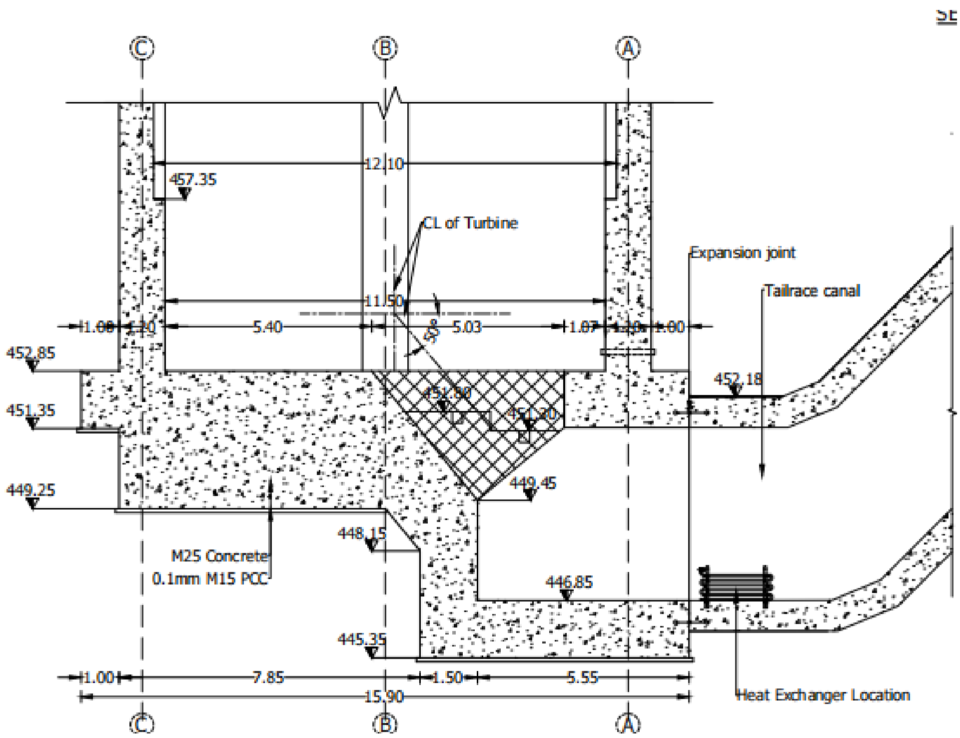


Figure 64 : Sectional view of Tailrace

2.16 Switchyard

The switchyard is located above the powerhouse at an elevation of 477masl, covering a total area of 54.74 meters by 34.43 meters. It will be responsible for stepping up the generated electricity to the required voltage level for transmission and distribution. The switchyard will be equipped with transformers, circuit breakers, busbars, and other electrical components necessary for ensuring efficient and safe power evacuation. Its strategic location close to the powerhouse will minimize transmission losses and improve the overall reliability of the power system.

2.16.1 Functions of Switchyard

- Voltage Transformation – Steps up generated voltage for efficient transmission.
- Power Distribution & Transmission – Directs electricity to transmission lines and substations.
- Fault Protection & Isolation – Uses circuit breakers and relays to detect and isolate faults.
- Load Management & Stability – Maintains voltage levels and grid stability.
- Synchronization of Power Generation – Ensures smooth integration with the national grid.
- Monitoring & Control – Enables real-time power system monitoring and operation

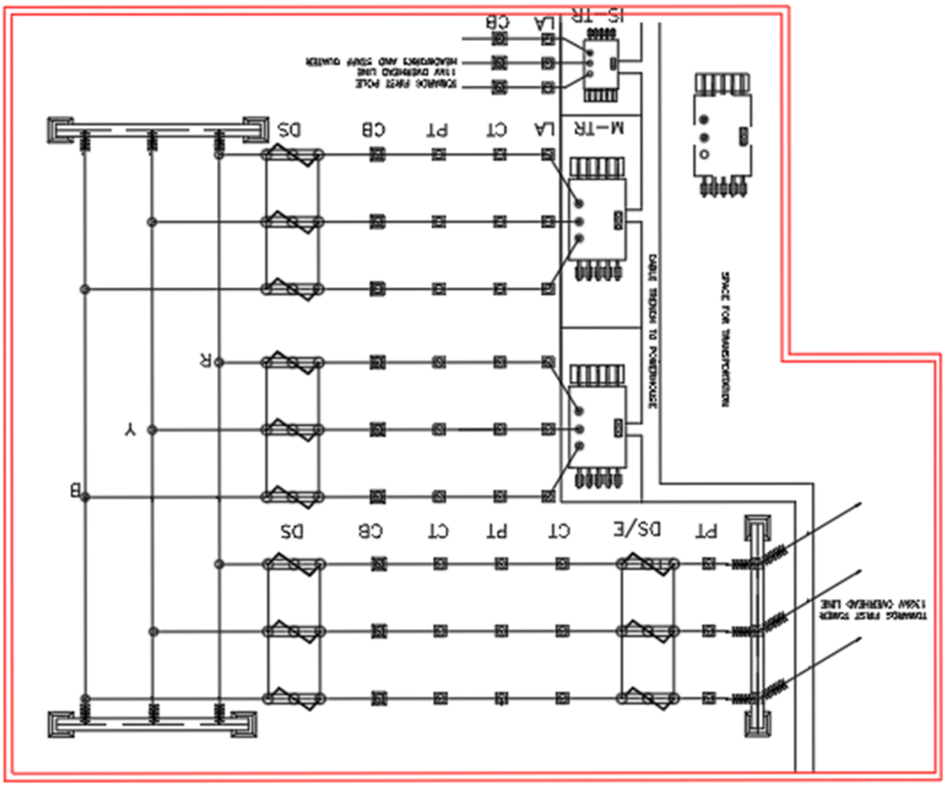


Figure 65 : Switchyard

2.17 Project supporting Facilities

2.17.1 Access Road

Access Road to Headwork

The proposed access road alignment to the headworks of the Kabeli-A Hydropower Project starts from Dhubichaur, which is located approximately 50 km from Phidim along the Mechi Highway. The road follows a hilly terrain with a total length of 7.4 km, consisting of multiple bends. It is currently a community-developed motorable earthen track, running along the edge of a forested area. Before the commencement of construction works, this road will be upgraded to ensure better accessibility for transportation and project execution.

Access Road to Powerhouse

The selected access road alignment (Alternative II) to the powerhouse of the Kabeli-A Hydropower Project is a community-developed motorable earthen track that starts from Bhanuchowk, approximately 48 km from Phidim along the Mechi Highway. The road extends for 15 km to the powerhouse at Pinasi. To ensure smooth transportation and construction activities, this road will be upgraded before the initiation of works.

2.17.2 Construction Power

For construction power, diesel generators with capacities of 450 kVA, 425 kVA, and 550 kVA is deployed at key locations, including the powerhouse audit, tunnel intake audit, and headworks area. These generator sets are installed and operated following best standard practices to ensure safety, efficiency, and environmental compliance. Measures are taken to prevent electrical shocks, fuel leakages, and noise pollution, ensuring a secure and reliable power supply throughout the construction phase.

2.17.3 Quarry Site

The construction materials such as sand, aggregates, and boulders required for the Kabeli-A Hydropower Project come from the Tamor and Kabeli River flood plains. For the headworks area, three locations along the Kabeli River provide construction aggregates, with a total estimated production capacity of 426,000 m^3, including 164,700 m^3 of boulders, 171,400 m^3 of cobbles, and 25,000 m^3 of sand, which meets the headworks' aggregate requirements.

For the powerhouse site, two quarry sites are identified along the Tamor River flood plain. The primary site, located on the left bank of the Tamor River, has an estimated aggregate production potential of 190,000 m^3, comprising 104,500 m^3 of boulders, 57,000 m^3 of cobbles, and 28,500 m^3 of sand. The right bank site serves as an optional source and will only be used if tunnel spoil, which is expected to provide 60% of the required aggregates, does not meet the project’s aggregate quality standards.

Since the proposed quarry sites lie within river flood plain areas, trenching operations for material extraction are strictly prohibited. These restrictions are included in bidding documents and contracts. Quarrying follows a stripping operation method to ensure that the landscape remains unchanged, although the land level will be altered. Additionally, excavation for aggregates occurs only up to the river's water level to minimize environmental impacts.

Figure 66 : Quarry Site

2.18 Technology Used

2.18.1 Batching Plant

A batching plant is specialized construction equipment that mixes coarse aggregates, sand, cement, and other admixtures to produce consistent concrete batches. It is made up of bins, conveyors, hopper and weighing equipment designed to precisely measure the components used in each concrete batch. Single batching plant is placed throughout the project. The batching plant for the kabeli project is located in the headwork site. The batch size in batching plant is $0.5m^3$, and the ingredient proportions are determined in the batching plant in line with the project's authorized concrete mix design.

Figure 67:Batching Plant

Components of a Batching Plant:

Aggregate Bins – Stores sand, gravel, and crushed stones.

Cement Silo – Stores cement and releases it in controlled amounts.

Weighing System – Ensures accurate proportions of each material.

Conveyor Belt – Transports aggregates and cement to the mixing unit.

Mixing Unit – Blends all components to form concrete.

Control Panel – Monitors and regulates the entire process.

2.18.2 Crusher Plant

A crusher is used to crush and screen big rocks into smaller sizes or shapes, such as particular size aggregates or finely textured sand. Two stationary crushers were strategically placed throughout the Kabeli project, one near the powerhouse, another near the headworks, to ensure adequate material supply. These crushers were responsible for generating fine and coarse aggregates ranging in sizes less than 4.75 mm, 10-20mm, and 20-30mm.

Figure 68 : Crushing Plant

2.18.3 RMC (Ready Mix Concrete) Mixer Truck

RMC Mixer Truck is used to carry the concrete mix from the batching plant to any location that requires concreting. The concrete mix remains fresh and is ready to be used in this vehicle. The vehicle which is used in the site has a capacity of 6 m^3.

Figure 69 : Mixer

2.18.4 SLM (Self-loading mixture) Mixture Truck

Self-Loading Mixer (SLM) Truck is a type of mobile concrete mixer that can self-load, mix, transport, and discharge concrete on-site. It combines a batching plant and transit mixer in one vehicle, making it ideal for small to medium construction site . The vehicle which is used in the site has a capacity of 4 m^3.

Figure 70 : SLM

2.18.5 Tunneling Jumbos (Boomer)

The Tunneling Jumbo machine is a robust face drilling rig for tunneling and mining applications with a coverage area of up to 65m². The BUT 36S booms and side platforms on both sides of the operator station make it possible to swing the feed to a position where the operator can safely load bolts without going in front of the machine. When you need high-performance drilling, you can rely on the Boomers to safely increase production.

Figure 71 : Boomer

2.18.6 Jack hammer

A jackhammer, also known as a pneumatic drill, is a handheld tool used in construction to break up hard materials like concrete, asphalt, and rock. It operates by delivering rapid hammering and chiseling actions, powered by compressed air, electricity, or hydraulics. Jackhammers are essential for tasks such as roadwork, demolition, and excavation, enabling efficient dismantling of tough surfaces.

Figure 72 :Jack Hammer

2.18.7 Shotcrete Machine

A shotcrete machine is a construction device used to spray concrete or mortar onto surfaces, creating a robust and durable layer without the need for traditional formwork. In kabeli, we used a wet mix while shotcrete. There are two primary application methods:

1.wet mix

2. dry mix

Figure 73 : Shotcrete Machine

2.18.8 Tunnel Ventilation Duct

A tunnel ventilation duct is a component in underground infrastructure designed to manage air circulation and maintain a safe environment within tunnels. These ducts facilitate the exchange of air, helping to remove pollutants, control temperature, and ensure proper ventilation for both vehicular and pedestrian tunnels. The efficient functioning of these ducts is essential for ensuring a safe and comfortable atmosphere within tunnels used for transportation or various industrial purposes.

Figure 74 : Tunnel ventilation Duct

2.18.9 Concrete Pump

A concrete pump is a piece of machinery used on building sites to pump liquid concrete. Concrete pumps assist to place concrete faster, more correctly, and with less labor than any other way. Concrete pumps are the greatest devices to use for speedier concrete placement on building sites

Figure 75 : Concrete Pump

2.18.10 Steel Bar Bending Machine

The steel bar bending machine is used in the project to bend steel bars with precision and efficiency. It features a sturdy frame with bending rollers and a motorized mechanism that applies force to bend the steel bars to the desired shape. They are essential for creating curved or angled reinforcement bars used in different structures of the hydropower with minimal effort.

Figure 76 : Steel Bar Bending Machine

2.18.11 Bar Cutting machine

Bar cutting machine is used in construction to cut steel bars (rebars) accurately. It operates with hydraulic or mechanical power, cutting bars of various sizes efficiently. It improves work speed, reduces labor, and ensures precise cuts. Essential for large projects, it comes in portable and heavy-duty models.

2.18.12 Grouting Machine

A grouting machine is an equipment designed for pressure-injecting grout into grout holes. It is primarily utilized for pressure grouting, a process involving the injection of grout material into interconnected pore or void spaces where the configuration or volume is unknown. This process is commonly referred to as grouting. Grout materials may include cementitious, resinous, or chemical mixtures. There were cement grouting machine in the headworks and powerhouse area.

Figure 77 : Grouting Machine

2.18.13 Dump Truck

It is a large truck for transporting heavy loads with a back part that can be raised at one end so that its contents fall out. The mostly used 20-ton dumpers and 10 tons in your site. In Kabeli - A The dumper transported the muck from the tunnel areas to the disposal area and also carried the big boulders, stones, aggregate, and soils from one site.

Figure 78 : Dump Truck

2.18.14 Backhoe

It is a mechanical excavator having a bucket that is attached to a rigid bar hinged to a boom and that is drawn toward the machine in operation. struck bucket capacity 0.38 (back) to 2(front) m³, the backhoe was mainly used for trench cutting, filling, and loading .

Figure 79 : Backhoe

2.18.15 Front Loader

A Front loader is a machinery used in mining and construction activities to efficiently load loose materials such as soil, gravel, or ore onto trucks. The muck from drilling and blasting in the tunnel face is loaded by muck loader into the trucks and transported outside the tunnel.

Figure 80 : Front loader

2.18.16 Water Pump

Water pumps are a common fixture on construction sites, tasked with removing excess water caused by heavy rain or higher water tables. They serve two primary functions in the site: preventing water from infiltrating foundations, tunnels, and excavation pits, and supplying water for various tasks such as curing.

Figure 81 : Water Pump

2.18.17 Air Compressor

An air compressor is a machine powered by an electric motor or gas engine that draws in air from the atmosphere, compresses it in a confined space to increase its pressure, and then transfers the high-pressure air to a receiver tank. The compressed air is then used to clean the area before concreting. Moreover, it has been used to run different machines and testing purpose.

Figure 82 : Air Compressor

2.18.18 Crawler Excavator

Excavators are pieces of construction machinery that consist of a boom, arm, bucket, and cab mounted on a rotating superstructure that runs along tracks. They are generally used for excavation, lifting, and moving materials in a wide range of applications

Figure 83 : Crawler Excavator

2.18.19 Needle Vibrator

A needle vibrator, also known as an internal vibrator or poker vibrator, is a construction tool used to compact concrete. It consists of a vibrating head (needle), a flexible shaft, and a motor. The needle is inserted into freshly poured concrete to remove trapped air and ensure proper compaction, which improves the strength and durability of the structure.

Components of a Needle Vibrator:

Vibrating Head (Needle) – The metal cylindrical part that is immersed in the concrete.

Flexible Shaft – Connects the vibrating head to the motor and transmits vibrations.

Motor – Provides the power to generate vibrations (electric, petrol, or diesel-powered).

Working Principle:

- When the motor is turned on, it creates high-frequency vibrations.
- These vibrations are transmitted through the shaft to the needle.
- The vibrating needle is inserted into the concrete, causing air bubbles to escape.
- This process compacts the concrete by reducing voids and increasing density.

Advantages of Using a Needle Vibrator:

- Eliminates air pockets in the concrete.
- Improves the strength and durability of concrete.
- Ensures a smooth and even finish.
- Reduces segregation and bleeding of concrete.
- Enhances the bonding between concrete and reinforcement.

Applications:

- Used in beams, columns, slabs, and foundations where proper compaction is essential.
- Commonly used in high-rise buildings, bridges, and dams.

Figure 84 : Needle Vibrator

3. Details Of Internship Project Activities

With approval from API Power Company Limited, we arrived at the Kabeli-A Hydropower Project site in Pachthar on February 23. Upon arrival, we had a brief discussion with Supervisor, Basant Chaudhary and senior surveyor Lakpa Sherpa, who provided an overview of the project and our roles as interns. Kabeli A is a 37.6 MW run-of-river hydropower project designed to utilize the flow of the Kabeli River for electricity generation. The project consists of key components such as a diversion weir, intake structure, settling basin, headrace tunnel, surge tank, penstock, powerhouse, and tailrace canal.

During the internship, we focused on understanding the design, construction, and operation of these components. We assisted in site surveys, inspecting structural works, and observing tunnel excavation using the drill and blast method. We also studied the power intake system, which regulates water flow into the headrace tunnel, and gained insights into the role of the surge tank in managing pressure fluctuations. We consulted with geotechnical, mechanical, electrical, civil and survey engineers involved in the project. Our duties were not only limited to the observation, inspection and supervision of the construction works at the site but also included the preparation of bar bending schedules, performing flakiness and elongation test of aggregates, calibration of batching plant and lab test of concrete.

Figure 86: Power House

Figure 85: Interconnection Chamber

3.1 Observation of Work Progress in All Components

The site visit to the Kabeli-A Hydropower Project in Pachthar was an invaluable experience, providing a comprehensive insight into the ongoing construction activities at the 37.6 MW project site, we had the opportunity to observe various construction processes firsthand. Under the guidance of our supervisor Lakpa Sherpa, we explored different work areas and analyzed the construction techniques being implemented to ensure project efficiency and structural integrity. One of the primary activities observed was concrete pouring, which was being carried out in several structural components, including the powerhouse foundation and retaining walls. Strict quality control measures were followed, ensuring proper mixing, transportation, and placement of concrete. Vibrators were used to remove air pockets and achieve compaction, while curing methods were implemented to prevent cracking and enhance durability. Additionally, we observed plum concreting, a technique used in mass concrete works where large stones were embedded in concrete to reduce material usage while maintaining structural stability. Another critical aspect of the visit was inspecting rebar detailing, where reinforcement bars were carefully placed and tied as per design specifications. This process was crucial for ensuring the structural strength of various elements, such as columns, slabs, and beams. The workers followed bending schedules, ensuring proper overlap lengths, spacing, and alignment according to engineering drawings. The excavation work for different structures, including the powerhouse and penstock trench, was also in progress. Heavy machinery such as excavators and dumpers were actively engaged in earth-cutting, rock blasting, and slope stabilization. At the tunnel inlet, excavation activities were carried out with safety precautions, including rock bolting and shotcrete application to stabilize the rock face and prevent collapses.

3.2 Observation & Inspection Before Concreting

Before the actual concreting process begins, a thorough inspection and observation are essential to ensure the structural integrity and quality of the construction work. During our site visit to the different structures of the project, we had the opportunity to observe and inspect various preparatory activities before concrete pouring. Under the supervision of our supervisor, we analyzed key aspects that needed to be checked to ensure a successful concreting process. One of the primary observations was the formwork preparation, which plays a critical role in shaping the concrete structure. The formworks were inspected for proper alignment, stability, and cleanliness. Any gaps or loose connections in the shuttering were sealed to prevent leakage of cement slurry during pouring. The supports and bracings were checked to withstand the load of fresh concrete without displacement or deformation. The reinforcement detailing was another crucial aspect of inspection. The placement of rebar was checked against the structural drawings, ensuring proper spacing, overlap length, and cover distance. Spacer blocks and chairs were placed correctly to maintain adequate concrete cover, which is essential to protect the reinforcement from corrosion and improve durability. Any debris, dust, or loose materials inside the formwork were removed to prevent contamination of concrete. The surface was moistened before pouring to avoid excessive absorption of water from the concrete mix, which could lead to early drying and reduced strength.

Figure 87:Desander Basin

Figure 88: Penstock Unit-1

3.3 Supervision During Concreting Works

We had the opportunity to closely observe the concreting process under the guidance of our supervisor. The experience provided us with valuable insights into the careful monitoring and control measures required during concrete placement. One of the key aspects of supervision was ensuring the proper pouring sequence and uniform placement of concrete. The concrete was poured in layers, and care was taken to avoid segregation of aggregates. The pouring was done continuously to prevent cold joints, which could weaken the structure. Special attention was given to complex sections, ensuring proper flow and compaction of concrete. Vibration and compaction were critical aspects of supervision. Mechanical vibrators were used to eliminate air voids and achieve proper consolidation of the concrete. The vibrators were carefully handled to avoid over-vibration, which could cause segregation, and under-vibration, which could lead to honeycombing. The team ensured that vibrators were applied systematically throughout the poured area, especially around rebar and formwork edges.

Figure 89 : Concreting of Settling Basin

Figure 90 :Base Concreting of Transmission Line

3.4 Quality Assurance

One of the primary quality assurance measures was verifying the concrete mix design before pouring. The concrete mix was carefully prepared as per the design specifications, ensuring the correct proportions of cement, aggregates, water, and admixtures. Regular slump tests were performed to check the workability of the mix and ensure consistency across different batches. If the slump value deviated from the required range, necessary adjustments were made to maintain the desired mix properties. One of the primary quality assurance measures was verifying the concrete mix design before pouring. The concrete mix was carefully prepared as per the design specifications, ensuring the correct proportions of cement, aggregates, water, and admixtures. Regular slump tests were performed to check the workability of the mix and ensure consistency across different batches. If the slump value deviated from the required range, necessary adjustments were made to maintain the desired mix properties. During concreting, compaction and vibration techniques were carefully monitored. Mechanical vibrators were used to eliminate air voids and ensure proper bonding between aggregates and cement paste. we inspected if the sets of cubes are casted and sent to the laboratory for compressive strength testing. These measures were all part of our efforts to uphold high standards of quality in the construction process.

3.5 Observation of Lab Tests

Under the supervision of Quality Control Engineer, we observed various lab tests essential for ensuring construction material integrity. Concrete cube tests were conducted to verify compressive strength, elongation and flakiness tests on aggregate samples to assess the particle shape and size distribution which is crucial for concrete workability and durability. Slump tests were also determined for the concrete's consistency and workability, with precise measurements of slump height and visual inspection of the concrete's behavior.

3.5.1 Test Performed on Concrete

3.5.1.1 Slump Test

This test helps determine the consistency of concrete by measuring how thick or fluid it is, as well as checking the water-to-cement ratio. It is conducted immediately after mixing the concrete. In this test, concrete is placed into a cone-shaped mold, which has a narrow top opening of 100 mm, a wider bottom opening of 200 mm, and a total height of 305 mm. The concrete is added in 3 to 4 layers, with each layer being compacted by patting it down 25 times. Once the mold is completely filled, the cone is carefully lifted, allowing the concrete to settle under its own weight. The amount by which the concrete slumps or spreads is then measured. If the concrete collapses or slumps excessively, it indicates that the mix has too much water, and adjustments are needed to achieve the right consistency. This test ensures that the concrete mix is suitable for construction, providing the necessary strength and durability.

Figure 91 : Slump Cone Test

3.5.1.2 Compressive Strength Test

The compression test is conducted using a Universal Testing Machine (UTM) to determine concrete's resistance to compressive forces. At hydropower sites, standard 150mm concrete cubes are carefully positioned in the UTM between two steel plates. The machine then applies a gradually increasing load until the specimen fails. The maximum load at failure, divided by the cross-sectional area of the cube, gives the compressive strength in MPa or N/mm². This crucial test is performed at 7 and 28 days after casting to track strength development. Testing at hydropower sites presents unique challenges due to remote locations, extreme environmental conditions, and the massive concrete structures involved. Quality control engineers document each test, recording the mix design, casting date, curing conditions, and any visible failure patterns. Regular testing ensures that concrete used in critical structures like dams, spillways, and powerhouses meets the required design specifications, depending on structural requirements. This verification is essential for the long-term safety and integrity of hydropower infrastructure that must withstand enormous water pressures and environmental stresses for decades.

Figure 92:Universal Testing Machine

Figure 93:Test Report

3.5.2 Test Performed on Penstock Pipe.

3.5.2.1 Dye Penetration Test

Dye Penetration Testing (DPT) is a widely used non-destructive testing method for detecting surface-breaking discontinuities in welded joints and connections. The process begins with thorough surface cleaning, followed by application of a bright red penetrant liquid via spraying, brushing, or immersion, with the test surface temperature maintained between 5°C to 52°C. During the critical dwell period (5-60 minutes depending on material and expected flaw characteristics), capillary action draws the penetrant into any surface defects. Excess penetrant is then carefully removed using lint-free cloths and appropriate cleaners, striking a balance between removing surface penetrant while preserving penetrant in defects. A white developer is applied in a thin, even coat, which draws the trapped penetrant from discontinuities to create visible red indications against the white background. After allowing 10-30 minutes for indications to fully develop, inspection is conducted under adequate lighting, with technicians evaluating the size, shape, and distribution of indications to characterize flaws according to acceptance. This method effectively reveals cracks, porosity, lack of fusion, and other surface defects that would otherwise remain invisible to the naked eye, making it invaluable for quality control in critical infrastructure including hydropower facilities.

Figure 94 : Dia Penetration Test

3.5.2.2 Ultrasonic Test

Ultrasonic Testing (UT) stands as a crucial non-destructive testing methodology for evaluating penstock pipe integrity in hydropower installations. The process utilizes specialized transducers that emit high-frequency sound waves into the pipe material, which then travel through the metal until encountering boundaries or discontinuities. These discontinuities including cracks, laminations, wall thinning, or weld defects cause the sound waves to reflect back to the transducer at different velocities and amplitudes. Technicians apply a coupling medium (usually gel or oil) between the transducer and pipe surface to eliminate air gaps that would impede sound transmission. For penstock inspections, UT is particularly valuable for measuring remaining wall thickness in areas vulnerable to erosion or corrosion, and for examining critical welds that must withstand extreme water pressures. The technique offers distinct advantages over radiographic testing at hydropower facilities, including absence of radiation hazards, portability for accessing difficult locations, immediate results for quick decision-making, and the ability to detect planar defects that might be missed by other methods. Technicians follow standardized procedures to ensure consistent interpretation, with results documented through detailed reports showing defect locations, sizes, and assessments against acceptance criteria, ultimately ensuring the safe and reliable operation of these critical pressure components.

Figure 95 : Ultrasonic Test

3.5.2.3 Calibration of batching plant

The calibration of a batching plant isn't just technical maintenance it's a crucial art that determines construction success. Picture it as fine-tuning a precision instrument where every component aggregates, cement, water, and chemical admixtures must be measured with perfect accuracy. This process directly impacts the concrete's quality, much like how precisely following a recipe affects a dish's outcome. When calibration is spot-on, the concrete meets exact design specifications for strength and durability, but even small measuring errors can compromise structural integrity and safety. Beyond quality concerns, proper calibration significantly reduces waste and cuts costs. What many overlook is the need for regular recalibration since environmental conditions, mechanical wear, and sensor drift gradually affect accuracy. Thorough documentation of these calibration processes isn't just paperwork. It provides essential traceability and regulatory compliance, ultimately protecting both the concrete supplier's reputation and the buildings that will stand for decades.

3.6 Observation of IPC Of the Project

We reviewed the Interim Payment Certificates (IPCs) of the project, not for verification purposes but to understand their preparation process and contents. The IPCs detailed the quantities of work completed and were backed by essential documents such as drawings, Bar Bending Schedules (BBS), and other relevant records. This observation provided valuable insights into how running bills are prepared by the contractor, the documentation required to support them, and the systematic process involved in project billing and financial tracking. Additionally, it helped us understand the importance of accurate record-keeping and compliance with contractual agreements in construction projects.

3.7 Study of Bar Bending Schedule

For the various structures at the project site, we carefully reviewed the Issued for Construction (IFC) drawings and prepared the Bar Bending Schedules (BBS) accordingly. This included elements like the interconnection chamber, canal, syphon canal, head pond, columns, beams, flood Walls, anchor Blocks, tailrace, and the Powerhouse Control Room. In preparing the BBS, we focused on ensuring the reinforcements matched the required specifications for shape, size, and placement while also ensuring they were easy to work with on site. We also aimed to reduce waste by reusing any extra rebar. Throughout the process, we remained adaptable, making necessary adjustments based on site conditions, which helped us optimize material use and maintain efficiency in construction.

The prepared bar bending schedules were also used as the medium for quantity calculation of Steel used for various structures.

Table 13: Bar Bending Schedule

Section LL-MM

Section length=60.41m

IPC: 13

QCS Sheet: 0005

SN	Member	Bar mark	Size	No. of member	Spacing c/c (mm)	Bars No.	Total no.	Length of each bar	Shape code	A *	B *	C *	D *	E*	E/R *	Total Weight	Shape	Remarks
										mm	mm	mm	mm	mm	mm	kgs		
A.	**Base slab**																	
	Main Bar																	
	Base Slab (Bottom Bar)	01	16	1	150	404	404	5276	21	140	5060	140				3368.303	A C B	
	Base Slab (Top Bar)	08	16	1	150	404	404	6596	41	440	360	5060	360	440		4211.017	C B D A E	
	Top slab (bottom bar)		12	1	150	404	404	5292	21	140	5060	140				1900.416	A C B	
	Top slab(top bar)		12	1	150	404	404	6212	41	240	360	5060	360	240		2230.798	C B D A E	
	Wall Vertical (Outer)	02	16	2	150	404	808	6596	21	800	5060	800				8422.033	A C B	
	Wall Vertical (Inner)	04	16	2	150	404	808	5876	21	440	5060	440				7502.709	A C B	
	Bottom and top Hunch Bar	03	16	4	150	404	1616	2100	25	300	1500	300				5362.726		
	Distribution Bar																	
	Outer Side bottom slab and top slab	09a	12	2	150	36	72	62760	1	62760						4016.640	A	09a+09b+09C lap
	Outer Side wall Complete	09b	12	2	150	33	66	62760	1	62760						3681.920	A	
	Inner Side bottom slab and top slab	09a	12	2	150	30	60	62760	1	62760						3347.200	D C A B A	
	Inner Side wall Complete	09b	12	2	150	28	56	62760	1	62760						3124.053	A	
	Distribution Bar on hunch bar		12	4	150	9	36	62724	21	62760						2007.168	A	
	Chair Bars uptodate		16	6	1500	13	78	1572	4	300	300	500	300	300		193.764	C B A D E	

3.7 Observation of Grouting in Headrace Tunnel

During our internship on the headrace tunnel construction, we had the opportunity to observe the detailed grouting process, which was key to ensuring the tunnel's stability and preventing water leakage. The grouting process included both contact and consolidation grouting, with contact grouting filling the space between the concrete lining and the surrounding rock to create a watertight seal, while consolidation grouting strengthened the rock mass by injecting grout into cracks and voids. Our responsibilities included monitoring the grout mix preparation, checking the consistency, and overseeing the injection pressures and flow rates to ensure proper grout distribution. Additionally, we kept a close eye on the concrete placement and curing process, documenting any inconsistencies or deviations from the planned specifications and ensuring corrective actions were taken. This hands-on experience gave us a deeper understanding of the importance of proper grouting and concrete application in maintaining the structural integrity and long-term safety of the tunnel.

3.8 Fabrication and Quality Testing of Penstock Pipes

In the workshop, we had the chance to observe the fabrication of penstock pipes, where attention to detail was key in every step, from cutting and welding to shaping the steel plates. The goal was to ensure everything aligned perfectly and maintained structural integrity. After the pipes were fabricated, several non-destructive tests were conducted to check their quality. Dye Penetrant Testing (DPT) was used to find any surface cracks or imperfections by applying a colored dye and developer. X-ray radiography helped inspect the internal welds, revealing any flaws like porosity or inclusions beneath the surface. Additionally, Ultrasonic Testing (UT) was carried out to measure the thickness and overall integrity of the pipe walls, ensuring that no internal defects were present and that the welds were properly fused. Throughout this process, we carefully documented the test results and any defects that were found, making sure everything met the project specifications and safety requirements.

Figure 96 : Fabrication of penstock Pipe

3.9 Office work

After returning from the Kabeli-A hydropower site after almost one month, we began a two-week office work session at Butwal Power Company, located at Trade Tower. During this period, we were involved in the following tasks:

- Survey data analysis for the hydropower layout
- Preparation of as-built drawings
- Drawing and drafting tasks
- Calculation of water level in the surge tank
- Turbine selection analysis
- Total energy calculation
- Preparation of the internship report
- Preparation of the project presentation
- Preparation of the academic poster

Throughout the office work period, we became more familiar with drawing and drafting practices. Additionally, we had the opportunity to connect with the office staff and interact with high-level professionals from our field, which was a valuable learning experience.

3.10 Challenges in our Internship

Steep Learning Curve

We were exposed to complex, real-world engineering concepts that were new to us, making it difficult to apply theoretical knowledge.

Hands-on Work vs. Theory

Transitioning from academic learning to practical were challenging, as we need to develop skills in tasks like welding or grouting, which require precision.

Communication and Coordination Challenges

Working with diverse teams and managing expectations, especially in technical settings, made communication difficult for us.

Site Conditions

Harsh weather, long hours, and working in remote locations were physical and logistical challenges for us.

Adapting to a Professional Environment

Adjusting to the professional workplace culture, managing relationships, and understanding company dynamics were difficult for us transitioning from academia.

3.11 Cost Estimate

3.11.1 Civil Works

The total estimated cost for civil work including general items and contractor's mobilization and demobilization, explosive management, surface structure works, underground structure work, alignment works in headrace canal, in penstock pipe, powerhouse-tailrace canal and switchyard work is NRs 3,059,330,618.9, without including contingencies and tax. The overall cost including contingencies and tax is NRs 3,647,655,684.7 from which NRs 189,492,767.8 is contingencies amount and NRs 419,641,804.4 is for tax.

3.11.2 Electro-Mechanical Equipment

The total estimated cost for Electro-Mechanical works including Electro-mechanical Equipment and its Accessories @245 per kW including interconnection facilities, LC Charge, Energy Meter and Other Charge is NRs 1,213,060,000.0 without vat and custom duty.

3.11.3 Hydro mechanical works

The total estimated cost for Hydro mechanical works including Price schedule for Trash-rack, Gates and Hoisting Systems, Price schedule for Pressure Penstock erection, painting and testing and Pipe Accessories, Miscellaneous items Spare Parts, Price Schedule for spiral pipe procurement including transportation upto site is NRs 177,161,524.7 without vat custom duty.

3.11.4 Transmission Line and Interconnection

The total estimated cost for Transmission Line and Interconnection including Loop in Loop out with 132 kv transmission line, Land Procurement + Others and Construction power is NRs 100,000,000.0 excluding vat and custom duty.

3.11.5 Contingencies

Provision for contingencies has been made as per CWC guidelines in each project components.

3.11.5.1 Office and camping facility

The total estimated cost for Office and camping facility including Office equipment and camping facility is NRs 40,000,000.0 without vat and custom duty.

3.11.5.2 Office equipment and furniture

The total estimated cost for office equipment and furniture is NRs 40,000,000.0 without vat and custom duty.

3.11.5.3 Vehicle

The total estimated cost for vehicle is NRs 40,000,000.0 excluding vat and custom duty.

3.11.5.4 Infrastructure Development/ Access Road

The total estimated cost for Infrastructure Development including Access Road Construction and Regular Maintenance is NRs 160,000,000.0 excluding vat and custom duty.

3.11.5.5 Environmental Mitigation and Social Contribution

The total estimated cost for Environmental Mitigation and Social Contribution including Land and Environmental Mitigation Cost is NRs 155,000,000.0 excluding vat and custom duty.

3.11.5.6 Project Management and Supervision

The total estimated cost for Project Management and Supervision including Office Running Cost, Consultancy Fees and Charges is NRs 216,000,000.0 without vat and custom duty.

3.11.5.7 Miscellaneous Expenses and Insurance

The cost for miscellaneous and insurance is estimated to be NRs 57,500,000.00

The total cost without IDC (Interest During Construction) is NRs 5658052143.64 excluding vat and custom duty. Cost of interest during Construction and Project Financing is NRs 955,501,661.4.

The grand total cost of the project is estimated to NRs 7,540,593,969.9 including vat, contingencies and IDC, so cost for per mw generation is NRs 200,547,712.0.

Table 14: Cost Estimation

S.N.	Description	Amount (NPR)	Contingency (%)	Contingency Amount (NPR)	Tax/VAT (NPR)	Total Amount (NPR)	Remarks
A	Pre-Operating	400,000,000.00	-	-	-	400,000,000.00	-
B	Civil Construction Works	3,059,330,618.90	5-7%	189,492,767.80	419,641,804.40	3,647,655,684.70	Includes various structures and protection works
C	Hydromechanical Works	177,161,524.70	5%	8,858,076.20	16,702,295.50	202,721,896.50	Includes gates, penstock erection, and spare parts
D	Electromechanical Works	1,213,060,000.00	5%	60,653,000.00	19,105,695.00	1,292,818,695.00	Includes turbines, generators, and interconnection facilities

E	Transmission Line & Interconnection	100,000,000.00	3%	3,000,000.00	13,390,000.00	116,390,000.00	Includes land procurement & power connection
F	Office & Camping Facility	40,000,000.00	5%	2,000,000.00	5,460,000.00	47,460,000.00	-
G	Office Equipment & Furniture	40,000,000.00	5%	2,000,000.00	5,460,000.00	47,460,000.00	-
H	Vehicles	40,000,000.00	5%	2,000,000.00	5,460,000.00	47,460,000.00	-
I	Infrastructure Development & Access Road	160,000,000.00	5%	8,000,000.00	21,840,000.00	189,840,000.00	Includes road construction & maintenance
J	Environmental Mitigation & Social Contribution	155,000,000.00	5%	7,750,000.00	21,157,500.00	183,907,500.00	-
K	Project Management & Supervision	216,000,000.00	5%	10,800,000.00	29,484,000.00	256,284,000.00	Includes office running cost & consultancy

L	Miscellaneous Expenses & Insurance	57,500,000.00	5%	2,875,000.00	7,848,750.00	68,223,750.00	-
M	Price Contingency (1.5% of Total Cost)	-	-	84,870,782.20	-	84,870,782.20	-
N	Interest During Construction & Project Financing	955,501,661.40	-	-	-	955,501,661.40	-
Grand Total Cost with IDC	-	-	-	-	-	7,540,593,969.90	-
Project Cost per MW	-	-	-	-	-	200,547,712.00	

Conclusion

Our internship at Kabel-A Hydropower Project has been an enriching and transformative experience that significantly enhanced our academic knowledge and technical skills. Throughout the internship period, we were actively involved in various aspects of construction of hydropower components, including concreting, testing, analysis of BBS and Surveying. Working closely with experienced engineers and technicians, we gained hands-on exposure to real-world engineering challenges and solutions, particularly in the areas of turbine assembly, generator alignment, penstock installation, and transmission systems.

This opportunity not only deepened our understanding of hydropower systems but also helped us develop essential soft skills such as communication, teamwork, and problem-solving in a professional environment. Observing safety practices and project management workflows on-site provided valuable insights into how large-scale energy projects are executed efficiently and responsibly.

Overall, this internship served as a crucial bridge between theoretical learning and practical application, preparing us for future roles in the field of Hydropower engineering. We are grateful to the Kabel-A Hydropower Project team for their guidance and support, and we look forward to applying these experiences in our future careers.

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Annex

Figure 97: Interconnection chamber


Figure 98: Desander Basin

Figure 99: Approach Cannel

Figure 100: Reinforcement Placement

Figure 101:Headrace Tunnel

Figure 102:Surge Shaft

Figure 103:Component of Hydropower

Figure 104: Saddle Support

Figure 105: Bifurcation

Figure 107: Landslide

Figure 106: Headpond

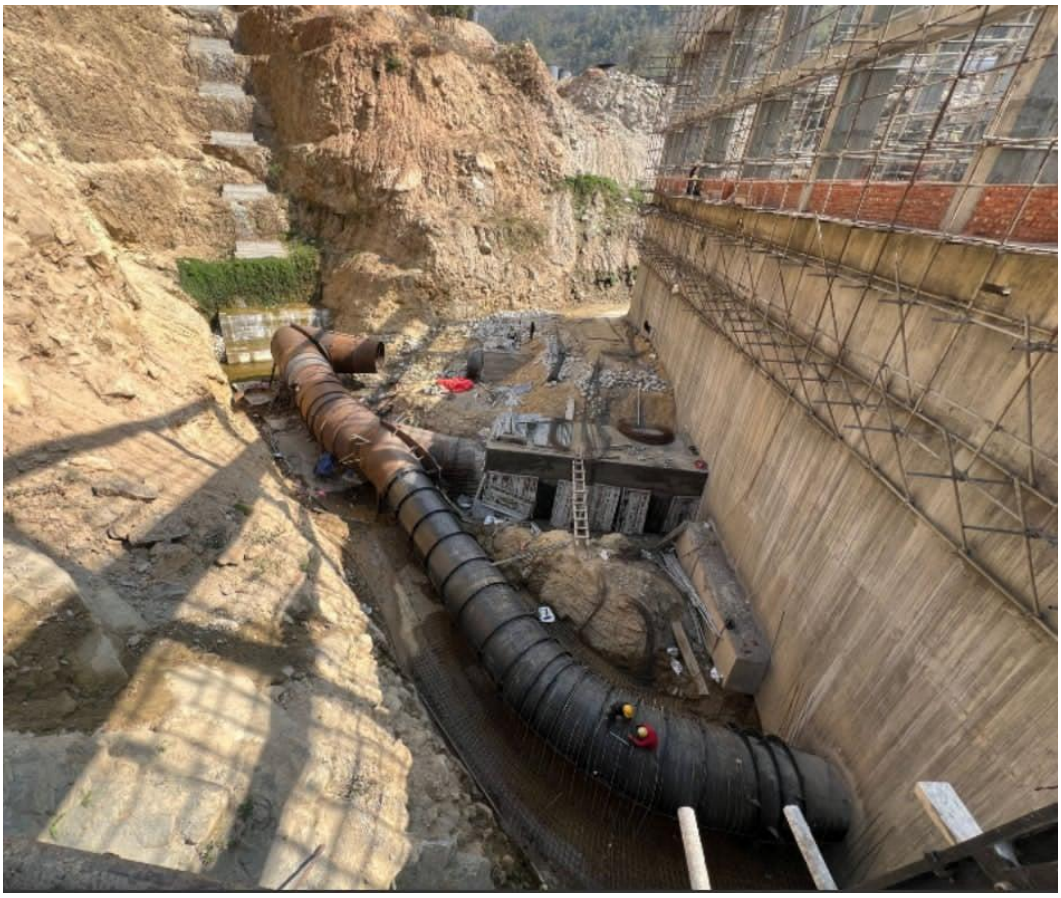

Figure 108: Bifurcation of Penstock

Figure 109: Farewell