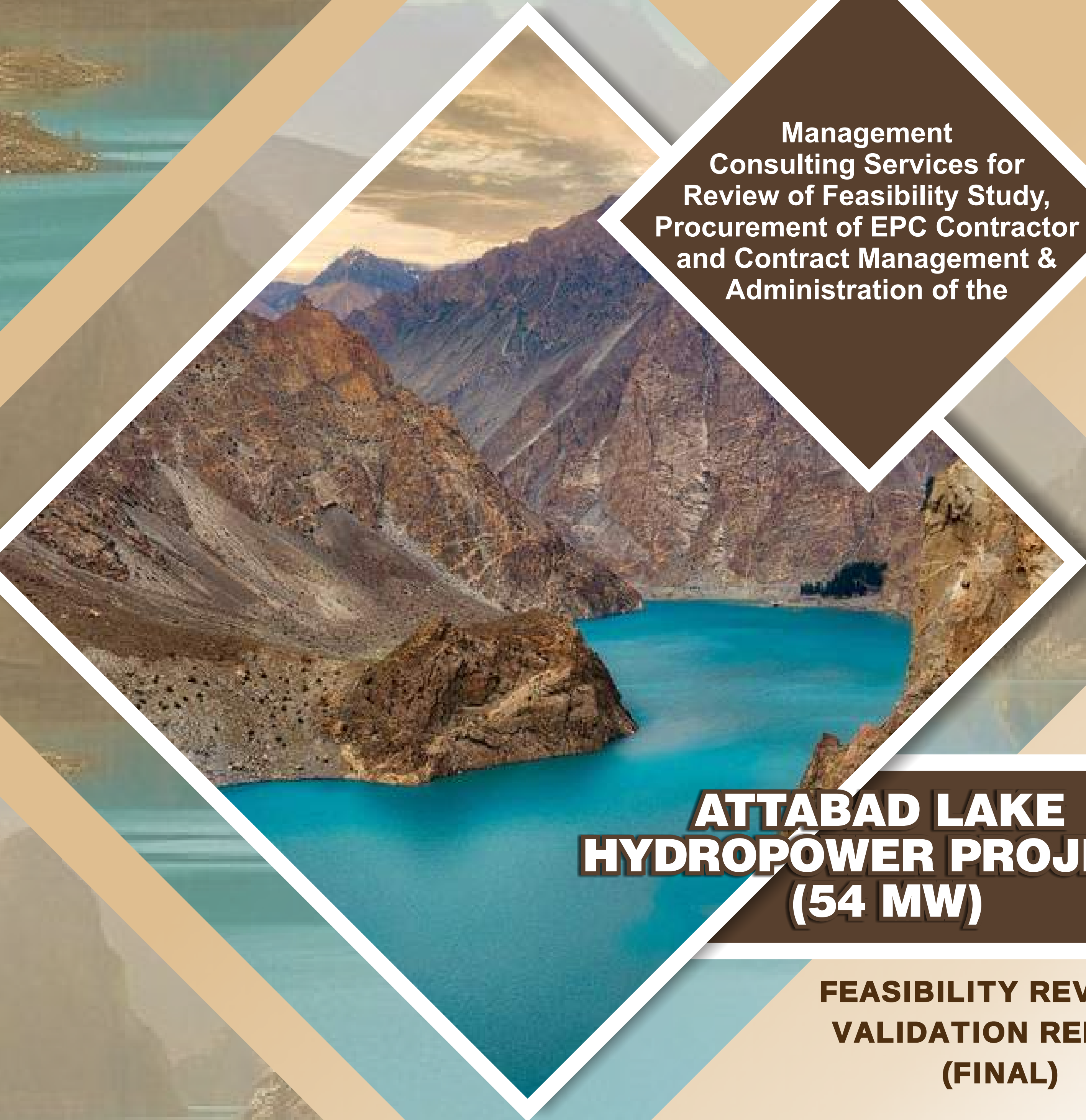
PAKISTAN
WATER AND POWER DEVELOPMENT AUTHORITY

Management Consulting Services for Review of Feasibility Study, Procurement of EPC Contractor and Contract Management & Administration of the



ATTABAD LAKE HYDROPOWER PROJECT (54 MW)

FEASIBILITY REVIEW / VALIDATION REPORT (FINAL)

DOCUMENT NO.: 4464-FRR

AHMC ATTABAD HYDROPOWER MANAGEMENT CONSULTANTS

FINAL FEASIBILITY REVIEW / VALIDATION REPORT

MAY 2023

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LIST OF ABBREVIATIONS

ACE	Associated Consulting Engineers
ADP	Annual Development Programme
ALHPP	Attabad Lake Hydropower Project
EPC	Engineering, Procurement, and Construction
FSR	Feasibility Study Report
GCEP	Generation Capacity Expansion Plan
GoGB	Government of Gilgit Baltistan
KKH	Karakoram Highway
KP	Khyber Pakhtunkhwa
MC	Management Consultants
MW	Mega Watts
NESPAK	National Engineering Services Pakistan
O&M	Operation and Maintenance
PES	Pakistan Engineering Services
RFP	Request for Proposal
WAPDA	Water and Power Development Authority
NEQS	National Environment Quality Standards
HSS	Hydraulic Steel Structure
EMMP	Environmental Management and Monitoring Plan
EIA	Environmental Impact Assessment
EDG	Emergency Diesel Generator
CMTL	Central Material Testing Laboratory
CCTV	Closed Circuit Television
UCS	Uniaxial Compressive Strength
RPM	Rotation per Minute
PCS	Plant Control System
ONAN	Oil Natural Air Natural
ONAF	Oil Natural Air Forced

1 INTRODUCTION

1.1 GENERAL

Gilgit-Baltistan of Pakistan is blessed with abundant renewable hydropower resources. More than 95% of Pakistan hydel potential is located in mountainous areas of Gilgit-Baltistan (GB), Khyber Pakhtunkhwa (KP) and Azad Jammu and Kashmir (AJK). The reconnaissance study of hydel potential for whole Gilgit-Baltistan is almost complete, and identified potential sites are still to be optimized in terms of capacity, cost and need of power for growing load centers.

Attabad Lake was created due to a massive land slide in January, 2010. In November, 2015, Prime Minister of Pakistan announced the construction of Attabad Lake Hydropower Project (ALHPP) and accordingly, Govt. of Gilgit-Baltistan undertook its Feasibility Study with proposed capacity of 32.5 MW.

Later, In August 2017, Ministry of Water and Power sought expert opinion from Chairman WAPDA on construction of ALHPP. WAPDA conducted Topographic surveys and Geotechnical Investigations for an updated study and proposed installed capacity to 54 MW with a 2320 m long headrace low pressure tunnel on the left side of Hunza River.

54 MW power generation was planned to meet the winter power demand of Hunza Valley by utilizing the peaking storage facility at existing Attabad Lake. As a result of the assessment of field investigations and engineering studies, it was concluded that the Project of 54 MW is technically, economically feasible and environmentally safe. The Project can further enhance due to available potential at the location of Hunza River once the regional and national grids are established.

The project is proposed to be implemented through Engineering, Procurement and Construction (EPC) mode for which services of Management Consultants (MC) have been acquired for the review of Feasibility Study, Procurement of EPC Contractor and Contract Management & Administration.

1.2 LOCATION

The most promising project layout, as assessed at Feasibility stage, was located at the Attabad Lake on the left bank of Hunza River at a distance of about 20 km from Karimabad, 120 km from Gilgit and 735 km from Islamabad.

The coordinates of the Weir and Power House site are given in the Table 1.1.

Table 1.1 Coordinates of Weir and Power House site (From Feasibility Report)

Overflow Weir/Ungated Spillway	484036.834 m E	4018143.837 m N	2414 m.a.s.l
Powerhouse	481645.920 m E	4017963.719 m N	2300 m.a.s.l

The Project is to be constructed on marginal lands owned by the community of Ahmadabad village, where minimum environmental and social issues have been anticipated.

The project location map and vicinity map are shown in Figure 1.1 & Figure 1.2 respectively.

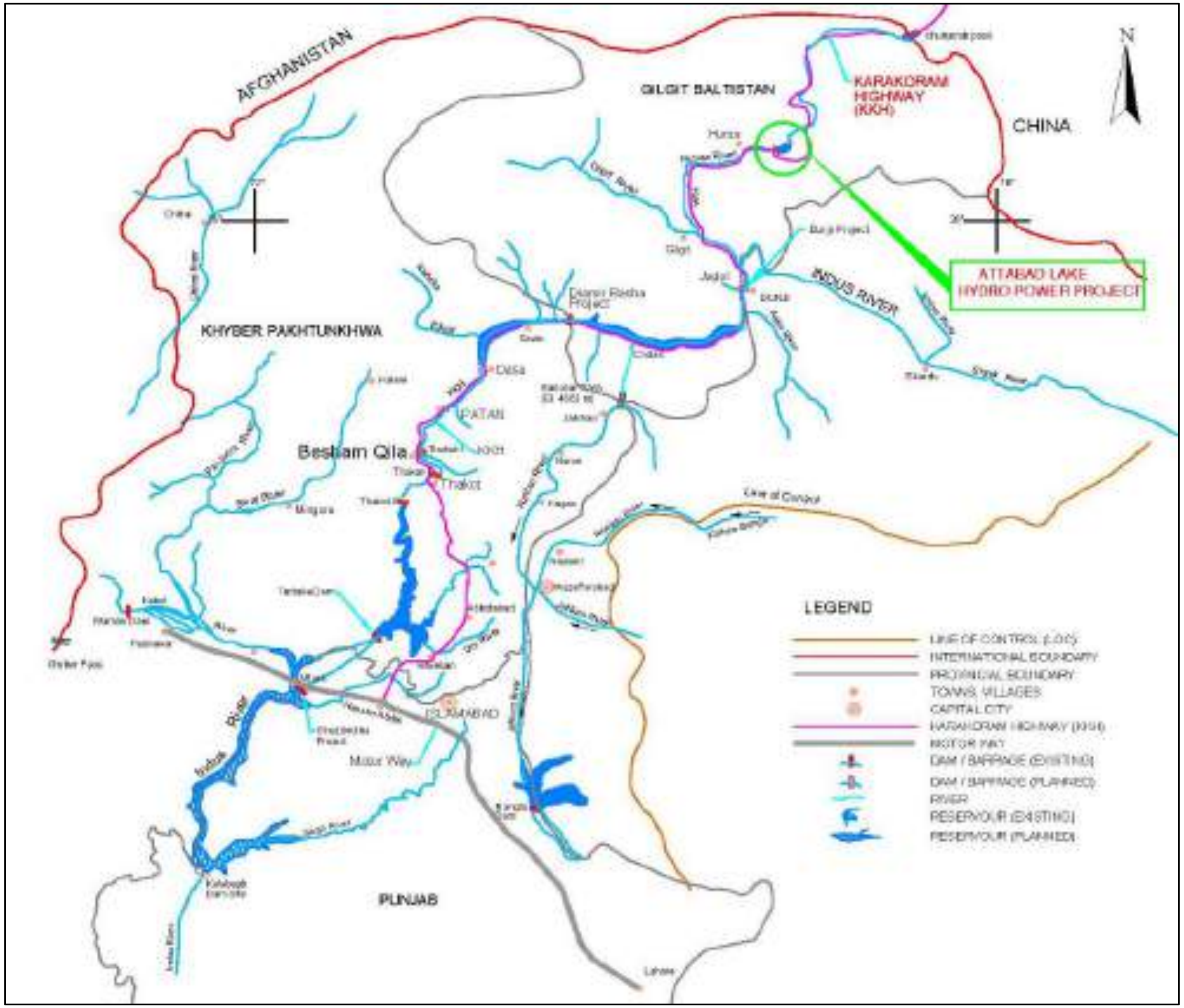


Figure1.1: Project Location Map

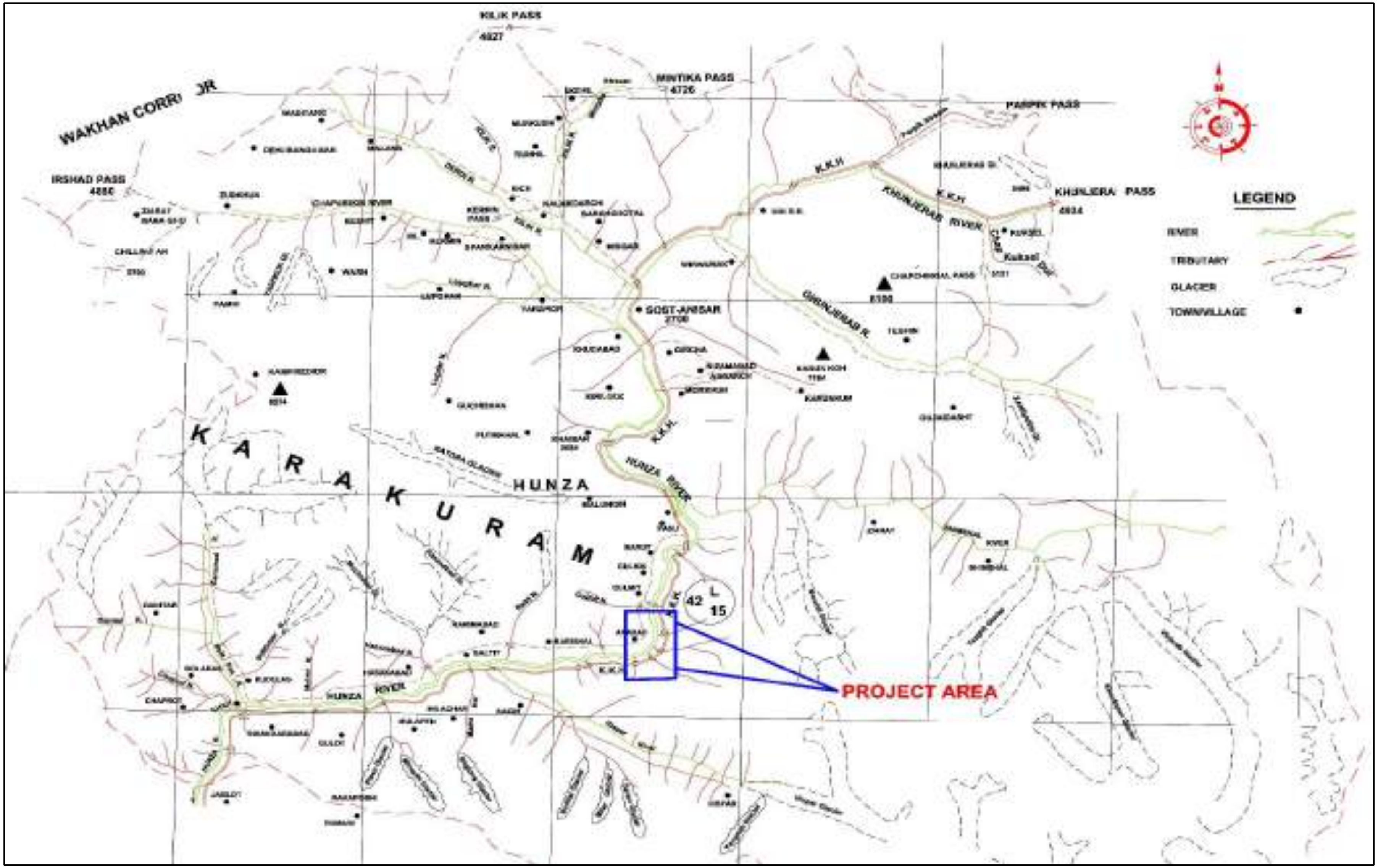


Figure 1.2: Project Vicinity Map

1.3 ACCESSIBILITY

The Project site is accessible from Federal Capital (Islamabad) connected to Gilgit along the existing N-35 Karakoram Highway (KKH), which leads to Project Area at Attabad Lake near Hunza. The route through KKH can be accessed via Motorways from Islamabad to Burhan Interchange near Hassanabdal (M-1 Islamabad-Peshawar Motorway-44 km) and from Burhan Interchange to Thakot (M-15 Hazara Motorway – 186 km). The access onward to Gilgit and Hunza continues as a part of KKH (Thakot to Gilgit– 390 km; Gilgit to Attabad Lake/ Hunza along Hunza River – 120 km). There is another route from Islamabad to Gilgit and Hunza via Babusar Pass, through National Highway (N-15). The route, although shorter than KKH through Besham, to reach up to Chilas, usually remains closed for six months in a year due to heavy snowfall during the winter season.

Gilgit Airport is the nearest air travel facility to the Project Area which is about 120 km from the Project Site. Domestic flights are regularly scheduled from Islamabad airport to Gilgit. However, flight operations largely depend upon the weather conditions. Project accessibility map is shown in Figure 1.3.

1.4 CLIMATE

The Gilgit Meteorological Station is the nearest weather station to Hunza Valley that has a sound historical record covering more than five decades. Monthly temperature variations at Gilgit indicate parabolic trend, with peak in the month of July. Summer prevails in months from June to August. In July and August, mean maximum and minimum temperatures are normally above 35˚C and 15˚C respectively. The winter is cold and chilly, starting from November and continuing till end of March. The lowest temperatures are in the month of January with mean maximum and minimum temperatures reaching below 10˚C and (-) 2˚C respectively.

1.5 PROJECT OBJECTIVES

The major objectives of the Project are following.

a. Meet the winter power demand of Hunza valley.
b. Ensure the socio-economic uplift of the locals.
c. Boost tourism in the area by uplifting power availability in the area.
d. Address the issue of pollution by increasing power capacity of the region and reduce the usage of industrial backup diesel generators.

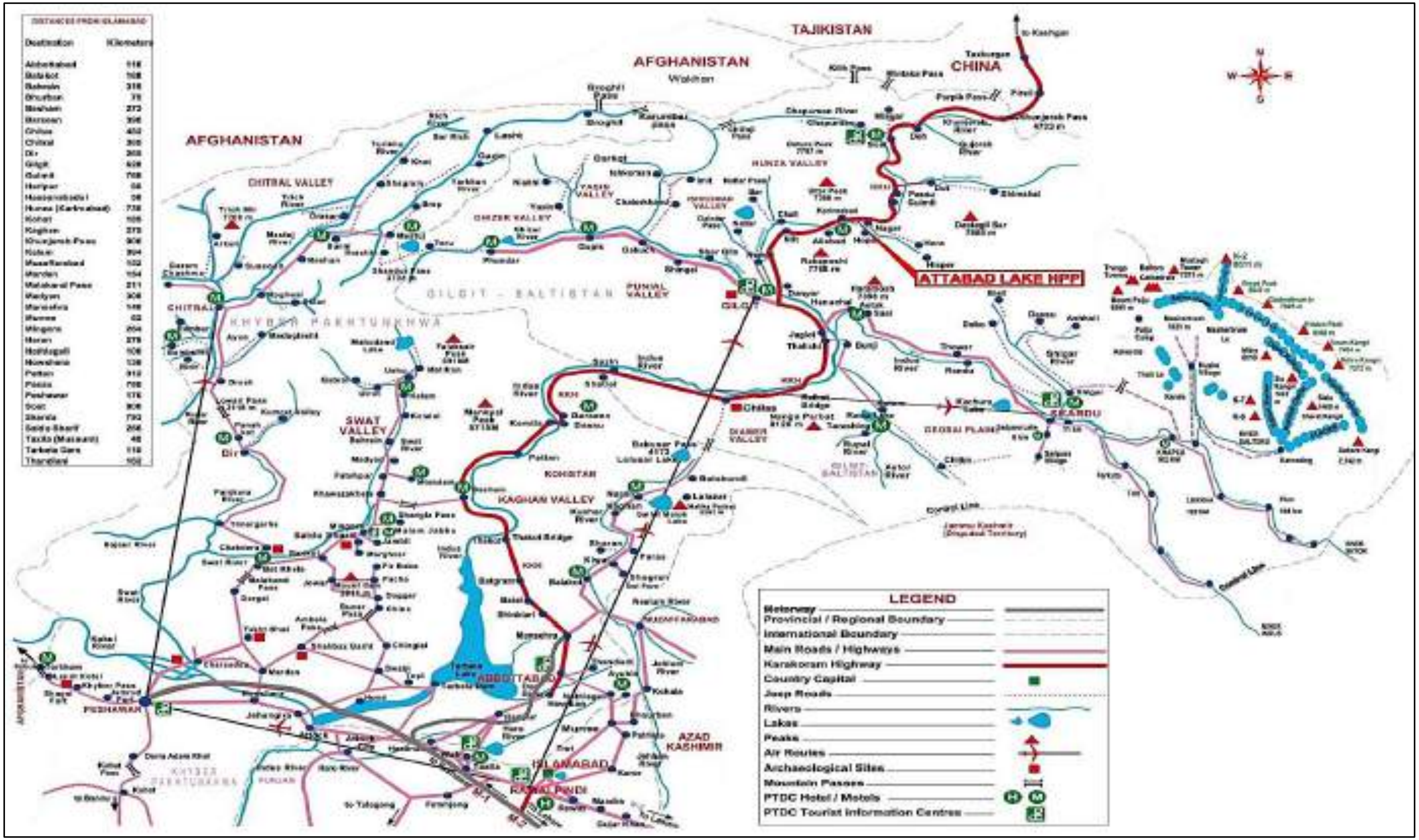


Figure 1.3: Project Accessibility Map

1.6 PREVIOUS STUDIES

Following studies have been previously carried out on Attabad Lake Hydropower Project:

a. Feasibility Study (conducted by local consultants, 2017)

Water and Power Department of Govt. of Gilgit Baltistan (GoGB) intended to utilize the storage of Attabad Lake as an opportunity to develop Hydropower Project of appropriate capacity to meet the ever increasing demand of Hunza City and its suburbs. In this regards, GoGB conducted a Feasibility study through local consultant of GB, Designmen Consulting Engineers (Pvt.) Ltd, Islamabad in association with Mountain Infrastructure & Engineering Services Gilgit for the power capacity of 32.5 MW.

The left bank of the Attabad Lake was used for the layout of the project where it comprised of intake with normal head pond level 2416 m.a.s.l., 2015.62 m long power tunnel, 340m long steel penstock, surface powerhouse and 255 m long tailrace with outlet into Hunza River.

b. Updated Feasibility Study (conducted by WAPDA, 2021)

WAPDA prepared a Feasibility Report by utilizing in-house technical expertise. The project was sized as per flows availability, size of conveyance system and suitability of surface powerhouse according to prevailing topographic and geological features. The plant size of 54 MW was planned to provide the installed power during peak hours throughout the years. A Weir was proposed to divert the flow of Hunza River into Head Race tunnel, 2300 m long nearby Karakoram Highway, then 450m long penstock bifurcating into three high pressure pipes through a manifold will lead to Power House. A deep flushing channel/ diversion/ under sluices structure was proposed to evacuate the water from lake on the left side of existing outlet and act as flushing channel for sediments from the lake once project comes into operation.

1.7 PROJECT COMPONENTS FROM FEASIBILITY STUDY BY WAPDA

Attabad Lake Hydropower Project comprises of the following components and Feasibility stage project layout is shown in Figure 1.4.

1. Natural dam of 45 m height from river bed to act as diversion structure with low level outlet for flushing and an overflow ungated spillway.
2. Power Intake structure, proposed on the left bank of river to take the design discharge of 60 m^3/s.
3. Low pressure headrace tunnel having diameter of 5.5 m with total length of 2320 m.
4. Vertical surge shaft of 10 m diameter with height of 49 m
5. Concrete lined pressure shaft of 3.6 m diameter having a length of 80 m.
6. Steel penstock having a length of 470 m and diameter of 3.5 m.
7. Surface powerhouse with three (03) Francis units with an installed capacity of 54 MW and open switchyard.
8. Tailrace channel of 18 m length.

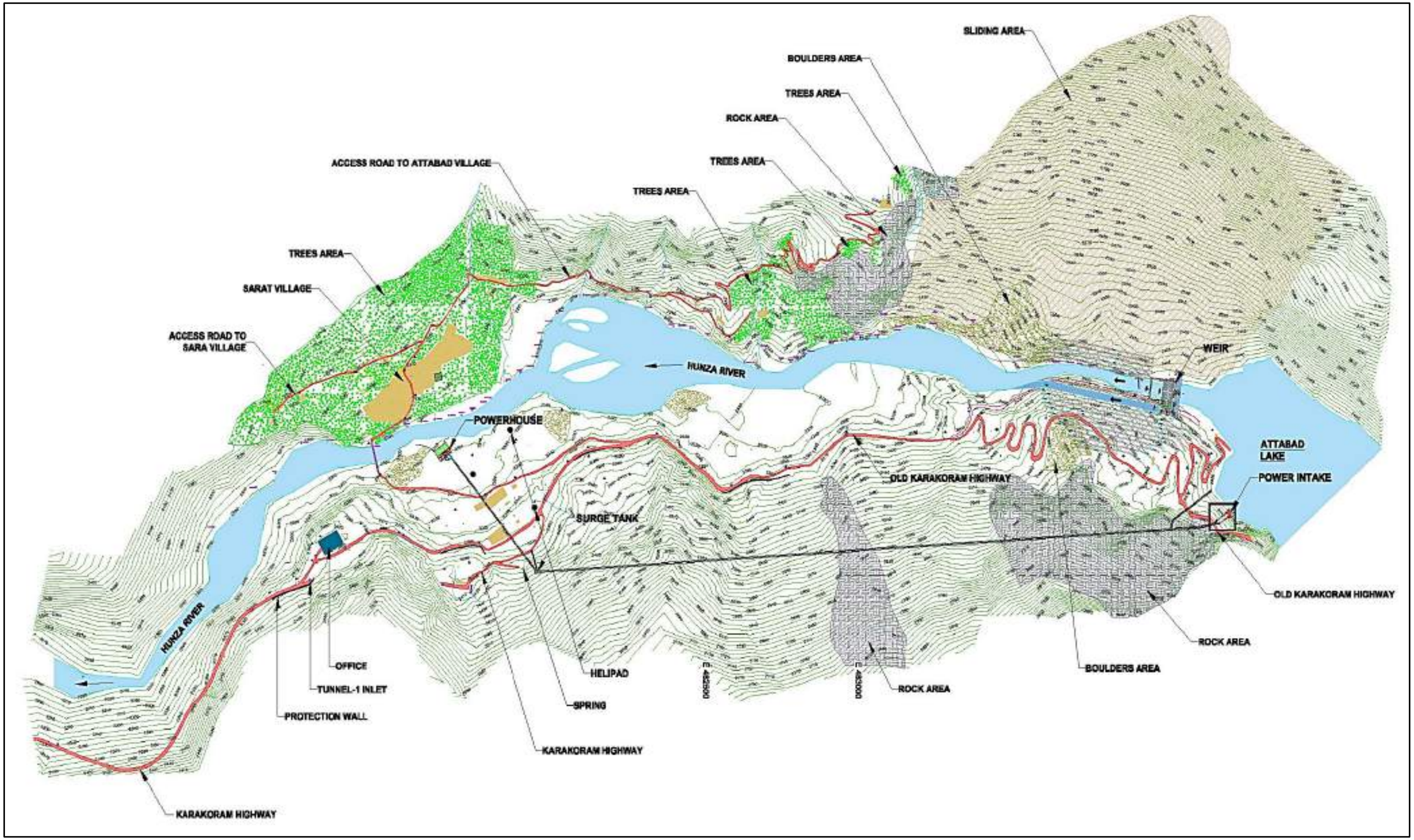


Figure 1.4: Feasibility Stage Project Layout

1.8 SALIENT FEATURES

Salient features of the project from Feasibility study are listed in Table 1.2.

Table 1.2 Project Salient Features (From Feasibility Report)

Location	Attabad village, Hunza District, Gilgit-Baltistan of Pakistan, 120 km west of Gilgit & 735 km north- of Islamabad	**Design Discharge** **Gross head** **Net head** **Installed Capacity** **Mean Annual Energy** **Plant Factor**	60 m^3/s 112.0 m 106.0 m 54 MW 373.69 GWh 79 %
River **Catchment Area** **Mean Annual Flow** **Mean Annual Vol.**	Hunza 8901 km^2 209 m^3/s 6591 M m^3	**Penstock** **Length** **Diameter** **Thickness**	1No. 470 m 3.5 m 15 mm
Weir **Crest elevation** **Crest length** **Weir height** **Design Flood** **Max. Flood level** **Sediment Flushing Channel** **Sill level** **Nos. and Gate size**	Overflow Weir/ Ungated Spillway 2414 m.a.s.l 60 m 30 m 3250 m^3/s (1:10,000 returned period) 2421 m.a.s.l Under Sluice/Diversion Channel 2402 m.a.s.l 3 & 5m x 5m	**Powerhouse** **Size** **Height** **Type of units** **No. of units** **Speed** **No. of Transformer**	External Slope type 45 m x 18 m 22 m Francis 3 375 rpm 3, 22 MVA
Attaabad Lake length **Lake width** **Lake depth**	14000 m 400 to 600 m 50 to 60 m	**Tailrace** **Turbine level** **Max. Flood level**	18 m, concrete lined 2300.0 m.a.s.l 2307.0 m.a.s.l
Tunnel length **Diameter of Tunnel** **Typel**	2320 m 5.5 m Horse shoe	**Firm Power** **Off peak winter** **Peak Summer**	26 MW 18 MW 54 MW
Peaking storage **Full supply level** **Minimum supply level** **Gross Storage**	5 M m^3 2414 m.a.s.l 2412 m.a.s.l 125 M m^3	**Distance up to Gilgit** **Voltage Level** **Transmission Line** **Areas to be served**	120 km 132 KV 70 km single circuit upto Sost & 27 km double circuit upto Aliabad. Hunza, Gilgit & Ghizar Region

1.9 PROJECT MANAGEMENT CONSULTANTS

Request for Proposal (RFP) for Management Consulting Services for Review of Feasibility Study, Procurement of EPC Contractor and Contract Management and Administration of Attabad Lake Hydropower Project (54 MW) was issued by Pakistan Water and Power Development Authority (WAPDA). The Contract for Consultancy Services, through competitive bidding, was awarded to the Joint Venture of National Engineering Services Pakistan (Pvt.) Ltd. (NESPAK) the lead firm, Associated Consulting Engineers (ACE.) Ltd, Pakistan Engineering Services (Pvt.) Ltd. (PES), BARQAAB Consulting Services (Pvt.) Ltd. and DOLSAR Engineering Inc. Co., as JV Partners.

The Consultancy Services Agreement was signed after pre-contract negotiations between the Pakistan Water and Power Development Authority (WAPDA). i.e. the Client, and Joint Venture Members i.e. the Consultant, on June 08, 2022, at the office of GM Projects (Northern Areas) WAPDA, Sunny view, Lahore. The effective date of the commencement of services is July 07, 2022.

1.10 SCOPE OF MANAGEMENT CONSULTANCY SERVICES

The detailed scope of consultancy services is listed below:

a. Phase I (Procurement)

1. Review of Feasibility Study Report
2. Visit the site and submit recommendations.
3. Prepare Generation Capacity Expansion Plan (GCEP)
4. Prepare EPC Tender Documents for International Competitive Bidding.
5. Prepare pre/post qualification criteria for finalization and approval from the Client. Assist the Client in pre/post qualification of bidders, replies to bidders' queries, evaluation of technical and financial bids, negotiations with successful bidder, award and finalization of the Contract Agreement.

b. Phase II (Mobilization, Detail Engineering Design, Construction)

1. Review the EPC Contractor's major design documents and data and verify his basis of design.
2. Monitor submission of documents against the Engineering Review Documents Schedule, review in a timely manner, the EPC Contractor's drawings, specifications, O&M manuals etc., to ensure compliance with the EPC specification.
3. Provide timely notice of events or trends to Client, which could have potential negative impacts to project and propose resolutions.
4. Be cognizant of and pro-active in reviewing the overall design of the Project for design improvements, which may result in long term benefits including cost reductions, schedule improvements and/or better operability, maintainability etc.
5. Review and comment on major purchase orders and sub-contracts.
6. Respond to any major design or equipment change requests submitted by the EPC Contractor.
7. Attend any model tests or final workshop inspections for the plant to be installed in the Project.
8. Monitor procurement and expediting activities performed by the EPC Contractor.
9. Witness In-works Performance, Commissioning and reliability tests for major items of equipment.

10. Review the construction methods of the contractor prior to application, to avoid the use of unfair or improper methods, which could result in delays, defective works or intolerable exposure to hazards of persons or property.
11. Review the procurement, implementation, delivery and disbursement schedule.
12. Coordinate the post construction services to be carried out by the EPC Contractor during the Defect Liability Period.
13. Maintain the necessary presence until the site is cleared by the EPC Contractor.
14. Develop an effective working relationship with EPC Contractor ensuring clear and appropriate forms of communication are established and maintained through-out onsite construction, commissioning and testing phase of project.
15. Review, comment and monitor the EPC Contractor's compliance with its own procedures, for example: Construction Management Plan and Environmental Management Plan.
16. Certify the EPC contractor's requests for payment against the contract requirement.
17. Prepare an MC Project Close-Out Report containing a historical record of the project.
18. Review and comment on the EPC Contractor's Project Completion Final Report, including the As Built Documentation and O & M manuals before final acceptance.
19. Certify to client that the project works have been completed in accordance with the Contract in view of the monitoring of the construction and evaluation of the performance tests including defect liability period.
20. Review and approve Contractor's "Quality Assurance Plans" and monitor the work of the Quality Assurance units established by the Contractor.
21. Monitor / coordinate field and laboratory tests being carried out during construction.
22. Monitor the collection of samples through drilling and tests pits and testing of fill materials, rip rap, concrete aggregate, etc.
23. Coordinate and monitor the selection of prospective borrow areas and identification of processing requirements.

1.11 CURRENT REVIEW REPORT

After the site visit, a detailed technical review was carried out to confirm that the design study is consistent with Feasibility level engineering requirements. Consultants reviewed all the sections of the Feasibility Report of 54 MW Attabad Lake Hydropower Project. The Review Report includes some specific data, geologic maps, plans, and sections, taken from previous reports, for quick reference. Review findings are presented in each chapter with further recommendations for Detail Design by EPC Contractor.

2 REVIEW OF POWER MARKET AND DEMAND FORECAST STUDIES

2.1 GENERAL

Electrification of the remote areas remains a critical issue for the Gilgit-Baltistan (GB) region of Pakistan, where most of the population lives without access to the national grid and mostly relies on traditional energy resources, which ignores the huge potential of renewable energy in the region. This section addresses the energy access issue of 1.8 million people of Gilgit-Baltistan (GB), of which 86% reside in the rural areas.

Gilgit-Baltistan in the North, faces severe electricity shortfalls despite having a good potential of untapped photovoltaic (PV), wind and hydro resources due to lack of proper energy policy, infrastructural development issues, and investment barriers for private sector. Sustainable access to energy is a major issue of the region, with 15 to 18 hours of daily power outages that have critically hampered the commercial activities. Across Gilgit-Baltistan, energy sources used in households and commercial enterprises are still firewood, kerosene oil, candles, dung cakes, diesel oil, batteries, liquefied petroleum gas (LPG), and coal. The current energy mix of the GB region is 45% firewood, 30% LPG, 6% kerosene oil and only 19% from (distributed hydro plants).

2.2 POWER GENERATION AND DEMAND COMPARISON OF GILGIT BALTISTAN

Electricity being generated by small hydropower stations with capacities ranging from 50 kW to 4000 kW. Currently, 119 hydropower stations are in operation with a total installed capacity of 148.69 MW. During winter, the supply of electricity reduces due to low water flows in rivers; however, demand remains high because of harsh weather. The seasonal shifting of the population from higher altitudes to lower valleys also affects the overall demand for energy in winter. During summer, a substantial influx of tourists is another factor in the increased energy demand across GB. However, the region is significantly affected in winter, as river flows decrease to drastic low levels. Seasonal variation in electricity generation, demand, and shortfall of the ten districts of GB are illustrated in Fig. 2.1. Gilgit region has six (06) districts namely Gilgit, Nagar, Hunza, Ghizer, Diamer (Chilas) and Astore, while Baltistan Region has four (04) Districts namely Skardu, Ghanche, Kharmang and Shiger. Gilgit is the capital of the GB region and due to trade activities, it has highest gap of 54.8MW in winter, while the total shortfall of the whole GB is about 242MW.

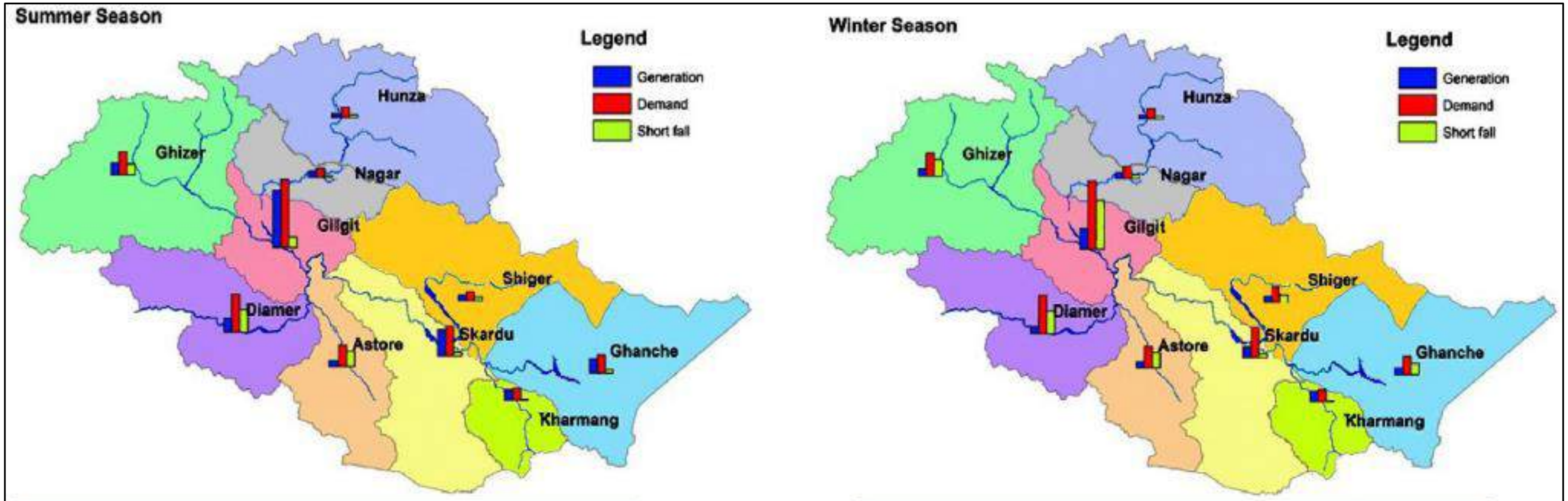
2.3 POWER DISTRIBUTION, TRANSMISSION AND NATIONAL GRID CONNECTIVITY OF GILGIT BALTISTAN

Gilgit-Baltistan has a rugged terrain that makes an extension of the national grid very costly. Therefore, GB remains electrically isolated from the country and consequently faces severe shortfalls. In near future, a mega hydro project like Diamer-Basha will enable the construction of a 765kV double-circuit transmission line to connect the region with the national grid (National Transmission & Dispatch Company, NTDCL). Though a small region of GB in south-east will be connected to national grid, interconnection of all the dispersedly populated areas through the mountainous terrain is a daunting task for provincial government. SG technologies made it favorable to share energy among distributed region with a robust centralized and distributed energy management system coupled with smart energy hubs and storages, which provide reliable, secure and resilient clean energy with minimum transmission congestions. Therefore, micro grid technologies will remain most suitable for provision of electricity to these off-grid areas. Moreover, micro grids are crucial building blocks of future national grids with in-built resilience.

Across GB, power is generally distributed over short distances through 11kV distribution lines for domestic, commercial, and industrial usage. In only a few areas of Gilgit and Skardu, the power supply network has been augmented with 66kV transmission lines, while a majority of the GB region relies on 11kV feeders, where each feeder is independently supplied by a group of hydel and thermal power plants. This transmission and distribution system experiences tremendous operational difficulties in terms of selective tripping and load management. Faults are cleared by tripping an entire feeder because of limited switchgear deployment along the length of the feeder. Moreover, load shedding by intentional simultaneous tripping of multiple feeders is practiced throughout GB to manage severe power shortages.

District Map of Gilgit Baltistan is shown in Figure 2.1 and District wise generation and demand comparison (from a research paper) for both summer and winter season is shown in Figure 2.2.

Figure 2.1: District Map of Gilgit-Baltistan



District	Generation	Demand	Short fall
Gilgit	37.95	45	7.05
Diamer	9.39	25	14.81
Skardu	17.5	20	2.5
Ghizer	7.79	15	7.21
Ghanche	9.7	12	2.3
Astore	3.9	14	10.1
Nagar	4.1	6	1.4
Hunza	2.9	7	2.1
Shigar	3.64	6	2.36
Kharmang	7.09	8	0.91

District	Generation	Demand	Short fall
Gilgit	25.2	80	54.8
Diamer	7.91	25	17.09
Skardu	13.55	30	14.45
Ghizer	4.99	25	19.43
Ghanche	6.79	20	13.05
Astore	3.74	22	18.26
Nagar	3	9	5.5
Hunza	2.5	9	4.5
Shigar	1.87	10	8.13
Kharmang	5.92	12	6.08

Figure 2.2: Electricity Generation and Demand Comparison of Gilgit-Baltistan in Summer & Winter season

Source: Research paper – Techno-economic assessment & sustainability impact of hybrid energy systems, Gilgit–Baltistan, Pakistan (2021)

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According to the statistics provided by Water & Power Division GILGIT-BALTISTAN, the power generation & demand is grouped for two seasons i.e. summer and winter. Table 2.1 shows these statistics season wise.

Table 2.1 Current Power Scenario of Gilgit-Baltistan

District	Present Generation (MW)						Present Demand (MW)		Present Shortfall (MW)	
	Summer			Winter			Summer	Winter	Summer	Winter
	Hydel	Thermal	Total	Hydel	Thermal	Total				
Gilgit	49.00	0.00	49.00	25.80	7.80	33.6	49.89	94.51	0.89	60.91
Diamer	9.79	0.80	10.59	6.75	4.00	10.75	36.02	50.34	25.43	39.59
Skardu	20.30	0.00	20.30	14.20	2.00	16.20	50.6	94.47	30.3	78.27
Ghizer	12.50	0.00	12.50	8.80	0.58	9.38	38.88	71.34	26.38	61.96
Ghanche	8.09	0.00	8.09	5.00	0.16	5.16	25.5	48.52	17.41	43.36
Astore	4.50	0.00	4.50	4.00	0.00	4.00	14.83	24.53	10.33	20.53
Nagar	4.56	0.00	4.56	2.03	0.50	2.53	9.27	16.97	4.71	14.44
Hunza	3.91	0.80	4.71	2.78	0.80	3.58	12.83	20.61	8.12	17.03
Shiger	3.50	0.00	3.50	2.00	0.00	2.00	10.35	18.36	6.85	16.36
Khermang	4.61	0.00	4.61	4.07	0.00	4.07	6.65	12.54	2.04	8.47
Total	**120.76**	**1.60**	**122.36**	**75.43**	**15.84**	**91.27**	**254.82**	**452.19**	**132.46**	**360.92**

Power scenario is sketched out in Figure 2.3 and Figure 2.4 for the summer and winter 2020, for Gilgit Baltistan, which show that due to bulk supply of water in summer deficit between demand and generation is less than from winter power scenario.

In Figure 2.5, comparison for the power demand in Gilgit-Baltistan for the both season of summer and winter is presented.

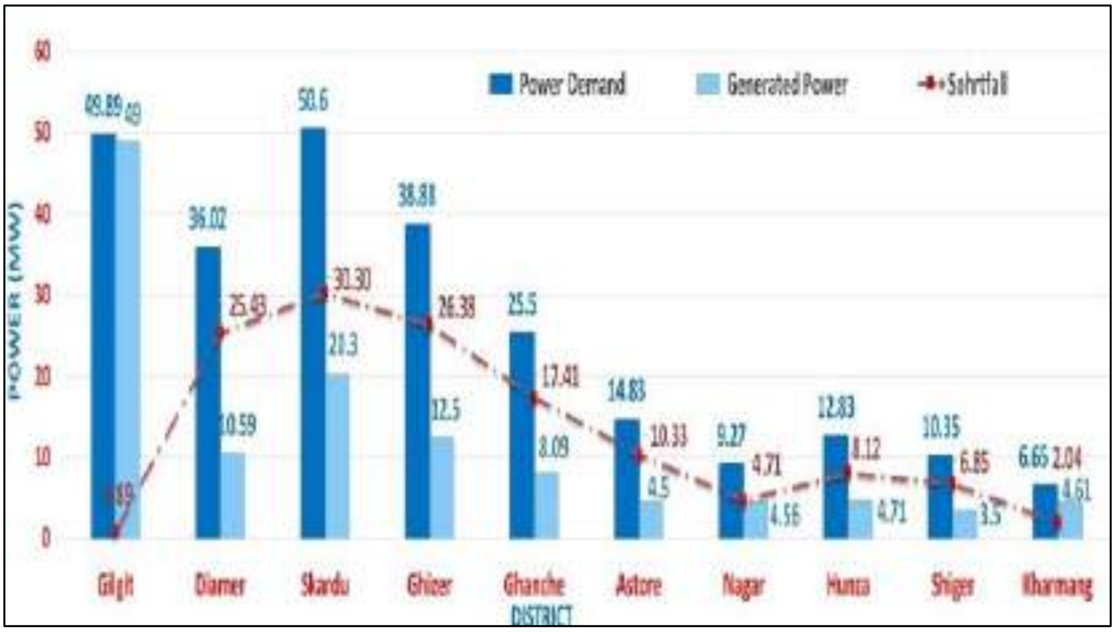


Figure 2.3: District wise Summer Power Scenario in Gilgit-Baltistan

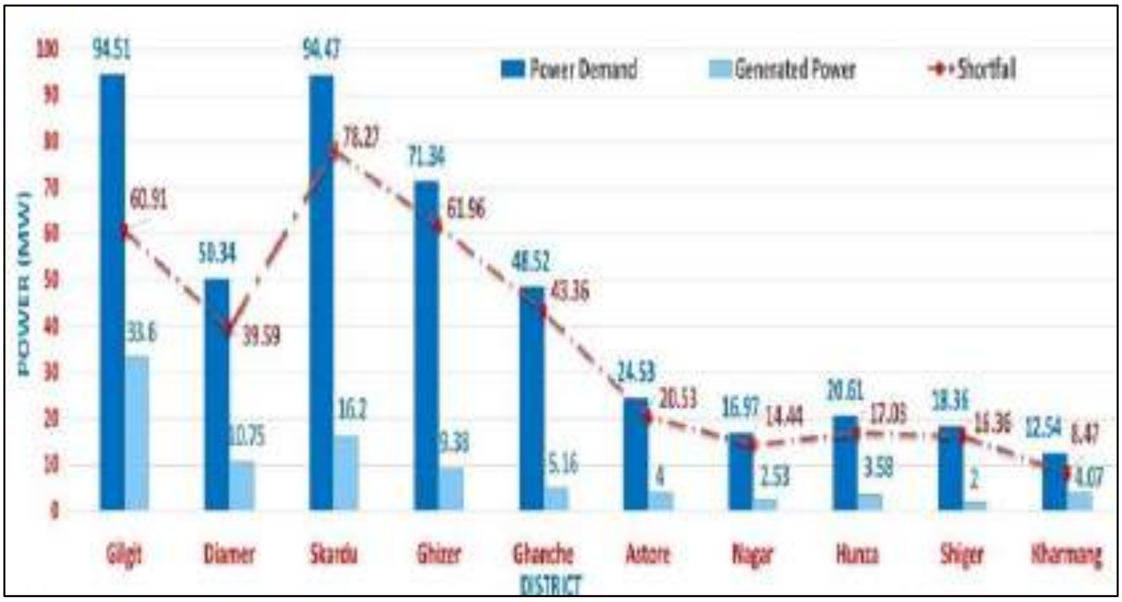


Figure 2.4: District wise Winter Power Scenario in Gilgit-Baltistan

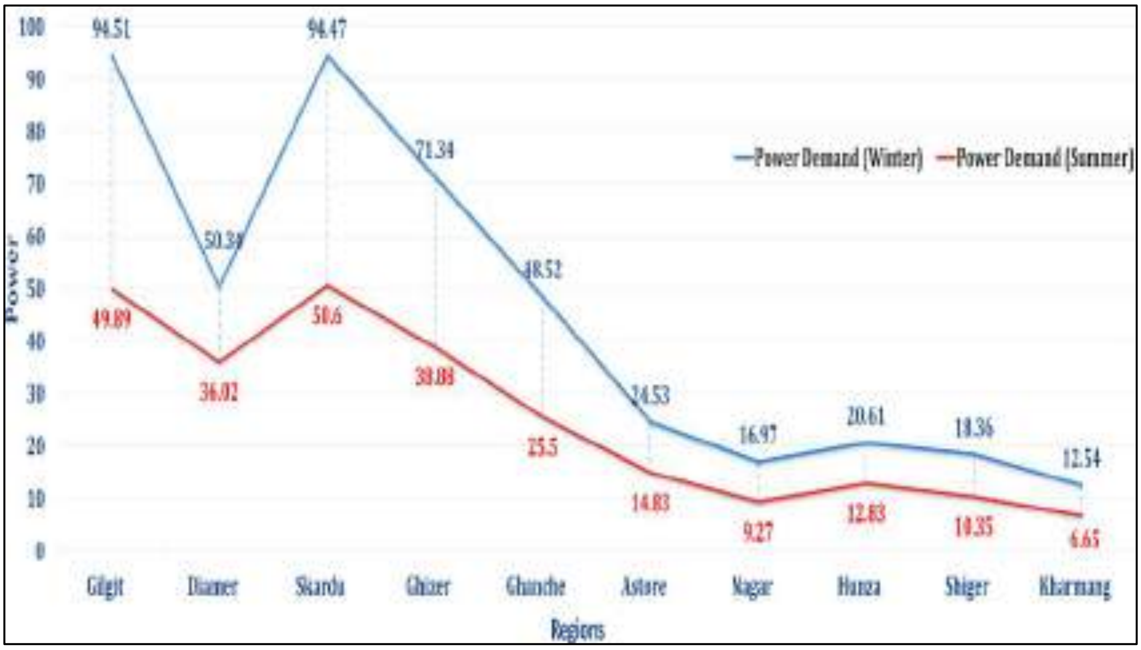


Figure 2.5: District wise Overall Power Demand in Gilgit-Baltistan

2.4 REVIEW FINDINGS AND RECOMMENDATIONS

Energy is a critical input for economic development and correspondingly power sector comprises an indispensable infrastructure in any economy. Providing adequate and affordable electric power is essential for economic development, human welfare and better living standards. The growth of the economy along with its global competitiveness hinges on the availability of reliable and quality power at competitive rates to all consumers throughout the country. Electricity is central to achieving economic, social and environmental objective.

Attabad Lake Hydropower Project is located in the Hunza District, it is worthy to discuss the current Power scenario of Hunza district in detail for understanding the imperial impact of project in the region and forecast the demand for the future. The power supply area of the proposed site covers mainly the Upper, Middle and Lower Hunza District.

Based on review of chapter presented in the Feasibility report MC validate the data and importance to construct the Attabad Lake Project as early as possible.

3 REVIEW OF TOPOGRAPHIC AND HYDROGRAPHIC SURVEY

3.1 GENERAL

The Feasibility stage topographic survey of ALHPP was executed using terrestrial survey and mapping technique. An area of approx. 04 km^2 was covered with an average extent of 1 km x 4.5 km. Following instruments were used to perform the activity.

- Topcon Total Station W0303
- DGPS Trimble R8s GNSS
- Raytheon echo sounder

As part of the Topographic survey the scope was as follows:

1. Setup a Project trigonometric network with a system of Benchmarks.
2. Terrestrial survey of the area of Project components about 400 hectares at scale 1:1000 with contour interval of 2 m, covering 4 km downstream of weir site along Hunza River.
3. Longitudinal profile of Hunza River, 4 km downstream from weir site.
4. Terrestrial survey of valley cross sections along Hunza River, 4 km downstream of weir site at an interval of 200 m, covering the main river course and valley slopes upto 200 m on either side.
5. Terrestrial survey of borehole locations.
6. Satellite based survey of the Project Area.

As part of the Hydrographic survey the scope was as follows:

1. Cross-sections of the lake along range lines from left to right bank, at 250 m interval for first 3 km & 500 m interval for rest of the lake.
2. Computation of existing storage capacity of the lake.
3. Development of area-capacity curve.
4. Development of contour maps of the Lake.
5. Discharge Measurement at Ganish Bridge.

3.2 REVIEW FINDINGS

3.2.1 Establishment of the Permanent Control Benchmarks

The project's permanent reference benchmarks, BM-1 and BM-2 were established using DGPS with an achieved accuracy of 50 cm and these benchmarks are 600 m apart, as discussed in section 3.7.3. The established two (02) permanent benchmarks do not conform with the survey standards and requirements discussed in its section 3.6.3, wherein it mentions the accuracy standard of

- Horizontal 3 mm + 0.1 ppm RMS
- Vertical 3.5 mm + 0.4 ppm RMS

The established permanent benchmarks and levels observed near the structure areas require field verification.

3.2.2 Observed Spot Levels

Considering the treacherous terrain and difficult access conditions, due to steep slopes, as the limiting factors, the spot levels collected during the field survey are very sparse, hence due to rigorous interpolation, the modeled topography does not depict the ground condition.

Moreover, near the Intake area, the developed contours do not match with the nearby spot levels and the difference ranges from 5 m to 10 m.

3.2.3 Survey of Locations of Geotechnical Investigations

In Feasibility report, refer section 3.8.6, the boreholes and test pits locations were marked at major project components prior to start of drilling work and considering the safe installation of the drilling rig the recorded ground levels were reported to be deviating up to 5 m from the NSL. The change of ground levels (Z) implies variation in the horizontal coordinates (X, Y) as well, hence requires clarity.

3.2.4 Satellite Based Survey of the Project Area

Satellite based various remote sensing dataset as discussed in the report, refer section 3.10, but the outputs and its application require clarity. Moreover, the calibration of remotely sensed datasets with respect to established geodetic control network is not known.

3.2.5 Results of Hydrographic Survey

In Feasibility report, refer section 3.12.2, describes that all the cross-sections observed during hydrographic survey are connected to the geodetic control, established for the topographic survey. Furthermore, a comparison of observed cross-sections with previous data of 2011, was made and deduced an average deposition of 30 to 40 m over 09 years. This comparison can be made if same geodetic reference is used during both survey campaigns.

3.3 REVIEW RECOMMENDATIONS

Considering difficult site accessibility and stated data limitations, the topographic survey data is required to be enriched by deploying modern survey and mapping techniques.

4 REVIEW OF HYDROLOGY AND SEDIMENTATION STUDIES

4.1 GENERAL

Northern Areas of Pakistan is unique in its geographical configuration and layout which houses confluence of three major mountain ranges i.e. Hindukush, Karakorum and Himalayans and 5 out of 14 peaks above 8000 m.a.s.l. The Hindukush-Karakoram-Himalaya (HKKH) Region, often called “Water Towers of Asia,” provides natural reserves of fresh water in the form of snow and glaciers that sustain water availability in mountainous or Himalayan Areas, as well as in adjacent areas.

Hydrology and its applications are inevitable for planning and building hydraulic structures. The hydrological series that represents variations in river flow throughout the year is utilized to estimate the water availability and sediment inflow at weir/dam site for assessment of water storage, reservoir operational study, flows diversion, power and energy production from the available stream flow.

Hydrological and sediment transport studies play a pivotal role in the planning, design and execution of the hydropower projects. The important hydrological parameters required to be evaluated are the availability of flows; extreme flow events along various project components. Flood studies are needed for sizing as well as safe design of structures. Moreover, the design flows for the hydropower machinery are also decided on the basis of available flows at dam site. Sediment study defines the life of reservoir.

In Feasibility study, Daily flow data of Dainyor Bridge station at Hunza River for the period of 1966 to 2015 was used for the estimation of flows at Attabad Lake. Mean Annual Flows Varies from 127m³/s to 310m³/s. Mean annual flows estimated as 208.36m³/s. Mean Monthly flows vary from 28.34 cumecs in March to 696.45 cumecs in July. Average mean monthly flows estimated as 206.13 cumecs. Rated design discharge of 60m³/s was available for almost 50% of the time for the installed capacity of 54 MW power generations. Design flood of 3,250m³/s at weir site had been selected for the calculations of design of structures. Average annual suspended sediment estimated as 20.06MMT and bed load had been considered as 20% of the total suspended load. Total Sediment load at weir site worked out to be 24.06 M Tons. Without flushing, reservoir would be filled with sediments in 10 years.

4.2 WATERSHED BOUNDARY OF WEIR

The catchment area upto proposed weir site has been meausred by using GIS software. It has been estimated as 8922.9Km². Maximum Elevation of catchment area is 5237m amsl. The watershed boundary for the proposed Weir site is shown in Figure 4.1.

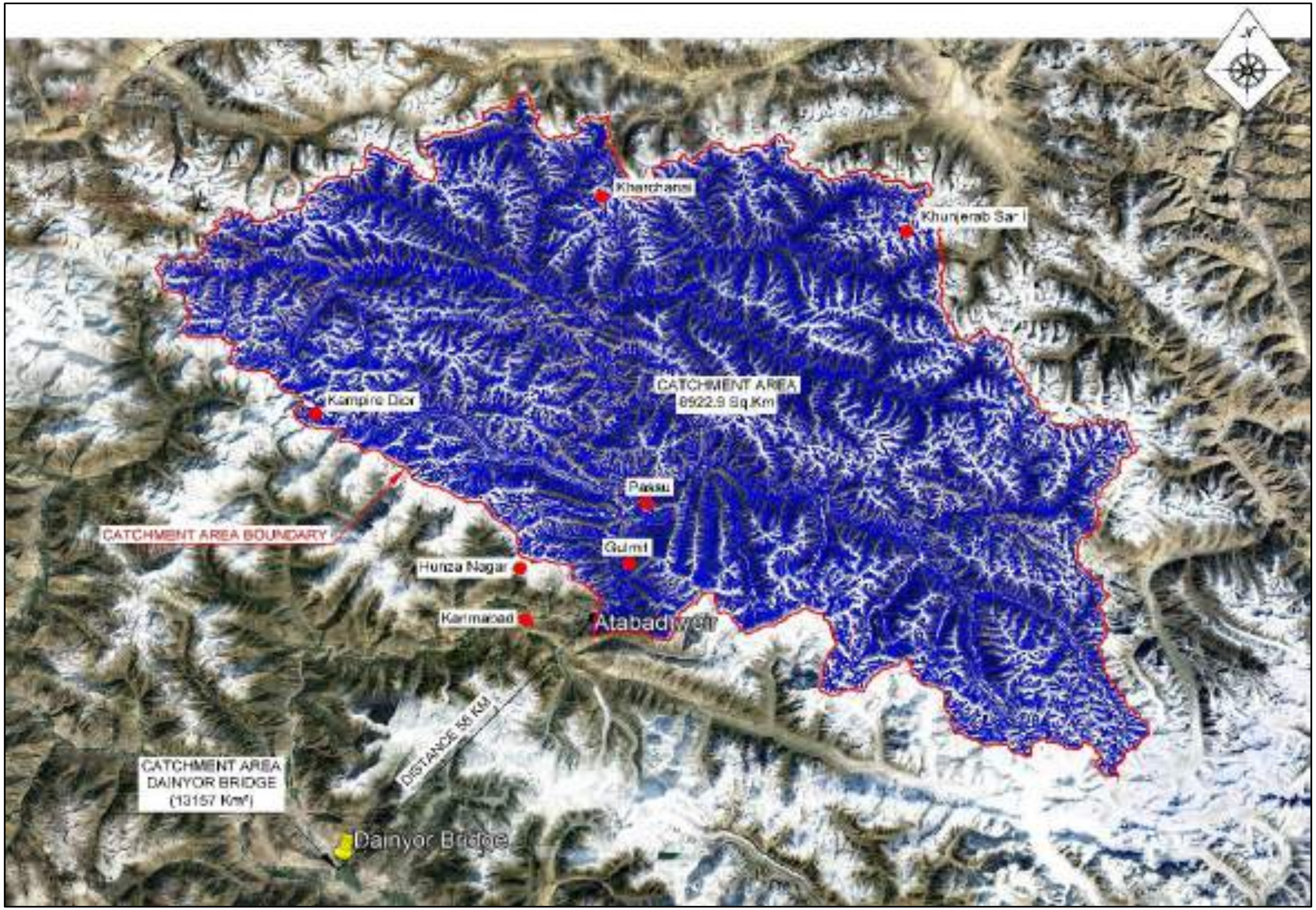


Figure 4.1: Watershed Boundary for Weir Site

4.3 CLIMATOLOGY

Precipitation, air temperature, relative humidity and evaporation are the important climatological parameters required for understanding the climate of the region. There are number of climatic stations in the vicinity of project area, which are operated by Pakistan Meteorological Department (PMD) and Surface Water Hydrology Project (SWHP) of WAPDA. In the review of feasibility study, the climatic data of Skardu and Gilgit station is collected.

4.3.1 Climatological Data - Skardu

Precipitation and tempreture data at Skardu are collected from Pakistan Meteorological Department (PMD), which are summarised in Table 4.1 and presented in Figure 4.2 & Figure 4.3.

Table 4.1 Mean Monthly Temperature & Precipitation - Skardu

Months	Air Temperature °C (1952-2004)			Precipitation (2007-2015)
	Mean Min Temp	Mean Max Temp	Mean Temp	(mm)
Jan	-8.08	2.87	-2.44	32.38
Feb	-4.79	5.70	0.45	63.27
Mar	1.51	12.01	6.72	41.63
Apr	6.53	18.49	12.53	31.04
May	9.67	22.86	16.28	31.59
Jun	13.45	28.36	20.91	10.68
Jul	16.65	31.61	24.13	12.63
Aug	16.19	31.12	23.66	17.18
Sep	11.83	27.24	19.53	30.76
Oct	4.57	17.91	12.37	4.10
Nov	-1.65	13.00	5.68	5.88
Dec	-5.38	6.31	0.41	33.90
Mean Annual Precipitation (mm)				**232.64**

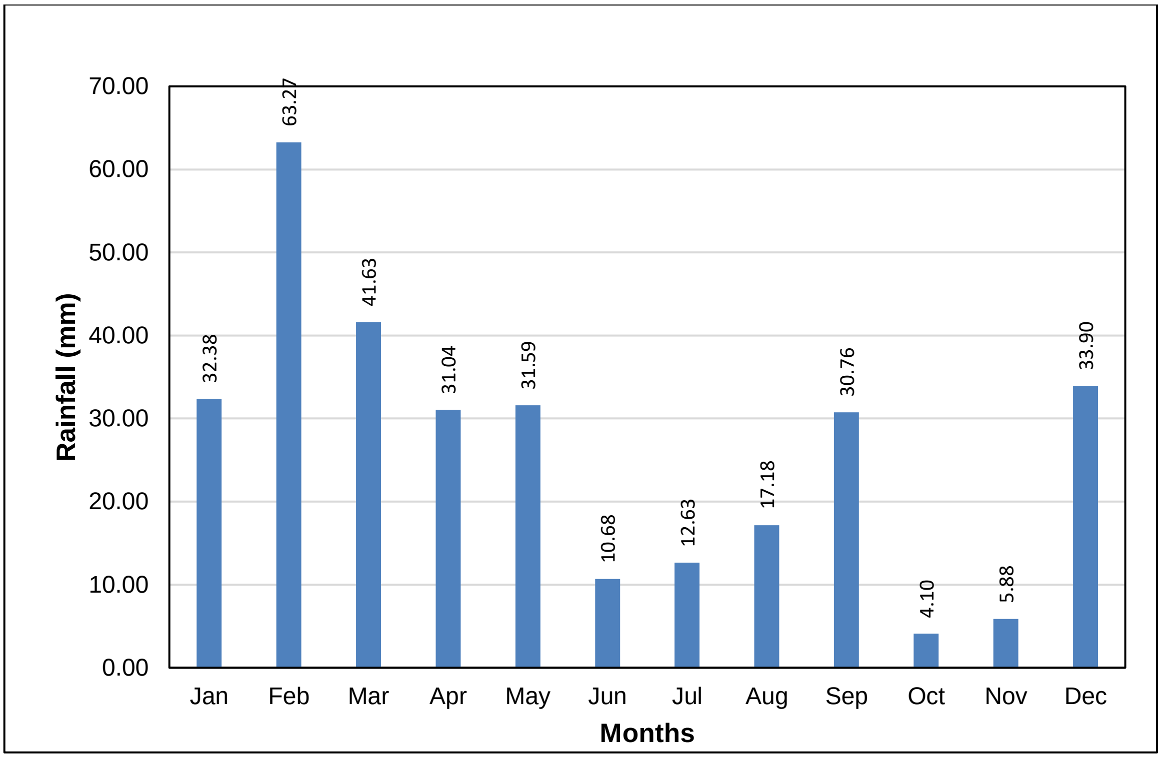


Figure 4.2: Mean Monthly Precipitation Skardu (2007-2015)

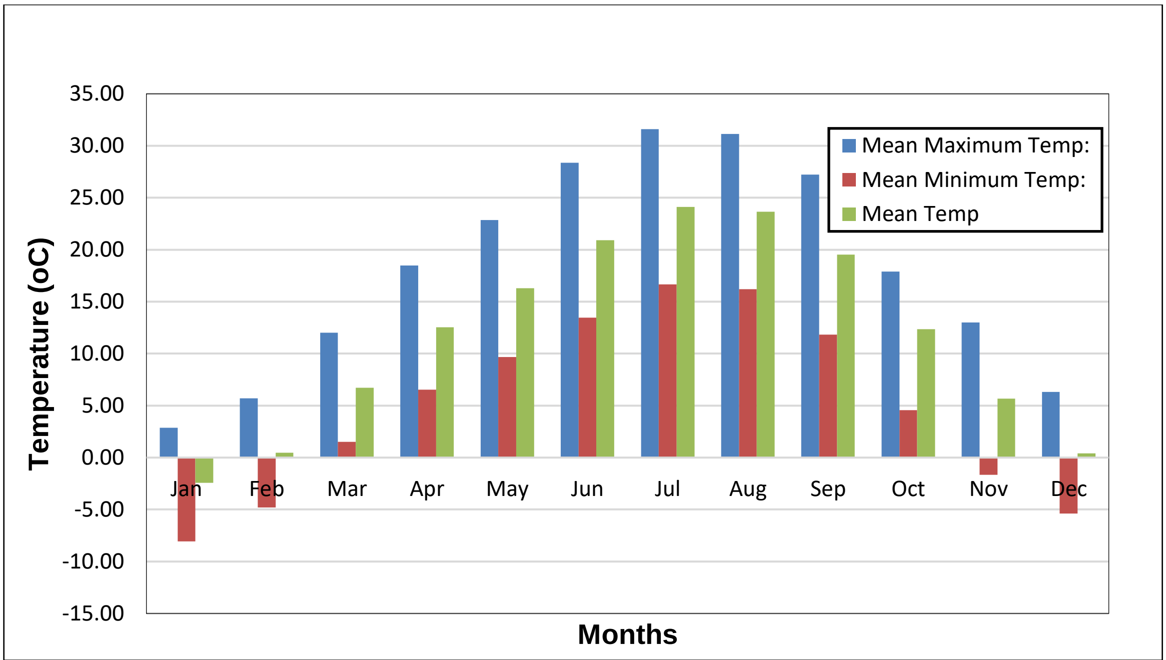


Figure 4.3: Mean Monthly Max, Min and Mean Temperature Skardu (1952-2004)

4.3.2 Climatological Data - Gilgit

Precipitation and tempreture data at Gilgit Station is collected for period of 1951-2014. Mean monthly (maximum and minimum tempretures) and precipation data are given Table 4.2 and represented in Figure 4.4.

Table 4.2 Mean Monthly Climatic Data at Gilgit

Month	Air Temperature °C (1951-2014)		Precipitation (1951-2014)
	Max Temp (°C)	Min Temp (°C)	(mm)
Jan	9.4	2.4	4.11
Feb	12.6	0.7	7.41
Mar	18.2	5.7	11.76
Apr	24.1	9.5	17.10
May	28.6	12.1	21.46
Jun	34	15.1	22.58
Jul	36.1	18.4	24.99
Aug	35.3	17.8	26.08
Sep	31.7	12.7	20.87
Oct	25.5	6.8	14.42
Nov	18.2	0.9	8.40
Dec	11.5	-1.9	4.96
Mean Annual Precipitation (mm)			184.14

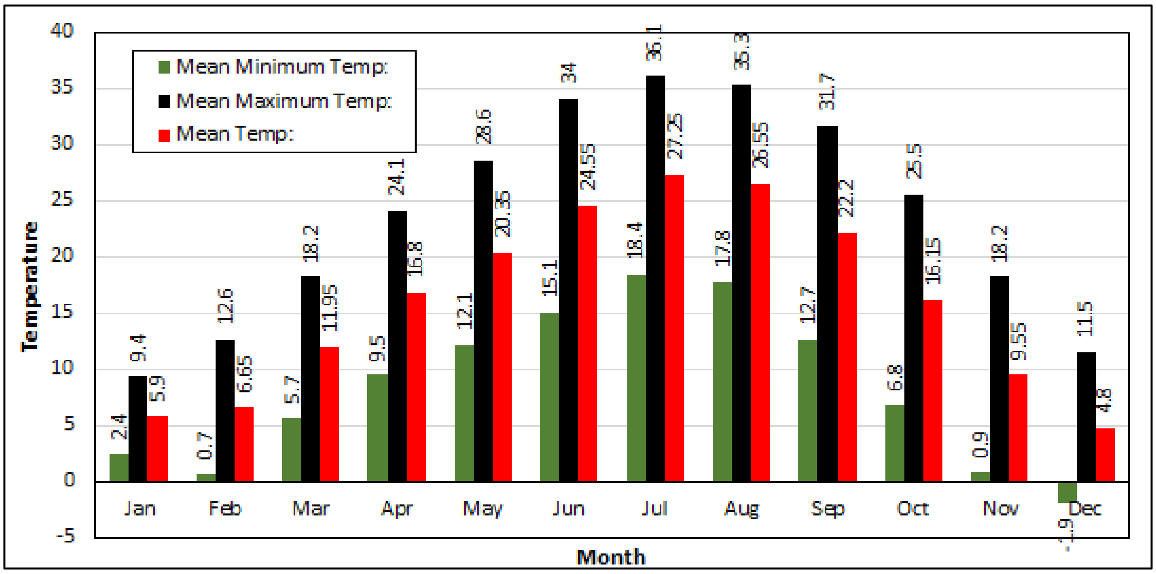


Figure 4.4: Mean Monthly Max, Min and Mean Temperature Gilgit (1951-2014)

4.4 HYDROLOGICAL STATIONS

There are a number of hydrological stations installed on main Indus River and its tributaries around project area for which flow data is available. Flow measurements have been carried out by the Surface Water Hydrology Project (SWHP) of WAPDA.

The list of few important gauging stations surrounding the Project area is given in Table 4.3.

Table 4.3 Availability of Data at SWHP Hydrological Stations

Station/River	Location		Elev. masl	Area (km^2)	Period of Available Record
	Lat.	Long.			
Kharmang/Indus	34°56’	76°13’	2469	67850	1992 to 2007
Yogo/Shyok	35°11’	76°06’	2469	33670	1973-2014
Shigar/Shigar	35°20’	74°45’	2438	6610	1985, 1987, 1990,1992-1998, 2001
Kachura/Indus	35°27’	75°25’	2341	11266	1984 to 2007 excluding 1991
Dainyor Bridge/Hunza	35°55’40”	74°22’35”	1494	13157	1966-2015
Gilgit/Gilgit	35°55’35”	75°18’25”	1554	12095	1992-2007
Alam Bridge/Gilgit	35°46’03”	74°35’50”	2195	26159	1975, 1992-2007
Partab Bridge/Indus	35°43’50”	74°37’20”	1228	142709	1992-1995, 1975, 1977, 1981, 1984, 1985, 1987-1990

4.5 FLOW STUDY

Estimation of flows is an important component for design of hydraulic structures, reservoir parameters, and hydropower projects as well as for other water sector projects. Flow study is essential requirement for the estimation of water availability at Attabad Lake HPP. For this purpose, a long-term flow series are required at a site being considered for diversion of flows. Proposed project location is situated at Hunza river at upstream of Dainyor gauging station. Flows generated at Attabad Lake by using daily flows data of Dainyor Bridge at Hunza River for period of 1966-2015. The methodology adopted for developing these correlations are as follows.

4.5.1 Estimation of Flows - Feasibility Study 2021

Flows were estimated at proposed weir by using observed daily flows data of Dainyor Bridge at Hunza River for period of 1966-2015. That gauge was operative by SWHP, WAPDA. The catchment area of Attabad Lake estimated as 8901 km^2. The flows generated at weir site by using catchment ratio method. The mean monthly available flows at weir site are shown in Table 4.4.

Table 4.4 Monthly Flows at Weir - Attabad Lake (1966-2015)

Months	Mean Monthly Flows (cumecs)
Jan	32.78
Feb	29.85
Mar	28.35
Apr	38.46
May	115.67
Jun	373.00
Jul	696.45
Aug	668.68
Sep	300.77
Oct	99.08
Nov	52.71
Dec	37.77
Average Mean Monthly	**206.13**

Flow duration curve was developed at the proposed weir site by using daily recorded flows of Attabad Lake for the period 1966-2015, which is presented in Table 4.5 and shown in Figure 4.5.

Table 4.5 Flow Duration Curve - Feasibility Study

Time (%)	Discharge (m^3/s)
10%	613.4
20%	376.6
30%	205
40%	103.3
50%	61.3
60%	44.3
70%	36
80%	30.3
90%	26.2
99%	20.5

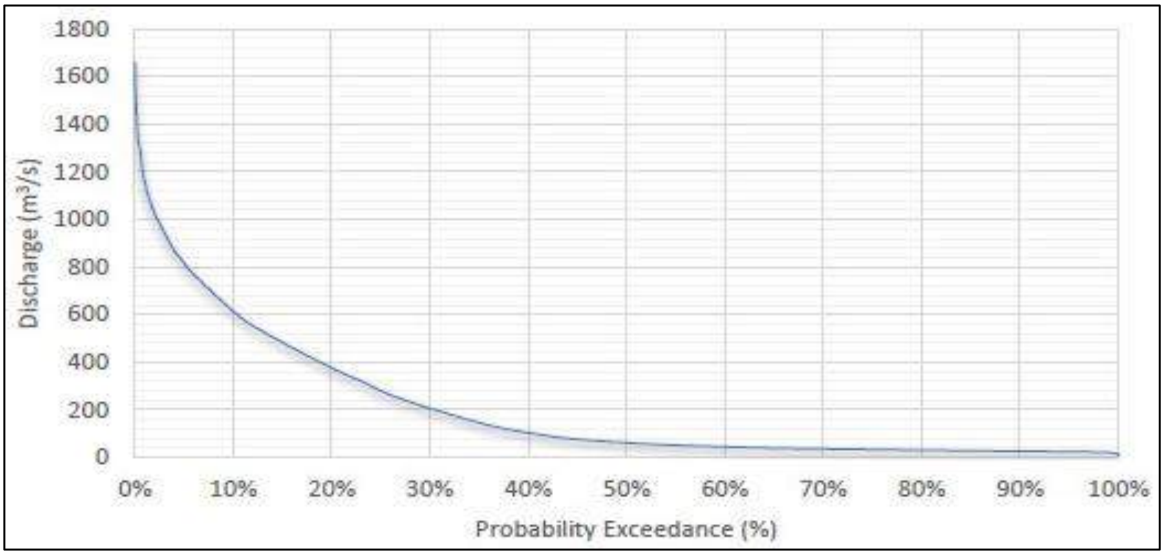


Figure 4.5: Flow Duration Curve - Feasibility Study 2021

4.5.2 Estimation of Flows - Review Study

In Present study, flows are estimated at Attabad Lake, proposed weir location by using daily flows data of Dainyor Bridge at Hunza River for period of 1966-2015. Catchment area of Dainyor Bridge at Hunza River is 13,157 km^2 and Attabad Lake having 8922.9 km^2. Flows are generated at Attabad Lake-Weir for period of 1966-2015, by using catchment area ratio (0.678) method at observed flows data of Dainyor Bridge station. Mean Monthly flows of Dainyor Station at Hunza River for period of 1966-2015 are represented in Figure 4.6 and are tabulated in Table 4.6

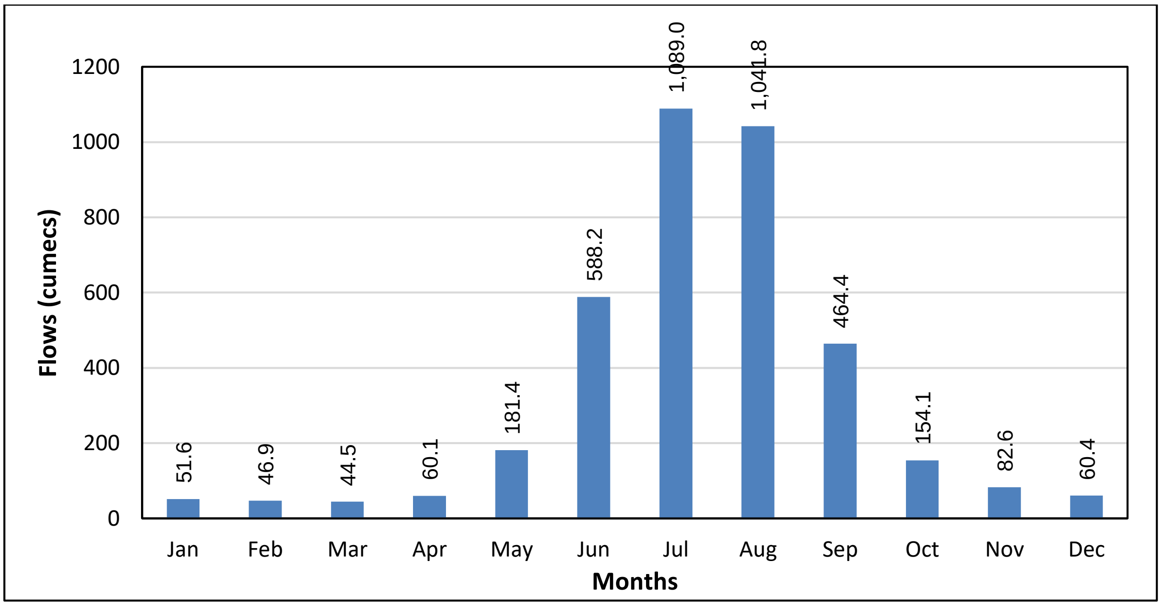


Figure 4.6: Mean Monthly Flows Dainyor Bridge at Hunza River (1966-2015)

Table 4.6: Mean Monthly Flows at Dainyor Bridge-Hunza River (1966-2015)

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Mean Annual (cumecs)
1966	54.4	49.1	36.3	48.6	107.6	774.8	1179.4	1305.7	623.1	168.5	75.5	61.5	373.70
1967	56.5	55.3	54.3	70.3	109.3	618.9	1469.0	1360.3	626.0	142.3	72.5	52.2	390.58
1968	42.1	39.3	36.2	45.6	125.5	746.9	1451.9	1519.9	455.4	135.2	72.1	58.7	393.91
1969	44.3	41.8	41.7	61.8	147.5	585.4	1443.2	1534.0	454.9	187.2	82.1	66.5	390.87
1970	54.0	46.4	42.7	77.1	200.8	759.5	1183.3	1641.4	893.1	222.8	76.5	55.0	429.39
1971	46.9	44.6	42.8	67.9	250.8	1266.3	1340.2	1324.4	473.4	148.6	78.3	63.3	428.96
1972	48.2	40.4	39.6	48.1	127.9	494.2	697.9	837.3	550.3	166.9	76.4	55.3	265.13
1973	48.1	42.8	38.9	68.2	199.0	1039.0	1798.9	1543.5	732.1	179.2	87.3	59.6	486.39
1974	52.1	48.6	48.5	60.3	155.6	448.0	1013.6	954.7	360.3	148.6	75.7	52.8	284.90
1975	45.0	43.3	40.0	52.4	139.7	464.4	940.3	1258.9	541.1	160.6	72.2	44.4	316.87
1976	35.2	35.3	36.6	45.8	195.5	485.5	1516.0	897.6	417.7	128.6	82.4	50.3	325.56
1977	45.0	42.4	39.7	60.4	168.6	809.8	1786.9	1297.6	462.8	161.5	74.3	46.9	416.23
1978	39.0	38.5	37.7	47.0	222.1	949.7	1869.5	1697.0	524.3	175.0	68.6	45.0	476.11
1979	55.5	49.9	48.1	60.8	124.3	617.8	1718.9	1468.5	439.2	169.1	88.3	59.7	410.07
1980	55.8	49.2	56.1	74.2	188.2	673.9	1148.7	954.9	459.4	170.4	78.2	57.4	330.54
1981	49.3	40.9	38.8	65.8	322.8	494.7	1236.7	1002.6	305.0	126.7	68.2	51.5	316.91
1982	45.5	42.6	39.5	61.8	212.6	479.9	785.8	1085.1	247.2	91.3	63.3	55.8	267.52
1983	51.7	46.6	37.8	46.4	200.5	410.8	888.4	1335.6	701.4	130.2	70.1	43.5	330.23
1984	40.8	37.9	37.4	43.9	75.6	623.4	1109.8	1639.6	425.9	104.2	73.9	53.3	347.14
1985	43.4	40.1	40.2	51.9	118.5	422.7	1260.4	1173.8	488.4	153.6	64.0	50.0	325.60
1986	42.4	39.3	37.2	49.8	126.9	418.9	1156.6	946.4	411.2	205.7	80.2	54.6	297.35
1987	46.7	41.7	38.1	44.2	102.7	355.9	630.9	821.6	580.0	164.5	74.3	62.6	246.94
1988	51.6	44.0	38.8	83.7	221.8	536.8	1248.7	1023.2	429.7	116.3	80.3	62.3	328.09
1989	49.5	43.5	41.2	44.0	90.0	389.5	723.0	688.5	400.2	153.1	77.6	60.0	230.02
1990	52.1	45.7	39.4	44.4	311.0	713.6	1147.0	1128.2	675.0	202.4	82.8	63.8	375.45
1991	50.9	46.0	45.7	53.1	110.4	454.8	774.6	855.4	647.4	116.4	63.5	54.6	272.74
1992	49.5	45.5	40.9	46.7	108.6	314.4	826.2	811.2	385.2	139.4	87.4	59.5	242.86
1993	44.3	39.9	36.1	72.1	146.6	424.7	788.7	664.2	422.6	156.4	81.0	54.4	244.16
1994	47.0	41.0	38.5	41.9	188.2	535.9	1476.7	1368.8	495.1	132.2	73.0	54.4	374.40
1995	45.8	43.0	37.0	43.8	151.7	393.1	1058.9	1014.8	279.8	105.8	63.3	50.3	273.95
1996	40.9	36.0	34.6	47.0	76.0	464.6	787.9	917.2	441.6	139.0	64.6	51.5	258.40
1997	49.7	47.1	41.6	59.4	107.1	293.7	655.6	564.9	303.3	142.3	71.6	58.2	199.38
1998	46.0	46.1	42.2	63.3	213.1	723.8	1162.7	813.4	641.4	169.3	88.2	63.9	329.45
1999	48.4	42.9	42.7	54.9	138.7	513.9	1021.7	1177.2	370.3	121.4	65.8	57.5	304.45
2000	47.6	38.4	36.2	57.2	345.0	588.6	818.9	768.1	450.9	118.0	81.3	51.6	281.81
2001	47.1	42.7	39.7	76.5	320.8	750.4	1310.4	990.1	382.3	146.6	70.9	49.7	352.18
2002	43.6	41.7	40.3	56.9	176.7	555.6	695.0	875.4	411.1	158.3	71.0	53.1	264.97
2003	46.8	41.1	34.8	56.6	125.3	579.7	1172.5	807.3	633.2	133.9	91.2	61.0	306.88
2004	76.8	67.7	63.4	83.4	251.9	694.7	643.9	536.4	226.7	114.1	100.9	91.7	245.96
2005	84.2	77.4	57.6	88.0	315.7	940.5	1771.6	758.5	420.2	153.5	83.4	73.8	402.13
2006	57.1	55.1	52.9	62.1	381.4	474.6	737.1	774.4	305.4	152.2	108.8	91.1	271.01
2007	77.3	57.3	53.1	56.6	311.9	720.4	629.9	500.2	288.4	153.0	85.5	70.9	250.37
2008	64.3	65.3	68.4	83.1	212.5	900.7	1102.6	956.4	414.9	126.3	106.0	78.3	348.23
2009	63.6	61.9	60.2	64.0	144.3	489.1	806.5	1005.5	433.7	259.7	160.0	90.7	309.09
2010	33.2	25.4	23.3	46.4	99.9	427.7	991.2	1238.3	481.6	170.0	113.4	68.0	309.86
2011	85.0	66.0	100.1	91.6	190.2	701.6	819.6	742.2	521.0	212.1	136.9	99.9	313.85
2012	69.5	86.5	63.1	74.3	180.5	417.1	887.1	917.3	553.1	135.2	83.5	60.2	293.93
2013	65.9	49.2	59.1	77.4	216.8	793.7	987.9	946.1	670.4	204.7	103.7	68.9	345.34
2014	55.4	51.8	44.6	50.3	136.2	366.8	800.8	818.3	355.4	163.1	94.8	64.1	250.12
2015	45.9	40.9	41.6	64.5	174.1	348.8	900.9	1030.1	306.3	167.2	104.3	62.1	273.90
Mean Monthly (cumecs)	51.6	46.9	44.5	60.1	181.4	588.2	1089.0	1041.8	464.4	154.1	82.6	60.4	322.1

Mean monthly flows for Dainyor bridge are minimum during March and maximum during July with values of (44.5 m³/s and 1089 m³/s) respectively. Mean annual flows are 322.1 cumecs.

- **Extended Flow Series at Weir Site**

Flows have been estimated during review studies at Attabad Lake Weir for period of 1966-2015 by using Dainyor Bridge data at Hunza River with the application of catchment area ratio method. Mean monthly maximum and minimum flows available at proposed weir site are 739.2 cumecs in July and 29.95 cumecs in March, respectively. Average mean monthly flows estimated at weir site with value of 218.48 cumecs. The mean annual flows and mean daily flows estimated as 220.07 cumecs and 219.69 cumecs, respectively. The mean monthly and mean annual flows are shown in graphically form in Figure 4.7 and Figure 4.8, respectively and in Table 4.7.

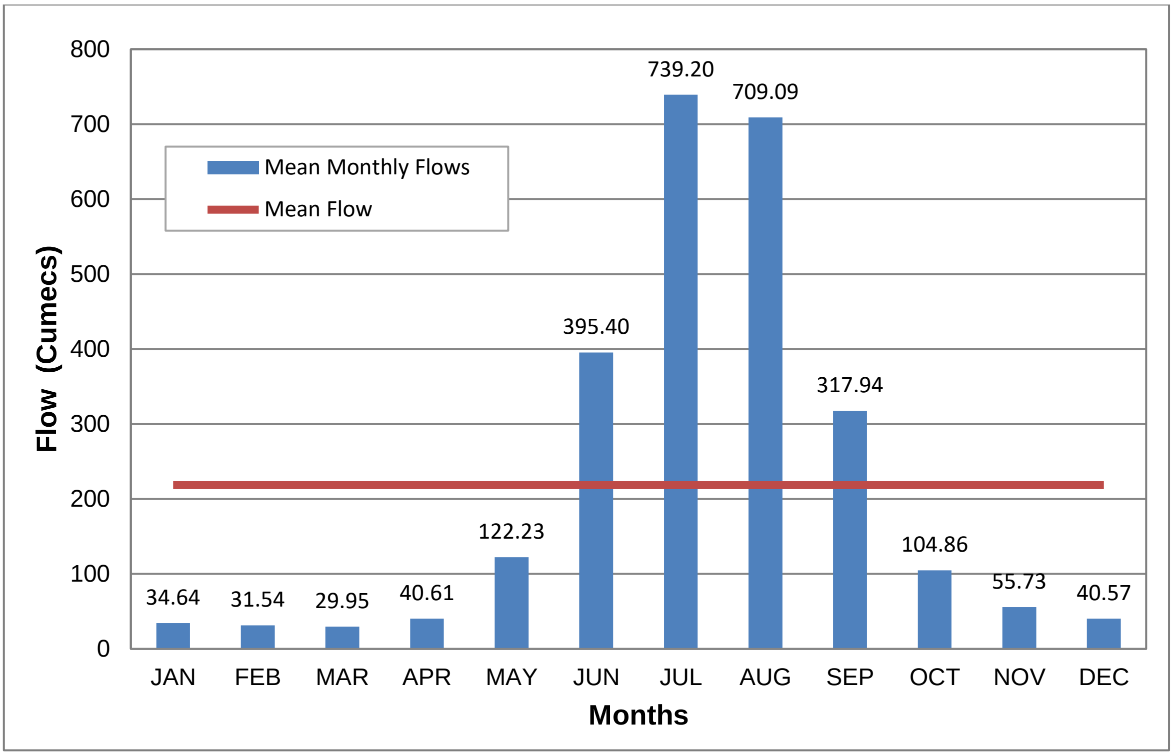


Figure 4.7: Extended Mean Monthly Flows at Weir (1966-2015)

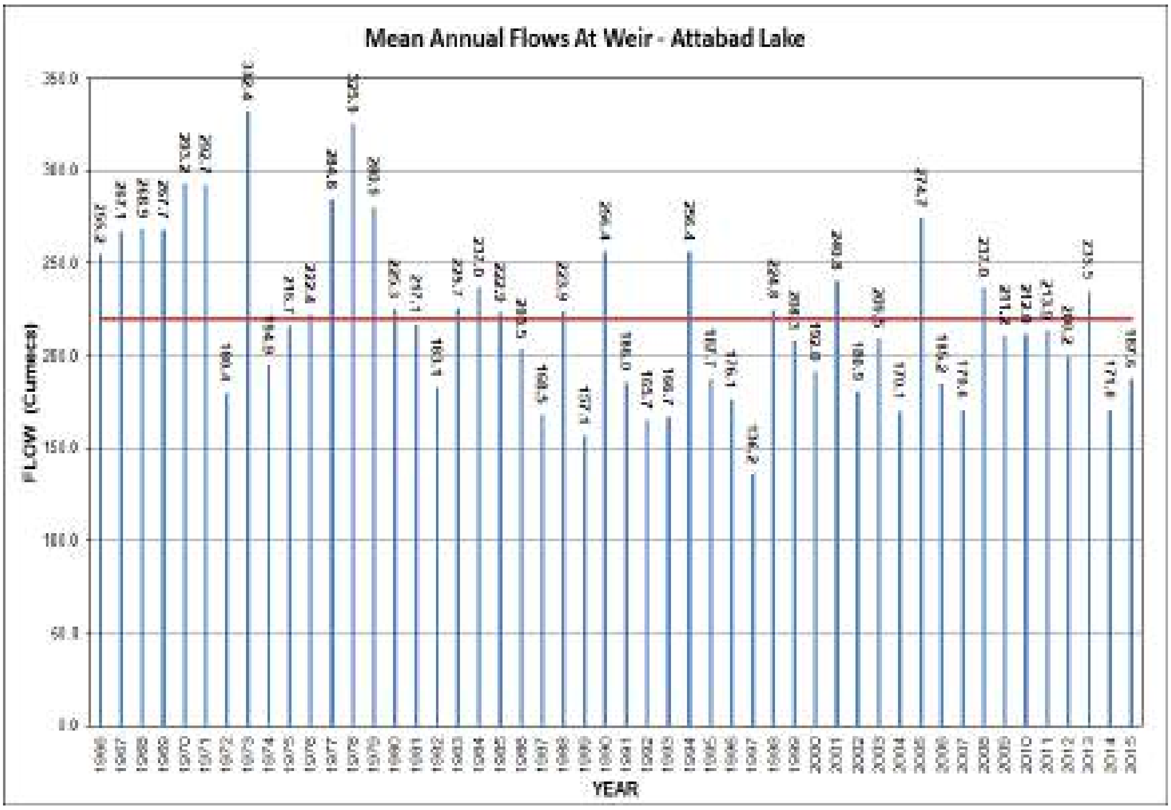


Figure 4.8: Estimated Mean Annual Flows at Attabad Lake - Weir

Table 4.7 Estimated Mean Monthly and Annual Flow at Weir

ESTIMATED MEAN MONTHLY & ANNUAL FLOWS AT WEIR SITE (Cumecs)														
YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	Avg Mean Monthly	Mean Annual
1966	36.86	33.28	24.63	32.94	72.99	626.46	799.82	886.49	422.69	114.27	51.20	41.73	263.44	255.30
1967	38.33	37.48	36.84	47.69	74.16	419.74	996.26	922.54	424.51	96.49	49.16	35.39	264.88	287.06
1968	28.56	26.68	24.55	30.94	85.14	506.54	984.67	1038.76	368.88	91.71	48.87	38.43	267.15	288.93
1969	30.02	28.35	28.29	41.92	100.06	397.81	978.79	1048.37	368.51	126.53	55.67	45.89	265.08	287.74
1970	36.64	31.49	28.97	52.27	136.18	515.12	802.48	1045.38	605.78	151.07	51.91	37.28	291.21	293.38
1971	31.83	30.27	28.87	46.08	170.11	858.81	906.91	898.19	321.09	100.75	53.13	42.56	298.92	303.14
1972	32.72	27.42	26.84	32.63	86.75	336.14	473.29	567.62	373.24	113.22	51.16	37.48	179.81	180.44
1973	32.60	29.06	26.19	46.28	134.99	704.66	1219.56	1048.76	486.41	121.51	59.19	40.44	329.66	332.37
1974	36.34	32.99	32.91	40.87	105.66	303.80	687.38	647.49	244.34	100.80	51.31	36.79	193.21	194.89
1975	30.51	29.39	27.12	36.53	94.74	314.97	637.70	853.78	366.99	108.56	48.94	30.14	214.90	216.68
1976	23.91	23.96	24.82	31.05	132.61	329.29	1028.16	608.71	263.29	87.22	42.31	34.14	220.79	222.40
1977	30.52	28.75	26.90	40.95	114.38	549.17	1211.16	879.98	313.84	109.53	50.37	31.81	282.28	284.54
1978	26.46	26.08	25.57	31.90	150.60	644.84	1267.84	1156.88	355.68	118.71	46.50	30.51	322.89	323.92
1979	37.61	33.83	32.61	54.80	84.38	418.86	1165.75	995.89	257.83	114.68	60.53	40.48	278.10	280.88
1980	37.86	33.39	38.86	50.33	127.66	457.86	779.01	647.43	311.63	115.68	53.02	38.56	231.16	233.36
1981	33.41	27.76	26.35	44.60	216.91	335.18	838.72	679.95	206.83	85.96	46.24	34.59	214.93	217.09
1982	30.04	28.86	26.76	41.08	144.16	325.47	633.01	736.08	187.62	61.90	42.90	37.86	181.43	183.59
1983	36.07	31.60	26.50	31.45	135.96	278.59	602.51	906.75	475.78	88.31	47.64	29.50	223.96	225.71
1984	27.64	25.71	26.35	29.74	51.29	422.80	762.60	1044.16	288.81	70.63	50.14	36.14	235.42	236.98
1985	29.45	27.21	27.26	36.21	80.38	286.57	854.77	796.04	331.24	104.20	43.41	33.94	220.81	222.92
1986	28.78	26.64	25.21	33.77	86.08	284.89	783.71	641.85	278.84	139.50	54.39	37.65	201.66	203.32
1987	31.69	28.26	25.87	29.96	69.67	241.39	427.88	557.17	353.33	111.58	50.39	42.45	167.47	168.49
1988	35.02	29.88	26.33	56.75	150.40	364.83	848.84	693.89	251.38	78.86	54.46	42.28	222.51	223.90
1989	33.59	29.52	27.83	29.82	61.04	264.16	490.34	466.96	271.43	103.85	52.68	40.70	156.00	157.06
1990	35.32	30.98	26.75	30.12	210.95	483.87	777.85	765.15	457.88	137.27	56.12	43.26	254.63	256.11
1991	34.51	31.16	30.39	36.02	74.68	308.44	625.34	680.12	439.07	78.97	43.08	37.83	184.97	186.00
1992	33.56	30.67	27.73	31.64	73.65	213.21	660.32	650.12	261.26	94.53	59.24	40.36	164.71	165.69
1993	30.03	26.96	23.84	48.91	89.46	288.81	634.89	450.44	286.63	106.07	54.93	36.90	165.59	166.68
1994	31.87	27.83	26.10	28.45	127.67	363.41	1001.51	928.29	335.75	89.63	49.62	36.91	253.91	256.42
1995	31.06	29.19	25.89	29.73	102.91	266.57	718.16	688.23	189.78	71.73	42.92	34.89	185.79	187.66
1996	27.72	24.41	23.45	31.87	51.54	315.11	534.34	622.02	259.42	94.30	43.81	34.90	175.24	178.09
1997	33.70	31.94	28.19	40.31	72.65	199.21	444.60	383.13	265.72	96.51	48.55	38.11	135.22	136.30
1998	31.16	31.24	28.63	36.14	144.63	490.30	781.73	551.63	367.19	114.81	59.81	43.33	223.43	224.84
1999	31.47	29.08	28.96	37.23	94.04	348.49	692.91	798.35	261.15	82.36	44.65	38.59	206.47	208.33
2000	32.31	26.06	24.52	38.81	233.94	388.59	555.36	620.89	385.88	80.03	55.18	35.80	191.12	193.47
2001	31.92	28.97	26.94	51.21	217.66	508.86	888.70	671.60	259.25	99.42	46.08	33.71	238.66	240.81
2002	29.56	28.29	27.33	38.67	110.86	377.49	471.33	693.65	278.81	107.33	48.17	36.83	179.70	180.88
2003	31.76	27.90	23.62	37.71	84.98	393.17	796.16	647.60	361.62	90.84	61.87	41.29	208.13	209.55
2004	35.22	32.31	32.84	49.42	131.79	296.66	468.34	490.36	364.61	96.76	54.86	41.69	169.42	170.10
2005	57.11	52.47	39.89	60.36	214.13	637.87	1201.46	514.42	284.98	104.13	56.59	50.83	272.72	274.87
2006	38.70	37.39	35.86	42.11	258.63	321.86	499.90	525.17	287.11	103.24	73.81	61.76	183.79	183.34
2007	52.41	38.85	36.81	38.40	211.55	488.55	427.20	339.22	155.56	103.75	57.96	48.11	169.80	170.80
2008	43.59	44.31	46.41	56.37	144.10	610.86	747.74	648.62	281.36	85.64	71.90	53.68	236.16	237.35
2009	43.17	41.98	40.82	43.41	97.84	318.13	807.99	681.89	294.18	176.11	108.61	61.50	208.62	211.30
2010	22.49	17.19	15.83	31.60	87.76	290.18	672.25	839.02	326.69	116.26	76.87	46.89	218.16	212.85
2011	67.64	44.79	67.89	62.11	129.01	475.80	556.65	603.35	353.35	143.65	92.82	67.76	212.66	213.87
2012	47.14	59.63	42.78	50.37	122.40	282.86	601.69	622.10	375.18	91.67	56.60	40.83	199.34	200.28
2013	44.71	33.40	40.89	52.49	147.06	538.30	669.97	641.65	386.86	138.84	70.34	46.72	234.20	235.55
2014	37.56	35.11	30.22	34.11	52.35	248.75	543.07	554.98	241.02	110.59	64.32	43.50	169.63	170.96
2015	31.13	27.71	28.19	43.75	118.08	236.58	610.99	698.59	267.75	113.39	70.75	42.14	185.75	187.38
Mean	**34.84**	**31.54**	**29.35**	**40.61**	**122.23**	**395.40**	**739.20**	**709.09**	**317.94**	**104.86**	**55.73**	**40.57**	**218.48**	**220.08**

- **Daily Flow Duration Curve at Weir Site**

Daily flow duration curve for the period of 1966-2015 is developed by using daily estimated flows at proposed weir site are tabulated below in Table 4.8 and shown graphically in Figure 4.9. The flow duration curve represented the required discharge of 60 cumecs is available at 52 % of time availability.

Table 4.8 Daily Flow Duration Curve at Weir Site

% of Time	Flow (m^3/s)	% of Time	Flow (m^3/s)
1%	1244.43	55%	54.13
5%	877.51	60%	47.02
10%	660.62	65%	41.75
15%	522.35	70%	38.22
20%	408.61	75%	35.08
25%	306.85	80%	32.45
30%	222.77	85%	30.15
35%	158.58	90%	28.04
40%	112.19	95%	25.93
45%	81.43	100%	12.79
50%	64.91		

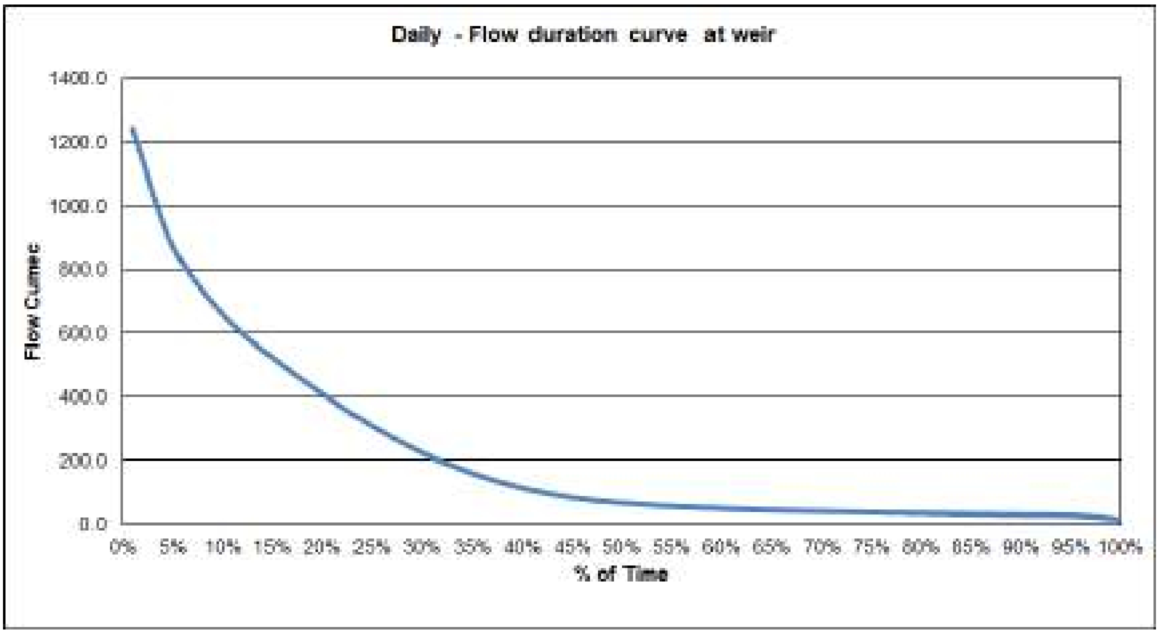


Figure 4.9: Net Daily Flow Duration Curve at Weir Site

4.5.3 Water Availability and Reservoir Operational Study - Review Study

Mean annual water availability at weir site is 220.08 cumecs and in volume 6940 MCM. Desired design discharge is 60 cumecs for the installed capacity of 54MW. Three turbines are suggested each of 20 cumecs design discharge capacity. Weir crest elevation as suggested in feasibility study is 2414 a.m.s.l and having gross storage capacity of 147 MCM against 2414 a.m.s.l according to elevation area capacity curve (EAC). But in feasibility study, gross storage capacity of Attabad Lake is taken as 125 MCM. So, in present study consider same gross storage capacity as reported in feasibility study.

Turbines run with full desired discharge capacity of 60 cumecs from May to October. There is a shortage from November to April and these shortages will be reducing by the utilization of stored water in reservoir. It means reservoir start ponding in May and store water of 125 MCM up to gross storage level in the same month.

From May to October reservoir storage remain constant. Reservoir storage of 125 MCM will be utilize for six months from November to April according to month wise water demand for power generation. Three turbines of each 20 cumecs design discharge capacity to be suggested to fulfill the required design discharge capacity of 60 cumecs. Turbine run with full design discharge capacity at 20 cumecs availability of flow for that turbine, otherwise turbine will be functional at certain limit of low flows availability depend upon turbine design requirement. It may be 40% to 50% of Turbine design discharge as per design parameters and below this flows limit, turbine will not be functional. So, also need for consideration of minimum flow requirement to keep turbines in functional and operative position.

Invert level of Power Intake is at 2404m with storage capacity of 101.87MCM and weir crest level 2414m with storage capacity of 147MCM as per EAC. Live storage (147MCM-101.87MCM=45.13MCM) would be utilized during lean period to reduce shortages. The detail Water availability and Reservoir Operational System for power potential is tabulated below in Table 4.9.

Table 4.9 Water Availability and Reservoir Operation for Power Potential

WATER AVAILIBILTY AT ATTABAD LAKE													
Months	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	Mean Annual
Available Flows in Cumecs	34.64	31.54	29.96	40.61	122.23	395.40	739.20	709.09	317.94	104.86	55.73	40.57	228.08
Available Flows in Volume MCM	92.8	76.3	80.2	105.3	327.4	1024.9	1979.9	1899.2	824.1	280.9	144.5	108.7	6949
SHORTAGES FOR UTILIZATION OF 60 CUMECS DESIGN DISCHARGE FOR INSTALLED CAPACITY OF 54 MW													
Shortages in Cumecs	25.36	28.46	30.04	19.39							4.27	19.43	21.16
Shortages in Volume (MCM)	67.92	68.86	80.50	50.26							11.06	52.03	330.63
RELEASES AFTER UTILIZATION OF 60 CUMECS DESIGN DISCHARGE FOR INSTALLED CAPACITY OF 54 MW													
Releases in Cumecs					15.56	335.40	679.20	649.09	257.94	44.86			
Releases in Volume (MCM)					41.7	869.4	1819.2	1738.5	668.6	120.2			

MONTHLY RESERVIOR OPERATION FOR POWER POTENTIAL ON THE BASIS OF WATER AVAILIBILITY						
MONTH	Availabale Live Storage in MCM	Storage Utilization in MCM	Remaining Storage in Reservior MCM	No of Turbines in Operational	No of Turbines Non Operational	Remarks
NOV	45.13	11.06	34.07	3	0	Three Turbines run with full design discharge capacity
DEC	34.07	34.07	0.00	3	0	Two Turbines run with full design discharge capacity of 20 cumecs and one with low flow 13 cumecs
JAN	0.00	0.00	0.00	2	1	One Turbines run with full design discharge capacity and other with low flow of 14 cumecs
FEB	0.00	0.00	0.00	2	1	One turbine run with full design discharge of 20 cumecs and other run with low flows of 11.54 cumecs
MAR	0.00	0.00	0.00	2	1	One turbine run with full design discharge of 20 cumecs and other run with low flows of 9.95 cumecs
APRIL	0.00	0.00	0.00	2	1	Two Turbines run with full design discharge capacity of 20 cumecs
May	Availabale Reservior Storage in MCM	Reservior start Pending and fill up to Gross Sotrage in MCM	Remaining Storage in Reservior MCM	No of Turbines in Operational	No of Turbines Non Operational	
	0.00	125.00	125.00	3	0	After diverting of 60 cumecs required discharge, reservior start to be Filling and full with storage capcity of 125MCM and remaining flow of 15.56 cumecs spill
	Availabale Reservior Storage in MCM	Storage Utilization in MCM	Remaining Storage in Reservior MCM	No of Turbines in Operational	No of Turbines Non Operational	
JUNE	125.00	0.00	125.00	3	0	Three Turbines run with full design discharge capacity, flows Spill 335.4 cumecs
JULY	125.00	0.00	125.00	3	0	Three Turbines run with full design discharge capacity, flows Spill 679.2 cumecs
AUG	125.00	0.00	125.00	3	0	Three Turbines run with full design discharge capacity, flows Spill 649 cumecs
SEP	125.00	0.00	125.00	3	0	Three Turbines run with full design discharge capacity, flows Spill 257.9 cumecs
OCT	125.00	0.00	125.00	3	0	Three Turbines run with full design discharge capacity, flows Spill 44.86 cumecs

4.5.4 Comparison of Flows Study (Feasibility Study vs. Review Study)

The comparison of flows study b/w Feasibility Study 2021 and Review Study 2022 is shown graphically in Figure 4.10, Figure 4.11 and Figure 4.12.

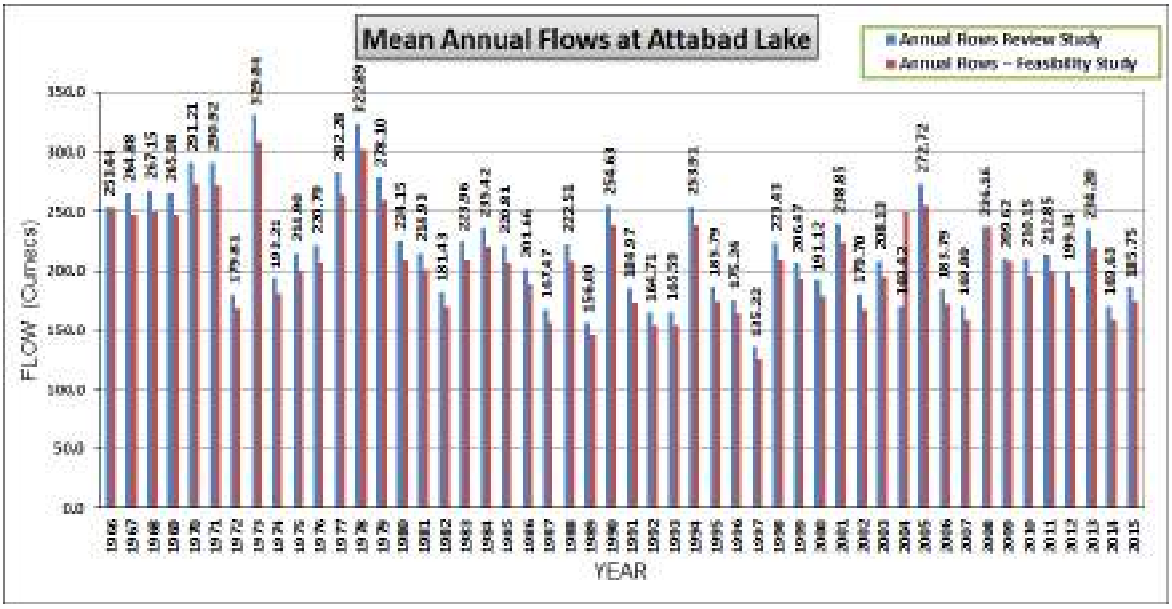


Figure 4.10: Comparison of Annual Flows (Review Study vs Feasibility Study)

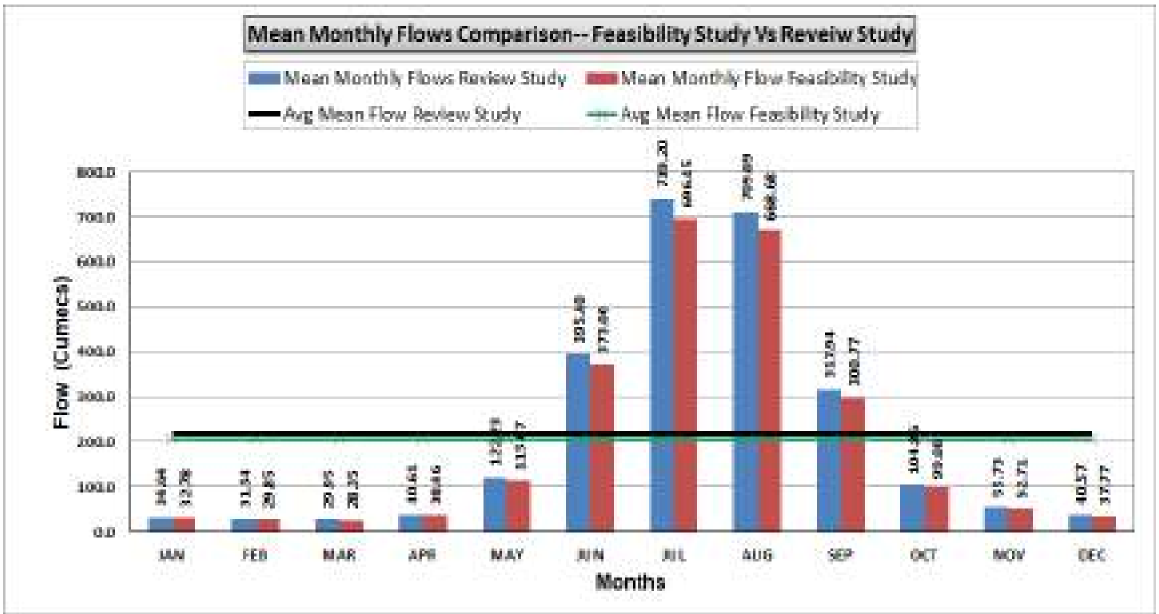


Figure 4.11: Comparison of Monthly Flows (Review Study vs Feasibility Study)

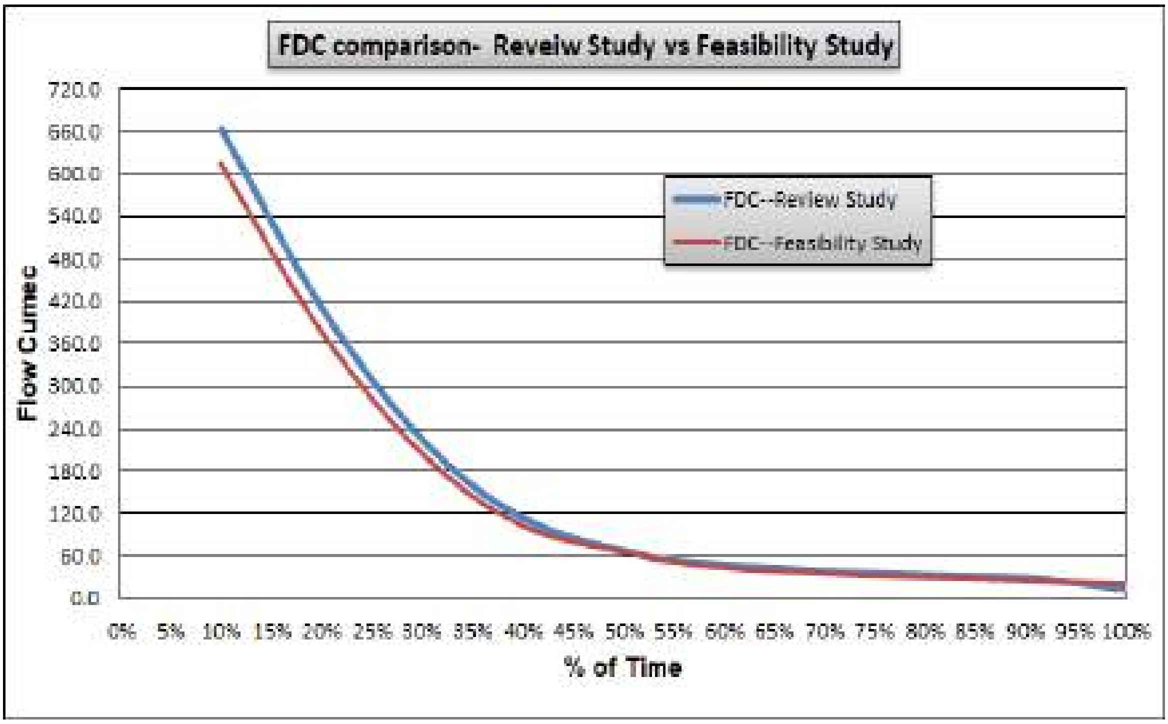


Figure 4.12: Comparison of FDC (Review Study vs. Feasibility Study)

4.6 Flood Study

Flood Frequency Analysis can be performed where maximum annual instantaneous flow values are available to estimate flood magnitudes for various return periods.

4.6.1 Flood Estimation - Feasibility Study 2021

In Study 2021, flood estimated by using two approaches as mentioned below.

- Flood Estimation by Flood Frequency Analysis
- Flood Estimation by USACE HEC-SSP Model

Flood Frequency Analysis (FFA) had been carried out to estimate the flood at different return periods. Three types of distribution series namely Gumble, Normal and Log Pearson Type-III Distribution series have been used to estimate the flood. Flood has also been estimated using the statistical model. The US Army Corps of Engineers (US-ACE) HEC SSP Model is used for this purpose. The results of flood estimated against different return periods at Attabad Lake by using two approaches are tabulated below in Table 4.10. It was recommended to adopt design flood of 10,000 years return period at weir site. Design flood of (3,250 m³/s) at weir site had been assumed for the calculations of design of structures.

Table 4.10 Flood Estimation Results - Attabad Lake

Flood Estimation	Flood Frequency Analysis -Results			HEC-SSP Model Result	
Return Period (years)	**Gumble Distribution (m^3/s)**	**Normal Distribution (m^3/s)**	**Log Pearson Type-III (m^3/s)**	**Computed Flood from Curve (m^3/s)**	**Expected Probable Flood (m^3/s)**
2	1,142	1,232	1,155	1,154	1,154
5	1,411	1,426	1,428	1,431	1,436
10	1,589	1,579	1,596		
20				1,759	1,781
25	1,813	1,767	1,796		
50	1,980	1,910	1,939		
100	2,145	2,073	2,077	2,098	2,153
200				2,238	2,314
1,000	2,691	2,520	2,518	2,559	2,698
10,000	**3,237**	**2,984**	**2,915**	3,016	**3,292**

4.6.2 Flood Estimation - Review Study

Flood has been estimated from the available record. The maximum peak daily discharges of Dainyor Bridge station at Hunza Rivers for every year recorded. The instantaneous peaks will definitely be higher than the measured daily flows. Instantaneous peak flows of Dainyor Bridge station for the period 1966 to 2015 have been calculated and generated one day maximum peak Instantaneous flow for every year at proposed weir site. The estimated instantaneous peak flow for the period of 1966-2015 at Attabad Lake is graphically represented in Figure 4.13.

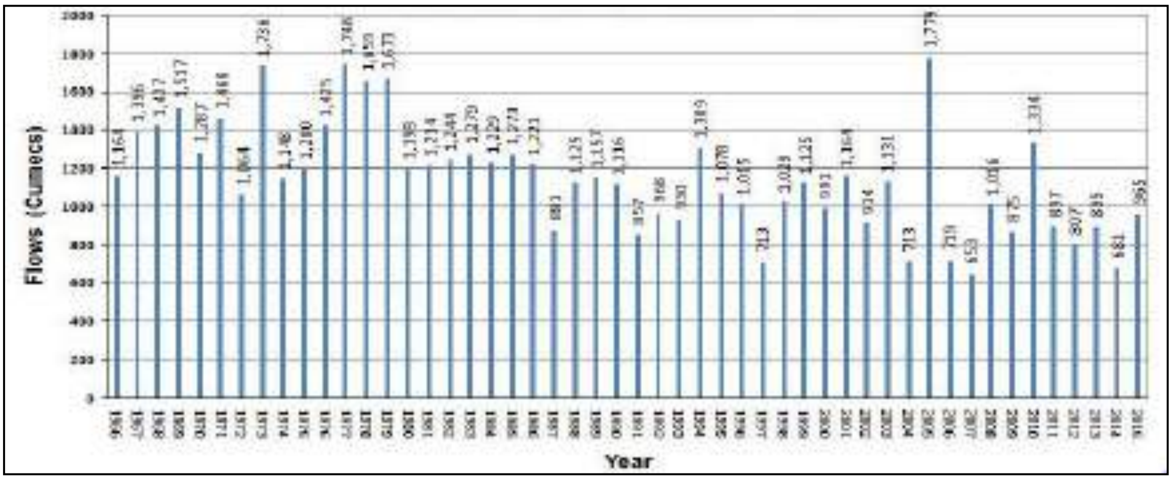


Figure 4.13: Instantaneous Peak Flows at Attabad Lake (1966-2015)

- **Flood Estimation at Attabad Lake - Weir**

Flood has been estimated at propose weir site by using Peak Instantaneous flows of Attabad Lake for period of 1966-2015. Flood Frequency Analysis (FFA) has been carried out by using different distribution (Gumble, Lognormal & Pearson type 3). Flood frequency results against different return periods are tabulated and represented below in Table 4.11 and Figure 4.14 through Figure 4.16.

Table 4.11 Floods against Different Return Period - Attabad Lake Weir

Flood Estimation at Weir by Flood Frequency Analysis			
Return Period (years)	**Gumbel (cumecs)**	**Lognormal (cumecs)**	**Log-Pearson type 3 (cumecs)**
10000	**3320**	**2840**	**2580**
2000	2910	2550	2370
1000	2740	2420	2270
500	2570	2300	2170
200	2340	2130	2040
100	2160	2000	1930
50	1990	1870	1820
20	1750	1680	1660
10	1570	1540	1520

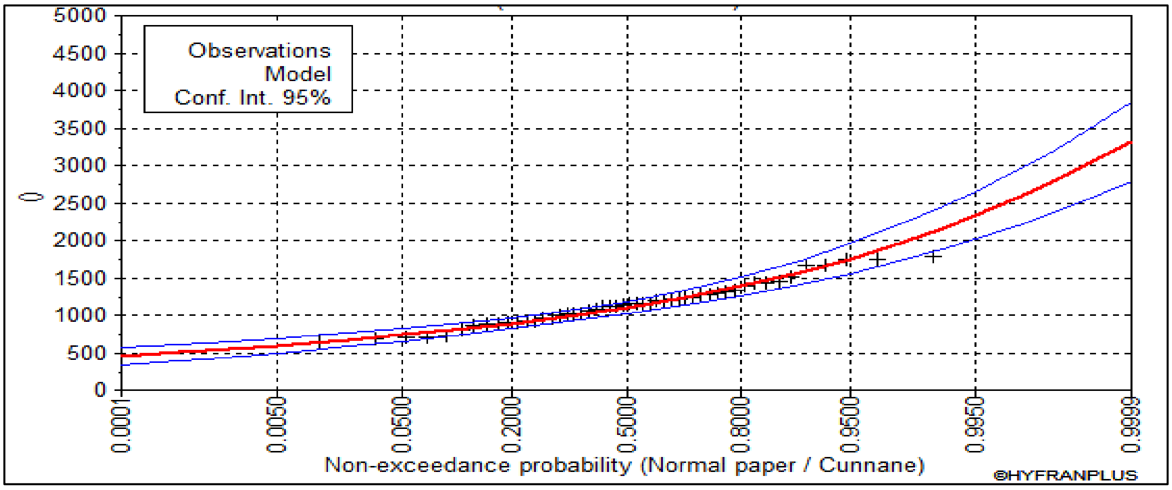


Figure 4.14: Flood Frequency at Weir - Gumble Distribution

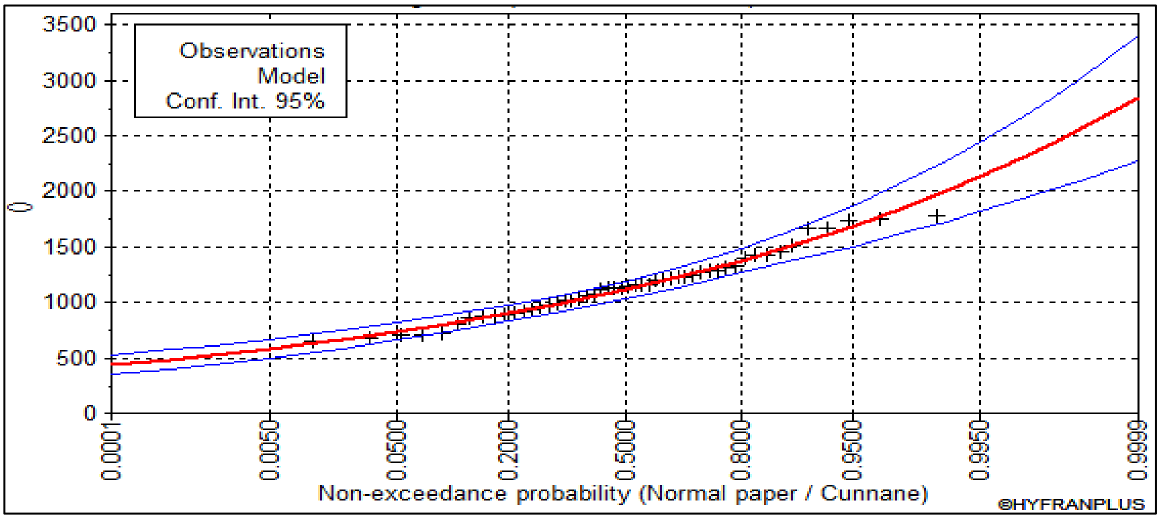


Figure 4.15: Flood Frequency at Weir - Lognormal Distribution

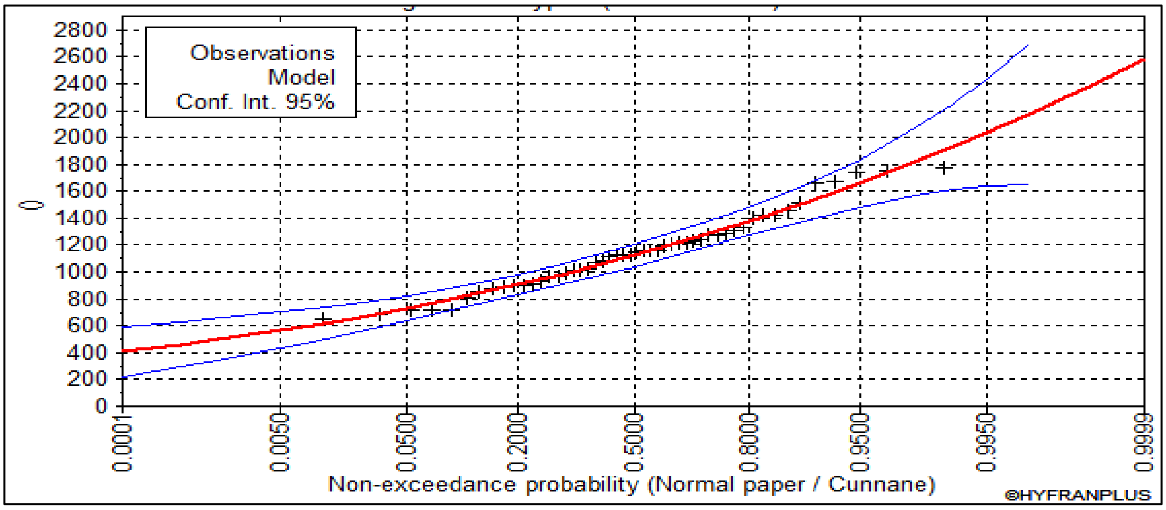


Figure 4.16: Flood Frequency at Weir - Pearson Type 3 Distribution

- **Recommended Design Flood**

Keeping in view the flood estimation against returns periods by using different distributions, it is recommended to adopt Gumble Distribution against 10,000 years of return period for the selection of design flood at proposed weir site.

Finally, Design Flood of 3320 m³/s at weir site has been selected for the design of hydraulic structures.

4.7 HYDROLOGICAL FLOOD ROUTING

Flood routing is a mathematical procedure for predicting the changing magnitude, speed, and shape of a flood wave as a function of time (i.e., the flow hydrograph) at one or more points along a waterway. Although, the lake has not much storage capacity but still it can lag and attenuate the peak generated by floods to minimize the adverse impacts on the downstream structures and areas. The temporarily stored water is then released through an outlet or spillway after the peak has passed.

4.7.1 Flood Routing - Feasibility Study 2021

To setup the HEC-HMS model, Storage-Discharge relationship had been developed for the reservoir. The hydrograph with the peak of 3,250 m³/s had entered in the lake as inflow and attenuated peak of 3,085 m³/s was attained after 4 hours of the inflow peak. The attenuated peak was attained at an elevation of 2421.3 masl. The inflow hydrograph of 3,250 m³/s , the rating curve of overflow spillway and flood routing of Attabad Lake are presented in Figure 4.17, Figure 4.18 and Figure 4.19.

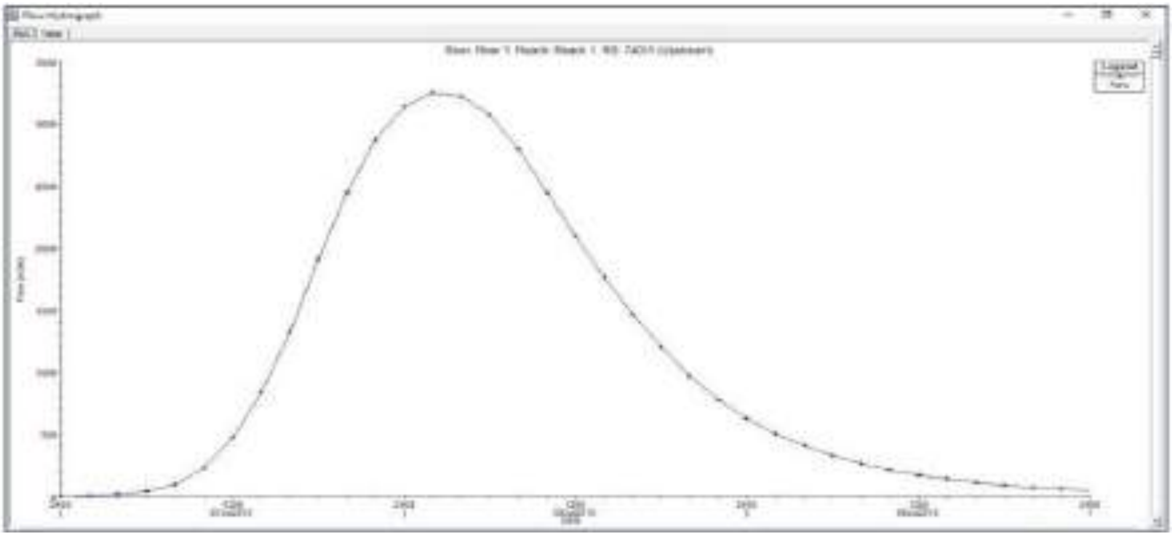


Figure 4.17: Flow Hydrograph of 3250 Cumecs

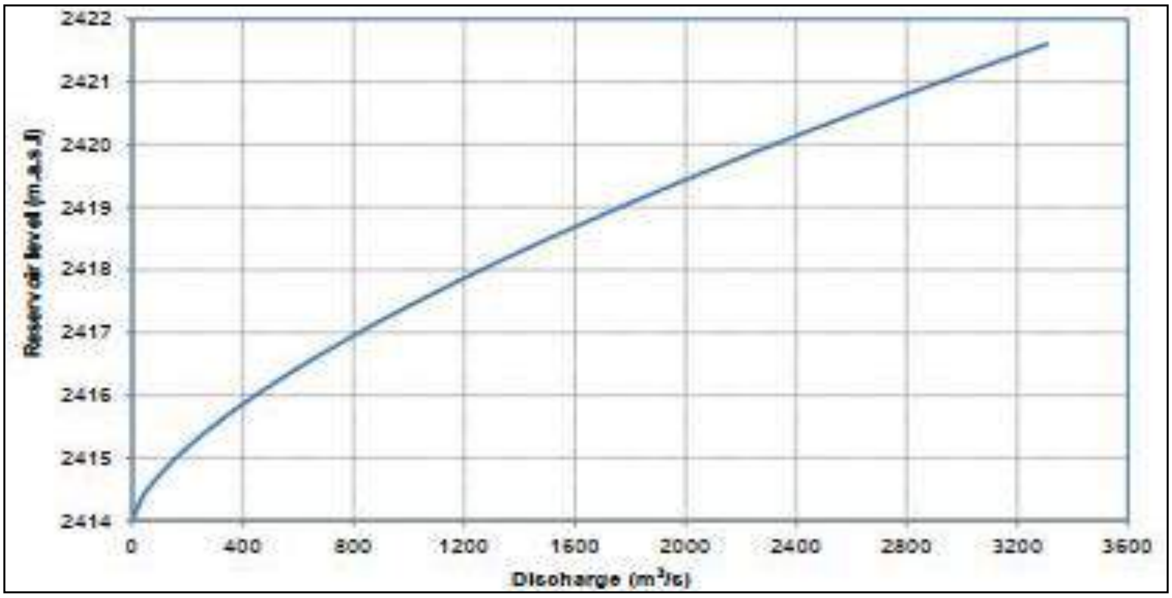


Figure 4.18: Rating Curve of Spillway

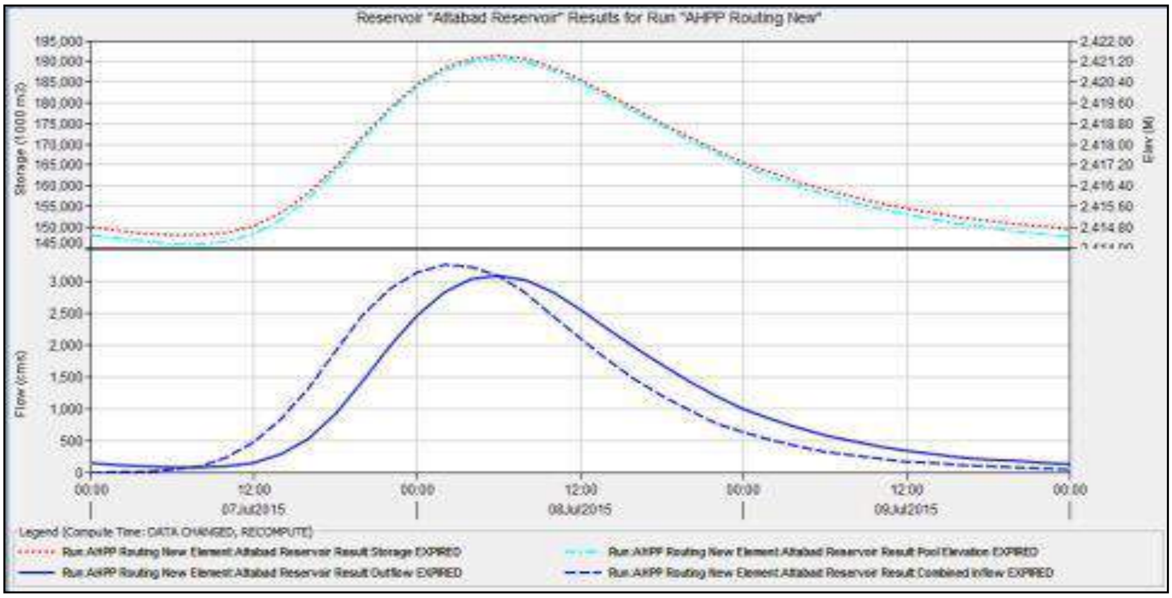


Figure 4.19: Attabad Lake Flood Routing Feasibility Study

4.7.2 Flood Routing - Review Study

In present study, for developing HEC-HMS Model, the model input parameters (Capacity Storage relationship and Elevation Area Capacity Curve) keeping same as in feasibility study. Taking catchment area 8922.9 sq.km and peak inflow 3320 m³/s. Assuming spillway crest level as 2414 m a.m.s.l. The inflow hydrograph of (3250 m³/s as in Feasibility Study) is converted into Unit Hydrograph by unity method and generated Inflow Hydrograph of 3320 m³/s on the basis of Unit Hydrograph. Unit hydrograph and Inflow Hydrograph of 3320 cumecs are as given in Figure 4.20 and Figure 4.21.

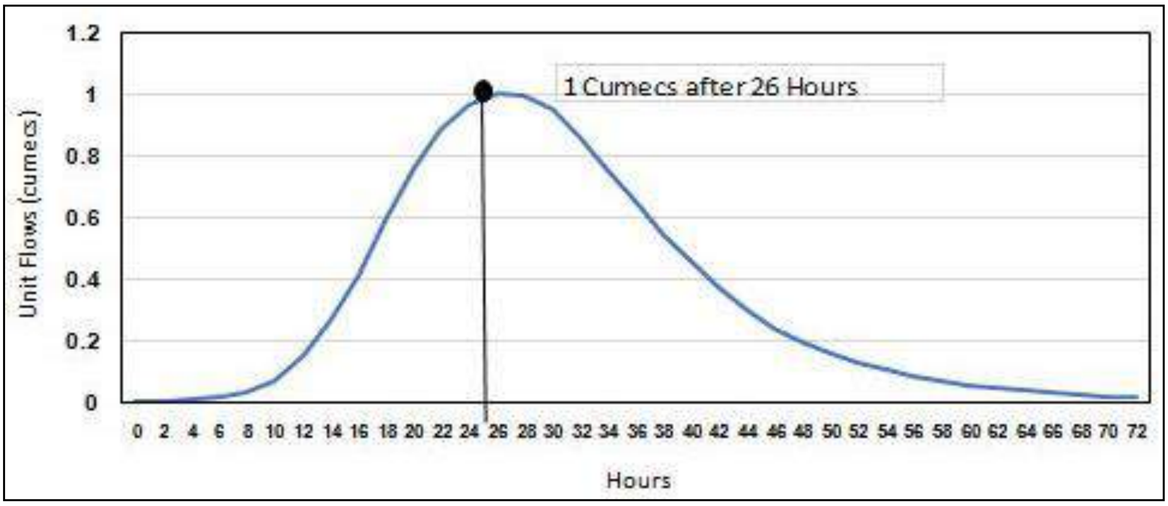


Figure 4.20: Unit Hydrograph

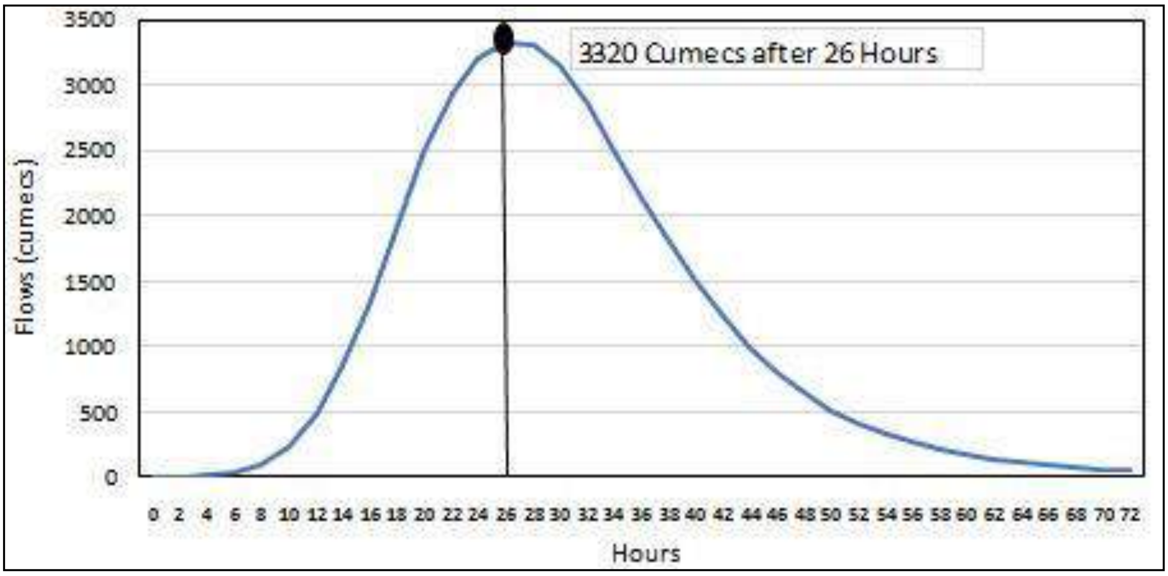


Figure 4.21: Flow Hydrograph of 3320 Cumecs

HEC-HMS, model run by considering inflow hydrograph with the peak of 3,320 m³/s has been entered in the lake as inflow and attenuated peak of 3,152.8 m³/s is attained after 4 hours of the inflow peak. The attenuated peak is attained at an elevation of 2421.4 m a s l. HEC-HMS results are shown below in Figure 4.22.

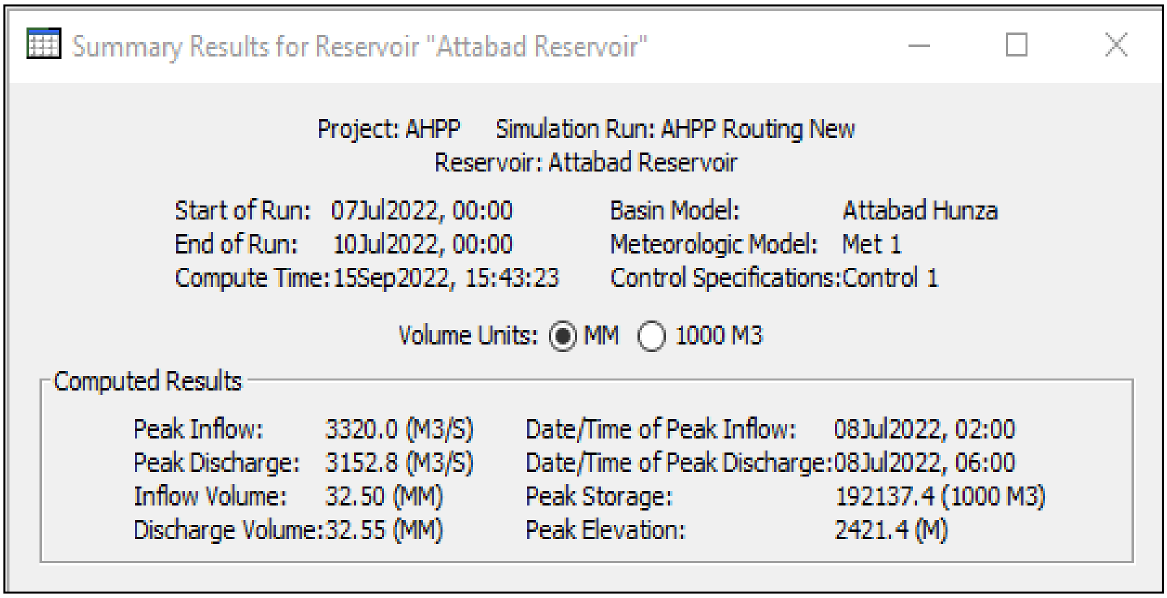


Figure 4.22: HEC HMS Model - Simulation Results

Attabad Lake Flood Routing results are given in Figure 4.23 and tabulated in Table 4.12.

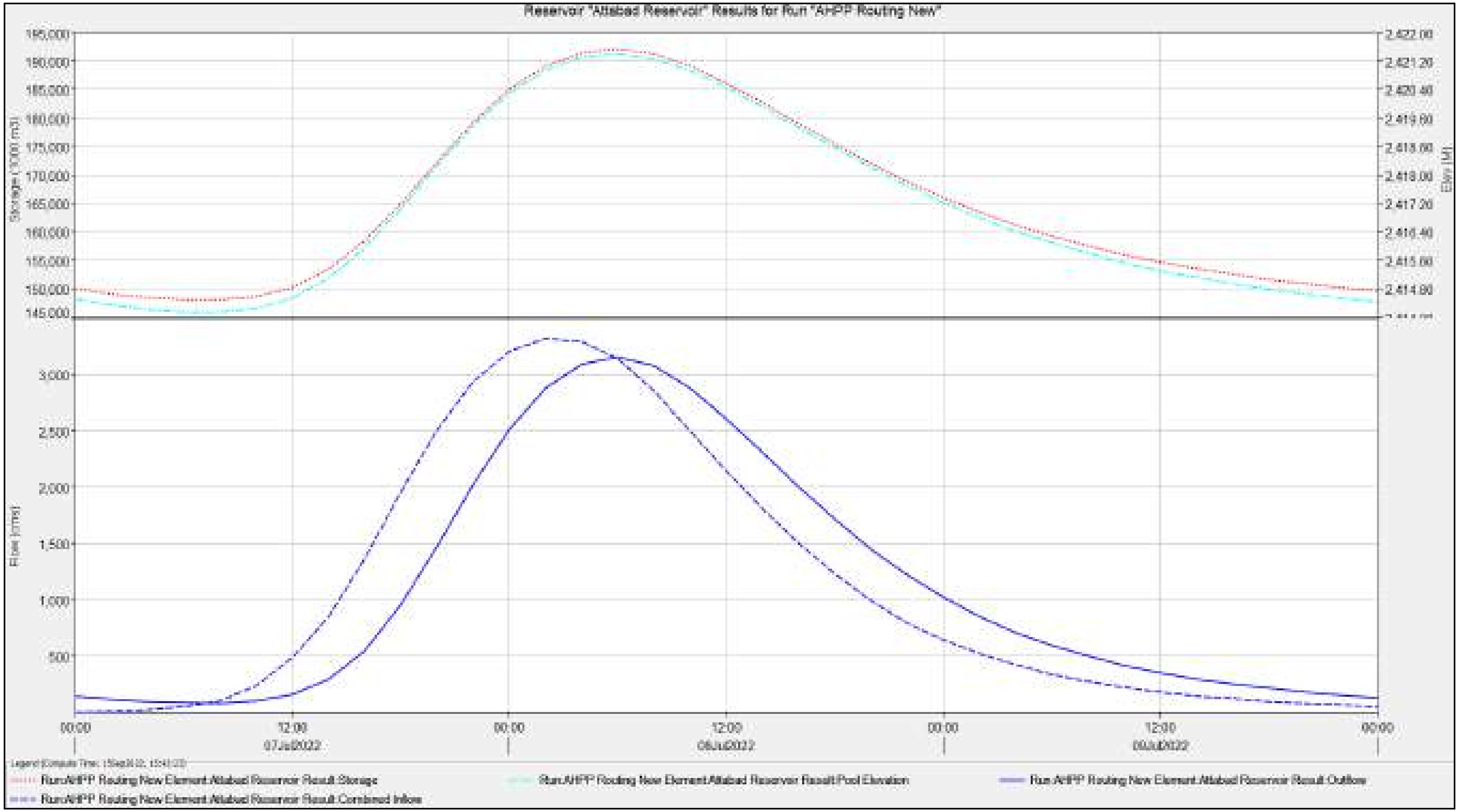


Figure 4.23: Attabad Lake Flood Routing - Review Study

Table 4.12 Flood Routing Results - Attabad Lake

Project: AHPP Simulation Run: AHPP Routing New
Reservoir: Attabad Reservoir

Start of Run: 07Jul2022, 00:00 Basin Model: Attabad Hunza
End of Run: 10Jul2022, 00:00 Meteorologic Model: Met 1
Compute Time: 15Sep2022, 15:43:23 Control Specifications: Control 1

Date	Time	Inflow (M3/S)	Storage (1000 M3)	Elevation (M)	Outflow (M3/S)
07Jul2022	00:00	0.0	150000.0	2414.5	142.4
07Jul2022	02:00	3.9	149094.6	2414.3	112.9
07Jul2022	04:00	17.0	148433.5	2414.2	91.5
07Jul2022	06:00	44.3	148041.1	2414.2	78.8
07Jul2022	08:00	96.3	147986.4	2414.1	77.0
07Jul2022	10:00	229.9	148541.3	2414.2	95.0
07Jul2022	12:00	481.8	150216.4	2414.5	151.4
07Jul2022	14:00	849.9	153429.0	2415.1	288.0
07Jul2022	16:00	1353.2	158372.5	2415.9	542.0
07Jul2022	18:00	1948.2	164910.0	2417.0	943.5
07Jul2022	20:00	2502.0	172256.3	2418.2	1466.1
07Jul2022	22:00	2928.9	179261.0	2419.4	2019.1
08Jul2022	00:00	3205.3	185036.7	2420.3	2510.8
08Jul2022	02:00	3320.0	189134.9	2420.9	2876.1
08Jul2022	04:00	3291.6	191458.6	2421.3	3090.0
08Jul2022	06:00	3139.7	192137.4	2421.4	3152.8
08Jul2022	08:00	2853.8	191294.7	2421.3	3074.8
08Jul2022	10:00	2494.0	189126.1	2420.9	2875.3
08Jul2022	12:00	2137.5	186080.9	2420.5	2602.0
08Jul2022	14:00	1800.6	182609.9	2419.9	2300.3
08Jul2022	16:00	1495.2	179001.5	2419.3	1997.9
08Jul2022	18:00	1225.5	175442.3	2418.8	1711.5
08Jul2022	20:00	989.5	172036.6	2418.2	1449.5
08Jul2022	22:00	792.1	168853.5	2417.7	1216.3
09Jul2022	00:00	637.0	165964.7	2417.2	1015.3
09Jul2022	02:00	514.4	163407.0	2416.8	846.6
09Jul2022	04:00	414.7	161163.7	2416.4	705.7
09Jul2022	06:00	334.4	159198.8	2416.1	589.2
09Jul2022	08:00	269.7	157476.2	2415.8	493.4
09Jul2022	10:00	217.5	155971.2	2415.5	411.9
09Jul2022	12:00	175.6	154653.9	2415.3	347.1
09Jul2022	14:00	141.8	153498.3	2415.1	291.3
09Jul2022	16:00	114.3	152489.0	2414.9	245.2
09Jul2022	18:00	92.1	151599.0	2414.8	208.4
09Jul2022	20:00	74.5	150814.5	2414.6	176.1
09Jul2022	22:00	60.4	150133.6	2414.5	148.0
10Jul2022	00:00	49.0	149536.8	2414.4	127.3

4.8 SEDIMENTATION STUDY

Sedimentation study is prerequisite for the design of any hydropower project. Sediment transport studies are essential to estimate total sediment inflow at the weir site, sediment load and the life of the reservoir with and without flushing or sluicing options and all other aspects related to sedimentation.

4.8.1 Sedimentation Study - Feasibility Study 2021

Suspended sediments at Attabad Lake were estimated by using sediment data of Dainyor Bridge station. Relationship developed between suspended sediment and discharge data of Dainyor Bridge. Sediment rating curve developed with that correlation and generated suspended sediment yield at proposed weir site with the application of developed sediment rating curve. Average annual suspended sediment estimated as 20.06MMT.

Average Volume of fresh sediment deposits for Attabad Lake HPP is estimated as: (20.06 MMT x 1000 kg/MT) / (1122.4 kg/m³ x 1234 m³/acre-feet x 106) ~ 14,483 Acre-Feet/year. Bed load had been considered as 20% of the total suspended load. Total Sediment load at weir site worked out to be 24.06 M Tons. Without flushing, reservoir would be filled with sediments in 10 years.

4.8.2 Sedimentation Study - Review Study

- **Estimation of Suspended Sediment at Weir**

Estimation of suspended sediment at weir site by using two approaches

  - o Observed Dainyor Bridge Sediment Data
  - o Development of Sediment Rating Curve

1st Approch: Observed Sediment Data of Dainyor Bridge

Observed annual suspended sediment data of Dainyor Bridge at Hunza river is collected from SWHP-WAPDA, for the year of 1966-2005 is tabulated below in Table 4.13.

Table 4.13 Observed Suspended Sediment at Dainyor Bridge

Suspended Sediment Yield at Hunza River (Dainyor Bridge) - SWHP WAPDA			
Year	**Sediments (m.s.t)**	**Year**	**Sediments (m.s.t)**
1966	74.1	1986	34.2
1967	76.2	1987	23.3
1968	81.2	1988	39.6
1969	78.2	1989	5.59
1970	85.3	1990	12.9
1971	86.9	1991	7.73
1972	32.2	1992	11.2
1973	111	1993	10.2
1974	40.6	1994	14.1
1975	31.2	1995	15.5
1976	31.3	1996	20.5

Suspended Sediment Yield at Hunza River (Dainyor Bridge) - SWHP WAPDA			
Year	**Sediments (m.s.t)**	**Year**	**Sediments (m.s.t)**
1977	56.1	1997	10.8
1978	59	1998	19.7
1979	47.7	1999	18.3
1980	33.3	2000	19.2
1981	31.4	2001	20.4
1982	29.2	2002	12.8
1983	33.7	2003	17.5
1984	46.4	2004	11.04
1985	39.3	**Avg**	**36.64**

The average annual suspended sediment at Dainyor Bridge estimated from the above table is 36.64 MTons. As Attabad Lake is located at upstream of Dainyor Bridge on Hunza river, average annual suspended sediment estimated at Attabad Lake by using catchment area ratio method. Thus, average annual suspended sediment at proposed weir site is worked out to be **24.85 MTons.**

2nd Approch: Development of Sediment Rating Curve

Observed daily suspended sediment data w.r.t discharges of Dainyor Bridge gauging station have been collected from SWHP-WAPDA, for the period of 1966-2015. Best correlation developed between suspended sediments with discharges for the period of 1967-1988 as represented in Figure 4.24.

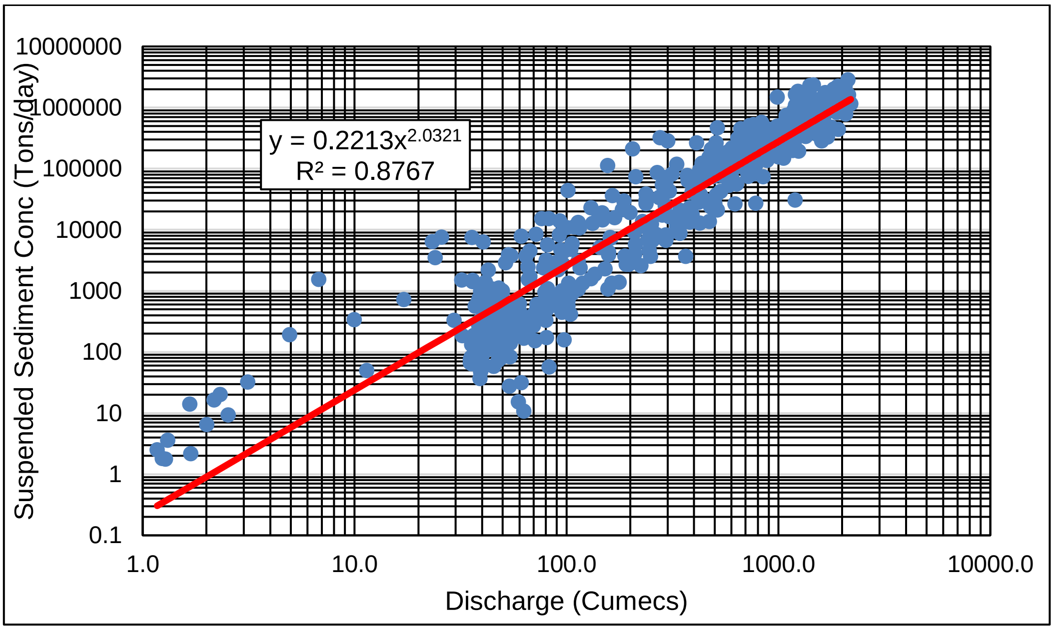


Figure 4.24: Correlation between Suspended Sediment and Discharge - Dainyor Bridge

The best fit relationship thus obtained was:

$$y = 0.221x^{2.032}$$

For the safety and life of project, appropriate estimation of sediment yield at weir site is needed. Therefore, keeping on safer side, select the years having above average estimated inflows at Attabad Lake. Putting the value of estimated available flows at weir site (x) in the above equation will give the suspended sediment (y) at weir. The mean monthly and mean annual suspended sediment at Attabad Lake are represented below in Figure 4.25 & Figure 4.26.

Average monthly suspended sediments at weir site are varying from 7721 Tons in March to 8,256,082 Tons in July. Average annual suspended sediment estimated at weir site is **20.30 MTons**.

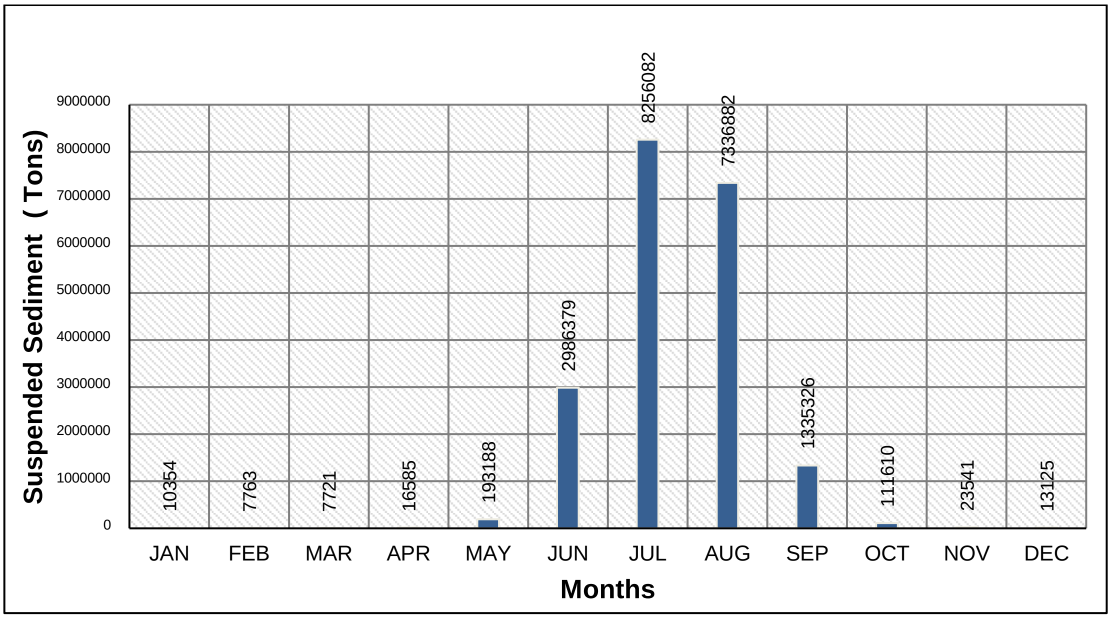


Figure 4.25: Monthly Suspended Sediment at Attabad Lake

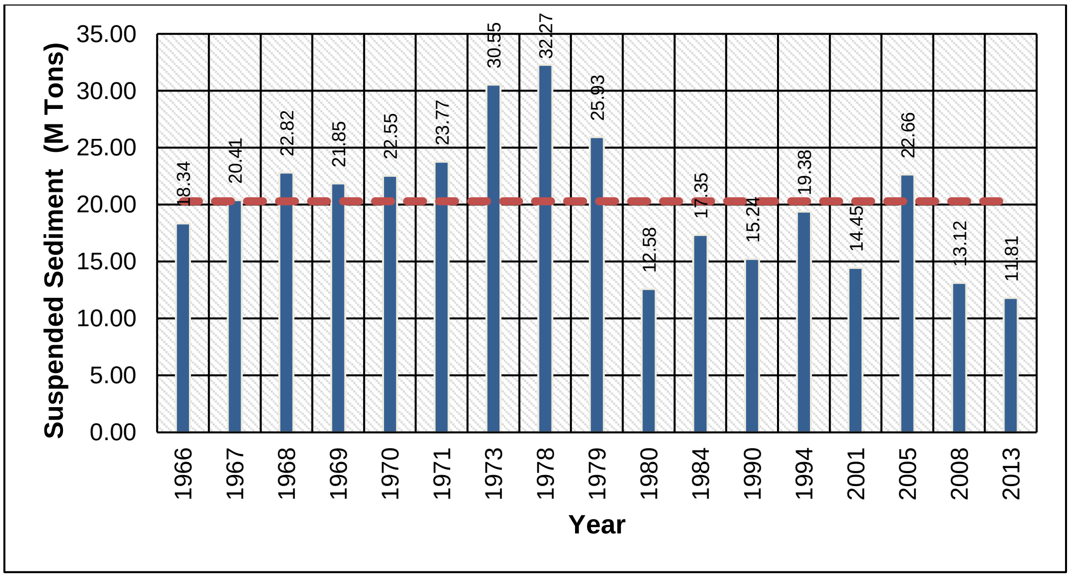


Figure 4.26: Annual Suspended Sediment at Attabad Lake

Average annual suspended sediment yield at weir site estimated with values of 24.85 MTons and 20.30 MTons against 1st and 2nd approach respectively. Keeping in view the above two approaches, finally estimated annual suspended sediment yield at Attabad Lake is taking average of above two values (24.85 MTons and 20.30 MTons). The resulted average annual suspended sediment at weir site is 22.5 MTons.

- **Estimation of Total Sediment Load**

Total annual sediment load at Attabad Lake is addition of suspended sediment load and Bed load at weir site. Bed load is considered as 20% of suspended sediment load. Annual suspended sediment load at weir site is 22.5 MTons. Bed load is 20% of Suspended load and Bed load worked out to be 4.51 MTons. **Total annual sediment load in Attabad Lake is estimated with value of 27.08 MTons.** Density of sediment is 62 lb/ft^3. Total annual sediment load in volume is **27.71 MCM** or **22,466 Ac FT**.

4.9 SUMMARY OF REVIEW STUDY

Comparative summary of Review w.r.t Feasibility Study is as shown below in Table 4.14.

Table 4.14 Comparative Summary of Feasibility vs. Review Study

Sr. No.	Description	Feasibility study 2021	Review Study 2022
1	Catchment area	8901 km^2	8922.9 km^2
2	Climatic Data	Gilgit	Gilgit & Skardu
3	Flows Data	Daily flows Dainyor Bridge data 1966-2015	Daily flows Dainyor Bridge data 1966-2015
4	Mean annual flows at Attabad Lake	209 cumecs	220.08 cumecs
5	Mean Annual flow in Vol	6591 MCM	6940 MCM
6	Mean Monthly Flow	206.13 cumecs	218.4 cumecs
7	Design Discharge	60 cumecs	60 cumecs
8	Flow duration curve	60 cumecs at 50% of time	60 cumecs at 52% of time
9	Weir crest level (m)	2414 a.m.s.l	2414 a.m.s.l
10	Design Flood against 10,000 yrs. return period	3250 m^3/s	3320 m^3/s
11	Peak Discharge after lake routing	3085 m^3/s	3152 m^3/s
12	Max flood level	2421.3 a.m.s.l	2421.4 a.m.s.l
13	Annual suspended sediment	20.06MTons	22.5MTons
14	Total annual sediment inflow	24.06MTons	27.08MTons & in Vol 22466 Ac Ft

4.10 ANALYSIS ON ADDITIONAL AVAILABLE DATA

Flow study review for the same period (1966-2015) of observe daily flows data of Hunza River at Dainyor Bridge station as used in Feasibility study. The analysis and comparative (Hydrology & Sedimentation study) as discussed above in detail.

The additional daily flows data of Hunza river at Dainyor Bridge for period of three (03) years (2016, 2019 & 2020) along with suspended sediment data of two years (2019-2020) had been acquired from SWHP-WAPDA. The data was studied and its findings are as follow.

- Missing daily flows data for the period of 2016.
- Discontinuous years of data availability.

For the analysis of flows pattern and water availability, continuous years of data required. In the current scenario, flows study done for the period of 1966-2015. The additional data was available for the period of 2019-2020, and reflected three years missing data (2016-2018). In the light of above observations, hydrological study could not be updated.

The comparative analysis of two years daily flows data of Dainyor Bridge for period of (2019-2020) w.r.t (1966-2015) is represented in Figure 4.27 and avgerage mean monthly flows are 275 cumecs in 2019-2020 & 322 cumecs in 1966-2015, respectively.

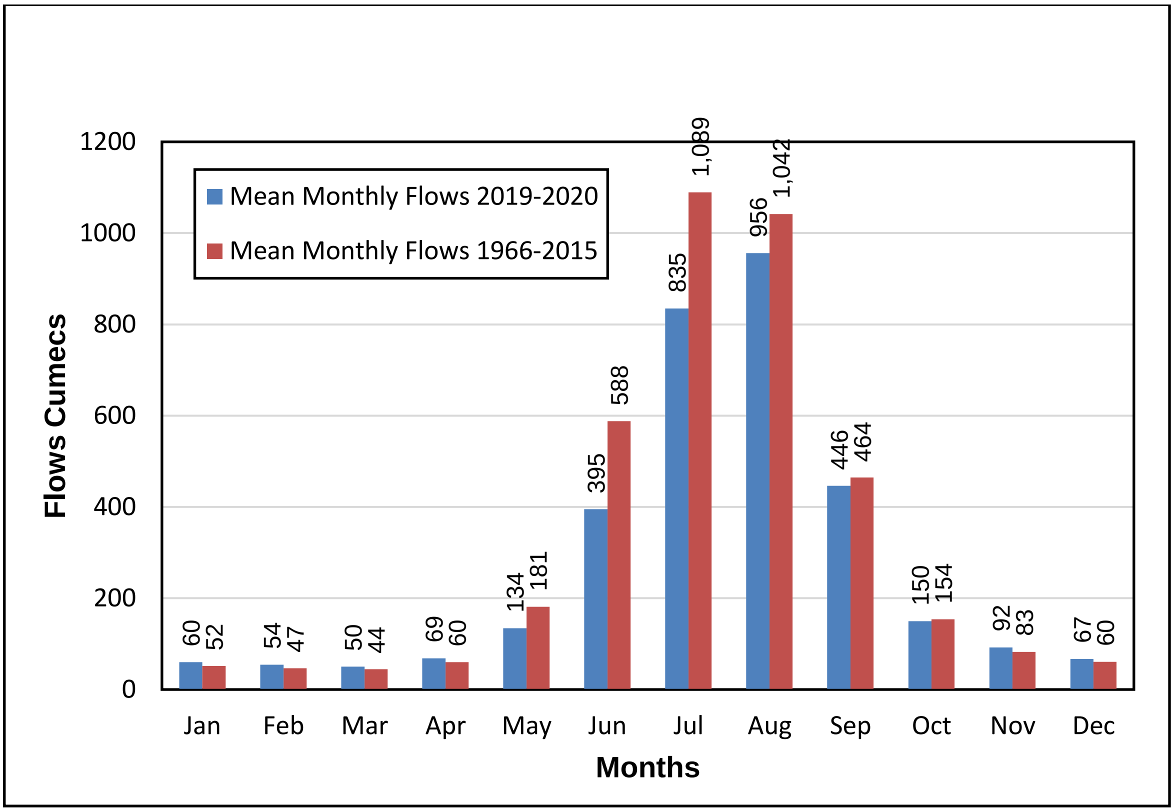


Figure 4.27: Comparison of Mean Monthly Flows - Hunza River at Dainyor Bridge

The flows generated at proposed weir location for the period of (2019-2020) by using Dainyor Bridge data with the application catchment area ratio method. The flows comparison study of Attabad lake for two (02) periods of durations (1966-2015) and (2019-2020) are represented below in Figure 4.28.

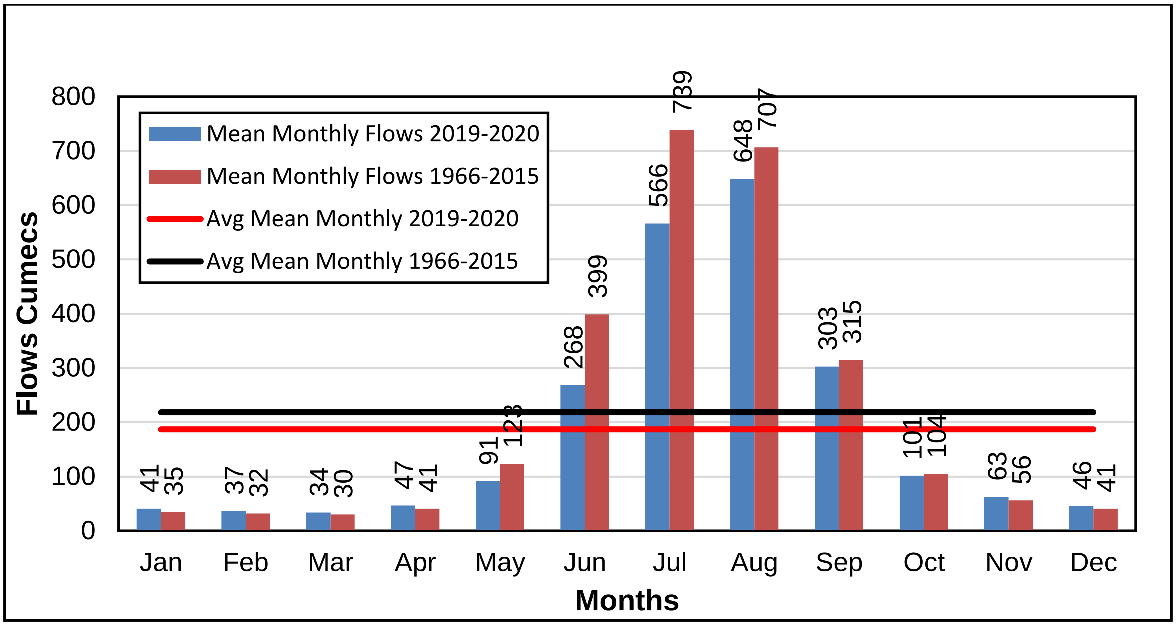


Figure 4.28: Comparison of Mean Monthly Flows - ATTABAD LAKE

The average mean monthly flows at proposed weir for period of (2019-2020) are 187.7 cumecs and for period of (1966-2015) are 218.4 cumecs, respectively. Above figure highlighted flows patter for both duration periods are similar but on other hand, flows availability reduce in period of 2019-2020.

Flows study need to be updated in detail design stage to get the effective estimated values of flows and all other hydrological parameters.

4.11 RECOMMENDATIONS

As part of the TORs, Detail Design of Attabad Lake shall be carried out by EPC Contractor. Following recommendations for EPC Contractor regarding Hydrology and Sedimentation Study are as under.

- Stream gauging station should be installed at project site for ascertaining the authenticity of water availability at project site (especially low flows during winter months).
- Flow study need to be revised/updated during detail design stage by EPC contractor. In present study, flows were estimated for the period of 1966-2015 with mean annual flows available at weir site as 6940 MCM.
- GLOFS study will be under taken during detail design stage by EPC Contractor.
- Detailed studies for design flood estimation need to be carried out by the EPC Contractor. In the Detailed Design, the Contractor shall select appropriate return periods for estimating the design floods for weir and power house site. However, the check flood not to be less than 10,000 years return period or GLOF value.
- It is suggested that GLOF value and recently flood occurrence in 2022 also be considered for estimation of flood at weir site. Design floods to be selected to ensure safety of structures and based on analysis of flood risks and consequences.
- Detailed studies for the estimation of sediment load should also be undertaken during the detail design study by EPC Contractor.

5 REVIEW OF GEOLOGY AND GEOTECHNICAL STUDIES

5.1 GENERAL

Geology and Geotechnical Studies of Attabad Lake Hydropower Project have been reviwed to ascertain the quantum and dependency of available data for further Detail design studies to be carried out by EPC Contractor. The field investigations, in-situ testing, samples collected and laboratory testing data has been reviewed to cover the available inventory of documented geotechnical investigations.

The review findings are given in forthcoming sections of this chapter along with further recommendations.

5.2 REVIEW OF GEOLOGICAL STUDIES

Attabad Lake Hydro Power Project (ALHPP) is proposed on Hunza River, a left tributary of Gilgit River. A large landslide occurred on January 4, 2010. Valley slope comprising a very steep side mountain suddenly collapsed. The slope failure was so massive and instantaneous that blocked the water flows. Vertical face of the landslide scarp as estimated by GSP (Geological Survey of Pakistan) was about 100 m high and the blocked segment of the valley was about 2 km (ISSMGE Bulletin: Volume 4, Issue 3 Page 21) that formed a natural dam and ceased the flow of Hunza river resulting in an upstream reservoir. WAPDA Authority planned to utilize the potential of hydropower due to this lake. Initially, Govt. of GB carried out Feasibility Study by engaging Local consultants in March, 2017. The study was deficient in many aspects including geotechnical investigations, laboratory testing, layout planning, impacts of right bank sliding, Neo-tectonic study and environmental issues. Therefore, Govt. of GB requested WAPDA to carryout detailed Feasibility Study and Hydro Planning Organization (HPO), WAPDA updated the Feasibility Study in June, 2021.

The geological and geotechnical investigations include Surface Geological Mapping, drilling of boreholes and Test pits at the proposed sites of weir, sediment flushing channel/under sluice, headrace tunnel, powerhouse area, and identification of borrow area for construction material. Samples collected during geotechnical investigations have been tested to ascertain geotechnical properties at Central Material Testing Laboratory (CMTL), Lahore. This chapter includes review of all geological and geotechnical investigations and studies which were carried out at Feasibility stage.

5.2.1 Review Scope of Geological Studies

The purpose of the review is to evaluate the available project information as documented in the feasibility report, (Volume I, Main report chapters) about site geology, geotechnical and foundation characteristics with respect to standard practice for a typical medium sized hydropower project located in valley largely incised by the fluvio-glacial geomorphic agents. The project site comprises the valley section located near the massive landslide which is a tectonically highly disturbed area marked by a pronounced seismic activity.

The review process activities involved the following:

- Review of Project Feasibility Report and accompanied Drawings (March, 2017).
- Review of Project Feasibility Report and accompanied Drawings (June, 2021).
- Review of geological and geotechnical investigations in terms of their adequacy for the interpreted geological model of the project.

- Review of subsurface geology particularly of the proposed weir site, power tunnel and powerhouse area.
- Registering the geological and geotechnical hazards and involved risks
- Review of suitability of recommended construction materials for various uses particularly recommendation of using the local available construction materials for the sake of improving the project economics and getting other desired benefits.
- Based on the integrity of the technical evidence, investigations and the syntheses of the analyses, the gravity and soundness of the conclusions and recommendations provided in feasibility report are adequate and/or sound enough to proceed further for the execution of the project.

The review has been completed in perspective of the following:

- Critical review of feasibility report, regional and project geological maps, relevant project documents and Google Earth satellite image.
- Critical review of the geological interpretation of the geotechnical investigations along with project layout in perspective of the terrain characterization and geohazards.
- Preparation of Feasibility Study (June, 2021) Review Report.

5.2.2 Regional Geology and Regional Faults

Regional geologic and tectonic conditions were evaluated using available geologic maps and relevant technical literature. The regional geological and tectonic Map of Pakistan published by Geological Survey of Pakistan (GSP). Geological map has been modified by Pecher et al.2008 after collecting data of different resources and the Regional Tectonic Map is compiled by Ali H.Kazmi & Riaz A. Rana under the supervision of Asrarullah Director General, Geological Survey of Pakistan in 1982.The modification of this Map has done by Chaudry and Ghazanfer in1986 and Ahsan et al, 2008. was used at Feasibility stage for the purpose of regional geological and tectonic assessment. In detail design studies, latest and elaborated regional geological maps/data/information should be used by the EPC contractor for extracting more regional geological information for the project area.

5.2.3 Site Geology

The recent deposits are the most widely spread soil units that overlies the bedrock. The slopes are covered with surficial deposits having variable thickness. Terrace deposits comprise mixtures of clay, sandy silt, boulders and gravel in varying proportions.

Soil units present in the Project Area are described below.

- **Angular Boulder Gravels with Sand and Appreciable Amount of Fines (ABGM)**
  Most of the slopes are covered by material comprising of ABGM. This is colluvial material broken from the rock units, rolled down slope and settled at the foothill or along the slopes. These deposits are mainly associated with crushed/brecciated rock units, originated/caused by brittle deformation due to tectonic compression resulting in faulting/shearing. ABGM material is present in the sediment flushing channel, surficial deposits in power tunnel alignment and in part of tailrace channel.
- **Rounded Boulder Gravels with Sand and Appreciable Amount of Fines (RBGM)**
  River bed is mainly occupied by RBGM material. Some exposures are also present at power house site Due to the formation of lake, the bed of the river is mostly covered with

silt and sand. RBGM is present in the main weir area, part of sediment flushing channel and part of power tunnel.

- **Terrace Deposits**
  Terrace Deposits consist mainly of unconsolidated mixtures of clay, silt, sand and gravels with occasional boulders and own re-worked origin being mostly derived from loose material of (RBGM, ABGM, Morainic and Brecciated deposits). Terrace deposits are present at a relatively higher elevations in the project area.

- **Morainic Deposits**
  It consists mainly of material ranging in size from large boulders to clay size particles, angular to sub-angular, poorly sorted, elongated and have flat surface, powerhouse area is covered by moraine deposits with clayey silt to silty clay.

Rock units present in the Project Area are described below.

- **Granitic Gneiss**
  It is light grey to grey in color on fresh surface and weathers to light brown, coarse to very coarse grained, moderately to highly jointed, highly sheared up to higher elevation due to geological faults, sparse to moderately weathered, moderately hard to hard. Part of power tunnel and tunnel outlet is in Granitic Gneiss.

- **Granodiorite**
  It is dark grey to blackish grey, weathers to brownish grey, medium to coarse grained, moderately weathered, moderately hard to hard, moderately jointed and is intruded by pegmatites and granitic gneiss. It is crushed, mylonised, and sheared at places. It is well exposed at the banks of Hunza River where it is intruded by many pegmatite dykes. Some shear zones are also present in this unit. Part of power tunnel and intake area are in Granodiorite.

5.2.4 Geological Structures

The whole project area bears the drastic imprints of two regional, seismically active and capable fault systems i.e., MKT and Karakorum Strike Slip Fault (KSSF). As the project area has undergone multiple episodes of tectonic events, many regional as well as local faults and their branch faults and off-shoot splays have formed. There are many faults present in the vicinity of proposed ALHPP. These faults owe their origin to the tectonic regimes associated with MKT and KSSF and transverse in between these two fault systems. Some of these faults have been observed during Surface Geological mapping and reported as following:

- **Attabad-Hispar Valley Fault**
  This fault crosses the Hunza River and forming a junction point with Hunza River and Attabad Fault in Attabad village. It is a thrust fault striking N30°E and dipping 40°SE and is well exposed at the left side slope near the project area. Granitic gneiss along its alignment is sheared / brecciated, forming fault breccia at many places. Due to high degree of shearing, many active landslides and rockfalls have been originated along its trace.
- **Attabad Fault**
  This Thrust Fault is striking N60°W and dipping 52°NE, exposed in area rear of Attabad village. It is relatively a small fault and terminates along junction point of three faults. A large brecciated and sheared rock is present along this Thrust Fault. Sheared and brecciated rock can be seen on outcrops at right bank slope.

- **Attabad River Fault**

The exposure of this fault is present at right bank slope of Hunza River. The breccia is well exposed at right side, just downstream of Sarat Suspension Bridge. The intensity of brecciation increases towards Salamanabad village. In fact, Salamanabad village is developed on brecciated rockmass.

5.2.5 Geomorphology

Geomorphologic studies were not conducted during the Feasibility stage.

The reviewer has made the geomorphologic assessment of the project area and its closed vicinity using satellite images and remote sensing techniques as shown in Figure 1. The segment of Hunza valley wherein the various components of the project are situated reflects very high relief conditions marked by relatively steeply inclined extensive mountain slopes along right side of the valley as compared to the left bank mountain slopes.

The mountain slopes are mostly marked by erratic slopes instabilities along with relatively less extensive segments of casuistic slopes. The mass wasting regimes active in the project area are drastically influenced by a broad spectra of variables such as geomechanical conditions, moisture distribution and dynamic loading. Most abundant slope hazards include but not limited to Rockfall/Debris flows, rolling slopes in unconsolidated loose overburden and mud/debris flows in sub-parallel rectilinear secondary drainage.

In perspective of the aforementioned volatile slope conditions, it is recommended that during detail design, prior to the initiation of specific and specialized investigations, a comprehensive and overall baseline terrain characterization based on geologic, geotectonic, geomorphologic, seismic geohazard and prevailing climatic conditions should have been conducted. Simplified Geomorphological Map of project area is shown in Figure 5.1.

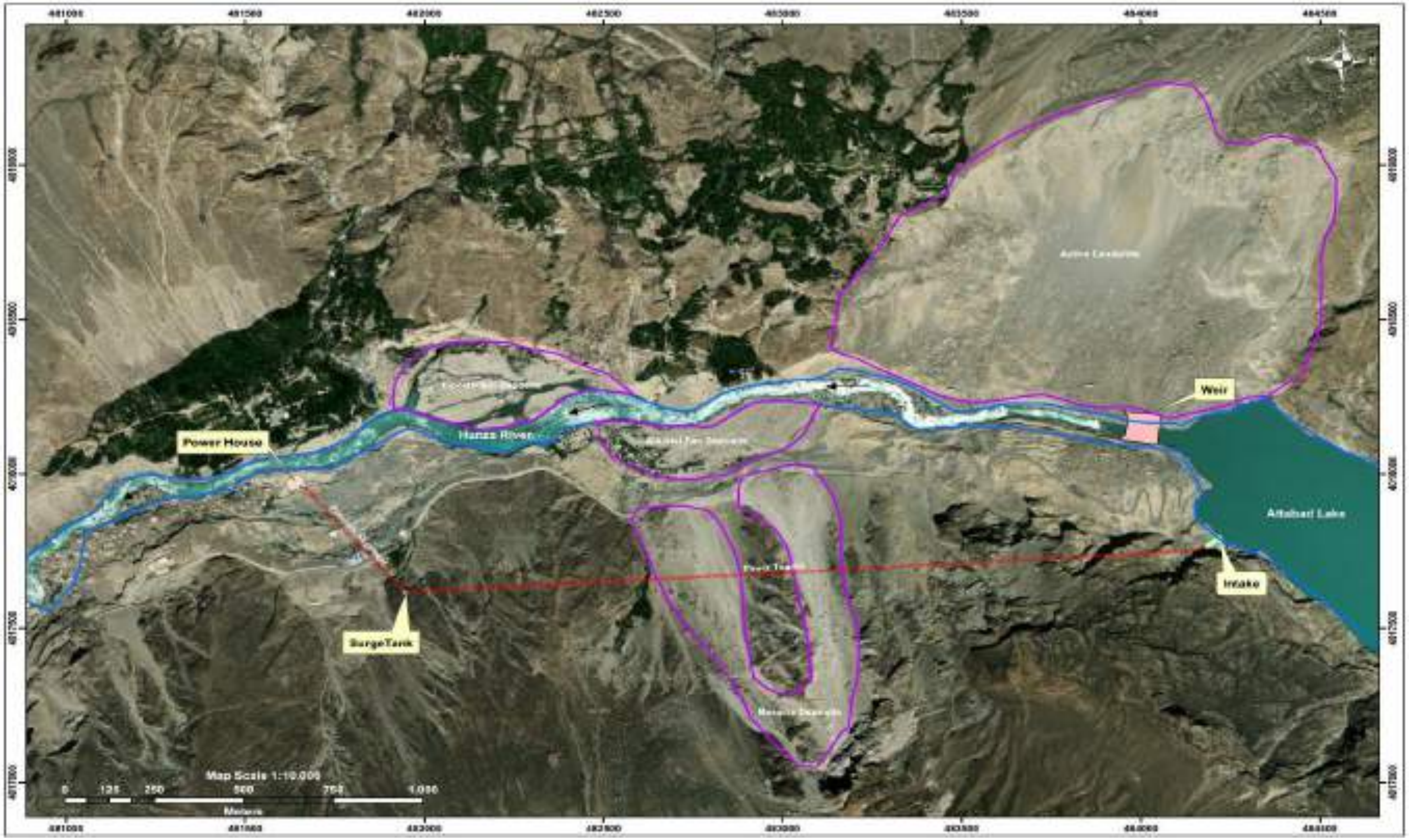


Figure 5.1: Simplified Geomorphological Map of Project Area

5.2.6 Review of Surface Geological Investigations

- **Geological Mapping**

The Surface geological mapping of the project area has been carried out on 16 topographic sheets prepared by Survey Division Peshawar, HPO, WAPDA on scale 1:1 000 with contour interval of 2 m, with the help of Total Station, Plane Table and Brunton compass and presented at scale of 1:2000.

The EPC contractor will verify the geological mapping conducted during the Feasibility Study and necessary amendments in the geological map will be made, where required. Moreover, additional geological studies will also be carried out focusing geomechanical properties and rockmass characterization.

The EPC Contractor shall perform the geologic mapping during detail design stage with an appropriate scale to capture the ground information for safely hosting the proposed structures. For engineering purpose, the mapping will define geologic conditions potentially affecting the project appurtenant structures. Emphasis, therefore, will be given on the physical properties of soil and rock masses as they may affect proposed foundations excavation, slope stability at abutment slope excavation and construction methods. The physical properties or geologic factors include the occurrence and properties of soil units, rock outcrop patterns, rock strength, weathering, permeability and condition of structural discontinuities (jointing, bedding, foliation, schistosity, faults, shear zones, etc.). The geologic materials (soils, rocks) will be described and classified following the accepted guidelines as used in other activities for soil or rock descriptions, classification and logging. Execution of the detailed geological mapping during the detail design stage will be done using Satellite Images/ Drone images.

The EPC Contractor shall perform the geologic mapping of the soil cover area/ quaternary deposits, using Unified Soil Classification System (USCS) will be conducted as required particularly focusing on (i) factors affecting water-tightness, (ii) occurrence of major faults and lineaments, and (iii) identification and characterization of significant geological hazards that might pose threat to the safety of safe operation of the project.

- **Scanline Survey**

To determine the geomechanical properties of the rockmass, scanline surveys at the critical locations plays a vital role for the assessment of the mode of failures present in the project area. However, no such studies have been conducted during the Feasibility study.

The EPC contractor should perform scanline surveys in accordance to the ISRM standards 2007and compile scanline data in tabulated form which should include joint type, dip/strike, interval, aperture, roughness, infilling and Uniaxial Compressive Strength (UCS). The collected dip/strike discontinuity data should also be plotted on software like DIPS or other structural geology software to define characteristics of major sets of discontinuity and mode of failures (planner, wedge and toppling) at the slopes as shown in a typical Kinematic Analysis in Figure 5.2.

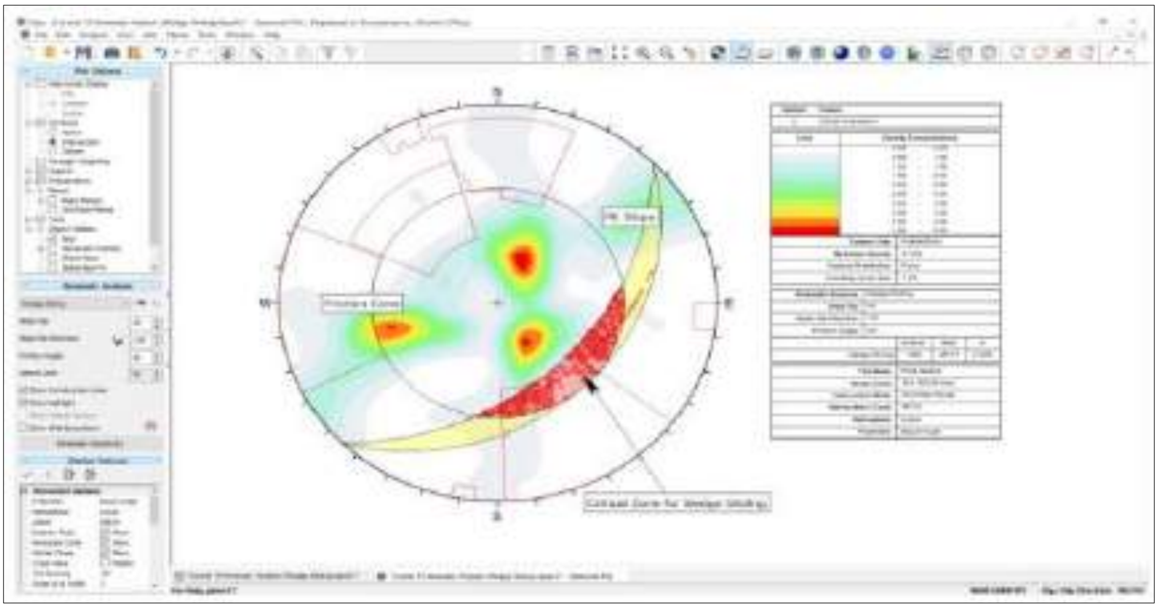


Figure 5.2: Interface of RocScience DIPS Software Showing Kinematic Analysis

- **Neotectonincs**

During the Feasibility stage, neotectonic study has been carried out in sufficient details. The area under study has been covered 40km upstream and 50km downstream of slided mass. However, during the detail design stage EPC contractor shall verify and confirm the local faults, shears, lineaments, scars and tectonic distortion trends specific to the project area and its 50 km radial vicinity.

5.3 REVIEW OF GEOTECHNICAL STUDIES AND SUBSURFACE INVESTIGATIONS

Field investigations for Feasibility study 2021 included drilling of six (06) boreholes, excavation of ten (10) test pits, field testing in boreholes and sampling for laboratory testing. Comprehensive field investigations plan of Feasibility study 2021 marked on satellite image is shown in Figure 5.3.

Figure 5.3: Field Investigation Plan of Feasibility Study 2021

Summary of drilled boreholes with locations, drilled depths and conducted water pressure tests is given in Table 5.1. Total drilling of 401 m have been executed for foundation evaluation of different structures, bore holes of varying depths have been drilled accordingly to estimated geological conditions prevailing in subsurface. Permeability tests have been performed using constant head method in overburden material at 3 m interval whereas water pressure tests (Lugeon tests) in bed rock have been performed at interval of 5 m or at change of strata. Rock cores and soil samples have been collected from all the bore holes at various depths for lab testing.

Summary of excavated test pits with depths is given in Table 5.2. A total ten (10) number of test pits having cumulative depth of 30m were excavated to study the surficial material present at different locations.

The review of subsurface investigation data revealed following:

- Low core recovery and RQD values recovered at shallow depths.
- Core recovery and RQD values increases with depth.
- Lack of discontinuity data recording in the core logs as only one (01) borehole i.e., BH-04 was drilled 74m into the bedrock.
- Lack of bedrock information.
- Lack of details of preserved rock samples in the borehole logs.
- Non availability of core box photographs.
- Details of permeability test have not been provided.
- Description of material encountered is lacking and confusing, like information about sand/fines, roundness or angularity of gravels etc.
- Sufficient Investigations at many important structures like pressure tunnel intake, surge tank, penstock, and right abutment of main weir were not done.

Table 5.1 Boreholes and Summary of Analysis

Area	BH No.	Depth (m)	Inclination	Material Encountered	Permeability Range (cm/sec)	Water Pressure Test
Weir L/S	BH-01	90	90°	Angular Boulder Gravels of slided mass, light grey to grey in color, mostly of Granodiorite and Granitic gneiss.	0-4m depth, K=0.04618, 13-16m depth, K=0.002858, 16-19m Depth, K=0.012294	
Sediment Flushing Channel (Inlet)	BH-02	60	90°	Angular Boulder Gravels of slided mass, light grey to grey in color, mostly of Granodiorite and Granitic gneiss	0-3m depth, K=0.06037, 3-6m depth, K=0.01327	
Sediment Flushing Channel (outlet)	BH-03	61	90°	Light grey to grey, fine to medium grained sand with angular gravels variegated origin	25-28m depth, K=0.001226, 28-32m depth, K=0.007040, 32-37m depth, K=0.003243, 37-42m depth, K=0.002813, 42-46m depth, K=0.001832, 46-49m depth, K=0.002621, 49-52m depth, K=0.001082, 52-56m depth, K=0.001436, 56-59m depth, K=0.000724	
Power tunnel Alignment	BH-04	80	90°	Cobbles Granodiorite & Granitic Gneiss, Mylonised Granitic Gneiss, Granitic Gneiss, foliated & fresh, moderately jointed, moderately strong to strong		Maximum Value=3.77 at 5Bar Minimum Value=0.01 at 3Bar
Powerhouse	BH-05	50	90°	Angular to Sub- Angular pebble, cobble and boulders of Granitic	12-15m depth, K=0.0277,	

Area	BH No.	Depth (m)	Inclination	Material Encountered	Permeability Range (cm/sec)	Water Pressure Test
				Gneiss, Grey to brownish grey, Fine Clayey Silt, Fine to Very Fine Silty Sand	15-19.5 depth, K=0.00945, 19.5-22.5m depth, K=0.00685, 22.5-25.5m depth, K=0.006359, 25.5-28.5m depth, K=0.004969	
Powerhouse	BH-06	60	90°	Grey to Brownish grey, fine to very fine grained, clayey silt, Cobbles and Boulders comprising of granitic gneiss, granodiorite and silty sand	10.4-16m depth, K=0.00241, 16-20m depth, K=0.00289, 20-23m depth, K=0.00392, 24-26m depth, K=0.00286, 29-32m depth, K=0.00338, 35-53m depth, K=0.00279, K=0.00781, 53-57m depth, K=0.00298	

Table 5.2 Test Pits and Summary of Analysis

Area	TP No.	Depth (m)	Inclination	Water Table Depth (m)	Material Encountered
Shishkat Bridge (Sand Aggregate)	TP-01	3	90°	1.3	Sand with Subordinate gravels
Powerhouse Area (Aggregate)	TP-02	3	90°	Nil	Boulder, Cobbles and pebbles of variegated origin, embedded in sand, Clayey Silt, grey to brownish grey
Power House Area (Fine Aggregate/Clay)	TP-03	3	90°	Nil	Clayey Silt, Light Grey to brownish grey in color, fine to very fine grained
Power House Area (Fine Aggregate)	TP-04	3	90°	Nil	Light grey to grey, medium to fine grained sand, grey to brownish grey, fine to very fine grained clayey sandy silt
Outlet of Sediment Flushing channel alignment	TP-05	3	90°	Nil	Clayey silty sand with gravels
Flushing Channel Slope Alignment (Left Side)	TP-06	3	90°	Nil	Gravelly Sand and ABGM, it consists of boulder, gravels of mostly granodiorite with subordinate clay and silt
Inlet of Sediment Flushing Channel Alignment (left Side)	TP-07	3	90°	Nil	Silty Sand with RBGM, grey colored silty sand with subordinate gravels and ABGM
Slope Area Above Spillway (Right Side)	TP-08	3	90°	Nil	Gravelly Sand and ABGM, it consists of boulder, gravels of mostly granodiorite with subordinate clay

Area	TP No.	Depth (m)	Inclination	Water Table Depth (m)	Material Encountered
					and silt, some crushed rock ground to silty/ sandy material
Slope Area Above Spillway (Right Side)	TP-09	3	90°	Nil	Gravelly Sand and ABGM, it consists of boulder, gravels of mostly granodiorite with subordinate clay and silt, some crushed rock ground to silty/ sandy material
Slope Area Above Spillway (Right Side)	TP-10	3	90°	Nil	Clayey Silt, Light brown to brownish grey, fine to very fine grained

5.3.1 Review of Field Testing Data

- **Permeability and Water Pressure Testing**

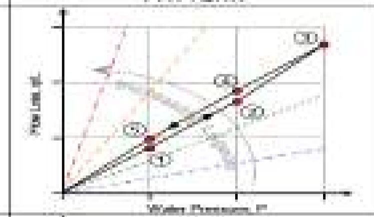
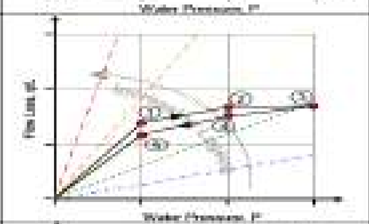
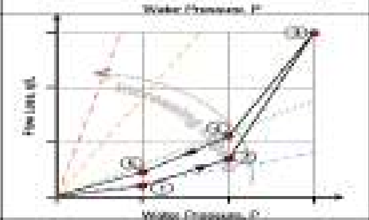
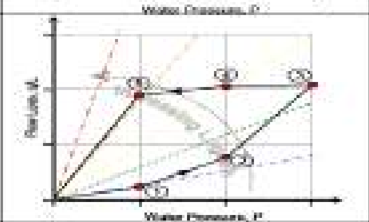
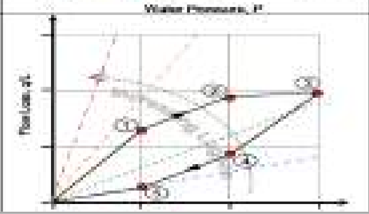
Summary of permeability and water pressure tests is given in table 5.1. Permeability and water pressure tests performed in feasibility study are limited and insufficient to use in the detail design.

During the detail design stage, the EPC Contractor has to drill appropriate number of additional boreholes at the project appurtenant structures and have to perform Lugeon at regular interval to assess Lugeon values under the weir, powerhouse and other structures of the project. These additional geotechnical investigations will be supervised by an experienced Engineering Geologist.

- **Determination of Lugeon Pattern**

The EPC contractor has to plot the values of 'Flow' verses 'Pressure' to assess the Lugeon pattern that will help in evaluating the discontinuities pattern in the test section as shown in Table 5.3 represents the proposed Lugeon interpretation procedure using the flow loss vs. pressure.

Table 5.3 Proposed Lugeon interpretation procedure using the flow loss vs. pressure

BEHAVIOR	WATER LOSS VS PRESSURE PATTERN	DESCRIPTION	REPRESENTATIVE LUGEON VALUE
LAMINAR		All Lugeon values about equal regardless of the water pressure	Average of Lugeon values for all stages
TURBULENT		Lugeon values decrease as the water pressures increase. The minimum Lugeon value is observed at the stage with the maximum water pressure	Range of Lugeon values observed at water pressures expected during operation. If water pressure expected during operation is unknown use the value corresponding to the medium water pressure (2nd or 4th stage)
DILATION		Lugeon values vary proportionally to the water pressures. The maximum Lugeon value is observed at the stage with the maximum water pressure	Range of Lugeon values observed at water pressures expected during operation. If water pressure expected during operation is unknown use the value corresponding to either low or medium water pressures (1st, 2nd, 4th, or 5th stage)
WASH-OUT		Lugeon values increase as the test proceeds. Discontinuities' infillings are progressively washed-out by the water	Highest Lugeon value recorded (5th stage)
VOID FILLING		Lugeon values decrease as the test proceeds. Either non-persistent discontinuities are progressively being filled or swelling is taking place	Use final Lugeon value (5th stage), provided that presence of non-persistent discontinuities and/or occurrence of swelling is confirmed by observation of rock core.

- **Processing of Lugeon Test Result**

Graphs of depth and Lugeon values for all the existing and new Lugeon tests have to be prepared as shown in Figure 4 for clarification of the relation between depth and permeability, as well as detecting abnormally high permeable zones in deep portions. This will help in evaluating the requirements of any grouting measures required in the foundations of the appurtenant structures.

The occurrence of unconsolidated and loose quaternary deposits such as ABGM, RBGM, Terrace and Morainic deposits at various project structures indicates that such materials may need to be treated for stability of the works and slopes.

5.3.2 Review of Laboratory Testing Data

A total of 10 rock core samples from BH-04 (Power tunnel alignment) have been selected for evaluation of geotechnical parameters for design purpose. A total of 15 samples, i.e., 3 block sample, 3 samples from borrow area, and 9 test pit samples have been selected for investigations. Samples from all Test Pits have been selected by cone and quarter method and different type of tests have been performed to check their engineering properties and confirm their suitability as aggregate material.

- **Petrographic Analysis**

Petrographic Analysis was performed on a total of 05 samples to confirm the mineral assemblage in the particular rock unit. For this purpose, 2 samples were collected through excavation of test pits, 2 samples from different locations were collected during surface geological mapping, and 01 drilling core sample was collected from power tunnel alignment.

Table 5.4 Results of Petrographic Analysis

Sr. No.	Borehole No.	Rock Name	Major Minerals					Minor Minerals					Remarks / Description
			Quartz	K-Feldspar	Oligoclase	Biotite	Amphibole	Magnetite	Chlorite	Epidote	Muscovite	Carbonate	
1	Rock Samples	Foliated Granite	24	18	17	21	1	2	2	6	7	1	ASR Potential exists
2		Granite	32	26	14	6	3	2	4	1	3	3	ASR Potential exists
3		Granodiorite	24	11	20	15	16	2	3	5	2	2	ASR Potential exists
4		Granodiorite	26	12	25	14	11	2	3	3	2	2	ASR Potential exists
5	BH-04	Granitic Gneiss	30	13	28	17	4	1	3	1	2	1	ASR Potential exists

However, as revealed from test results reproduced herein Table 5.4, it is suggested that more petrographic analyses are required on samples collected from various locations of additional investigations. For the assessment of ASR, the limited number of tests are not enough to describe the rock conditions existing in the project area.

- **Direct shear test**

Direct Shear Test is performed on Granitic Gneiss Sample. One sample from BH-04 was selected for Direct Shear Test from depth of 56 to 56.35m. The result shows that cohesion “c” has 0.595 MPa with angle of shear resistance “ϕ” is 21.8 deg. Only one test is insufficient for detail design of a hydropower project. Therefore, it is recommended that more direct shear tests should be performed by EPC Contractor during the detail Design stage at various project structures.

- **Rock Core Modulus**

Rock Core Modulus was determined for 2 core samples and 3 bulk samples. The core samples were of Granitic gneiss rock. The results are reproduced in Table 5.5.

Table 5.5 Results of Rock Core Modulus Determination

Sr. No.	Bore Hole	Location	Depth (m)		Test Results (MPa)		
			From	To	Young's Modulus	Modulus of Deformation	Mode of Failure
1	BH-04	Power Tunnel Alignment	48.0	48.25	$2.29*10^4$	$1.15*10^4$	Axial
2			61.0	61.30	$1.83*10^4$	$9.16*10^3$	Axial
3	Bulk Samples	Passu Area	-	-	$7.52*10^4$	$3.76*10^4$	Diagonal & Axial
4		Powerhouse Area	-	-	$4.16*10^4$	$2.08*10^4$	Diagonal & Axial
5		Slided Mass	-	-	$4.96*10^4$	$1.88*10^4$	Diagonal

It is recommended that more tests are required during detail design stage. EPC Contractor should conduct more tests on samples collected from additional investigations.

- **Point Load Strength Index Test**

Point Load Strength Index test is performed on 3 core samples, one sample of Mylonised Granitic Gneiss and 2 samples of Granitic Gneiss, and 4 bulk samples. The results are reproduced in Table 5.6.

Table 5.6 Results of Point Load Strength Index Test

Sr. No.	Bore Hole No.	Location	Depth (m)		Test Results		
			From	To	Is (Mpa)	Is 50 (Mpa)	Failure
1	BH-04	Power Tunnel Alignment	21.00	21.15	4.04	3.93	Diametrically
2			31.21	31.52	5.39	5.23	Diametrically
3			43.25	43.55	3.14	3.05	Diametrically
4	Bulk Samples	TP-08	-	-	1.81	1.66	Diametrically
5		Passu Area	-	-	4.11	4.30	Diametrically
6		Powerhouse Area	-	-	3.97	4.15	Diametrically
7		Slided Mass	-	-	5.68	5.94	Diametrically

As required, any additional tests may be conducted on the core samples.

- **Uniaxial Compressive Strength Test**

Uniaxial Compressive Strength test is performed on 3 rock core samples of Granitic Gneiss, 3 samples from Test Pits and 3 Bulk samples from borrow areas. The results are reproduced in Table 5.7.

Table 5.7 Results of Uniaxial Compressive Strength Test

Sr. No.	Bore Hole / Test Pits	Location	Depth (m)		Test Results	
			From	To	UCS (MPa)	Failure Mode
1	BH-04	Power Tunnel Alignment	38.14	38.43	49.2	Diagonal
2			56.00	56.39	45.2	Diagonal
3			68.00	68.23	38.5	Diagonal
4	TP-7	Slided Mass (Left side)	-	-	52.40	Diagonal
5	TP-8	Slided Mass (Right side)	-	-	71.91	Diagonal
6	TP-9	Slided Mass (Right side)	-	-	34.63	Diagonal
7	-	Passu Area	-	-	47	Diagonal
8	-	P/H Area	-	-	88	Diagonal & Axial
9	-	Slided Mass	-	-	59	Diagonal & Axial

Core samples are from only one borehole and all samples are of same lithology. More sampling and testing is required for the better assessment of results and prediction of actual rockmass conditions prevailing in the project area during the detail design stage.

- **Indirect Tensile Strength Test**

Indirect Tensile Strength test is performed on Granitic Gneiss rock. (ITS) Test has been carried out on core samples collected from BH-04, at depth from 43.25 to 43.55m. The (ITS) Test is mostly estimated as 10 to 15% of Uniaxial Compressive Strength (UCS). The labortory test value is 6.46 MPa which indicates that rock is moderately hard.

- **Hoek Triaxial Compressive Strength Test**

Hoek Triaxial Compressive Strength test is performed on 2 rock core samples of Granitic Gneiss. The values of cohesion “C” and angle of internal friction “Ø” calculated by (HTCS) Test, indicates that the shear strength of the rock core samples is low to moderate. The results are reproduced in Table 5.8.

Table 5.8 Results of Hoek Triaxial Compressive Strength Test

Sr. No.	Borehole No.	Depth		Cohesion "c" MPa	Angle (Ø) deg.
		From	To		
1	BH-04	52.60	53.00	2.18	27.749
2		63.00	63.35	0.60	15.543

The review of laboratory testing data revealed following.

- The number of test performed on rock samples are not adequate
- Rock samples collected from only 01 borehole i.e., BH-04 is not truly representing the sub surface engineering properties of rocks for whole of the project area
- Test results obtained from rock and soil samples are limited and presenting unexpected variations.
- In case of direct shear test only one sample is not enough to describe the Cohesion and Shear resistance values of a rock type furthermore this test should be performed on all types of rock units existing is the project area.
- The number of test performed are not enough to describe Young Modulus and Modulus of Deformation. The testing should be performed on samples of all rock types.
- For UCS test core samples are from only one borehole and all samples are of same rock type i.e., Granitic Gneiss. Adequate number of tests/ samples and testing is required for the better assessment of results and prediction of actual rockmass conditions prevailing in project area.
- Direct shear test and Uniaxial compressive strength test is performed on the sample taken from the same depth of same borehole (56-56.35m)
- Property Index tests (NMC, Specific Gravity, Porosity, Dry Density, Water absorption) are performed on the samples on which rockmass testing is already performed.

Considering the previously available data, and the project requirement, the subsurface deep investigation and refining/ conforming the information plenty of standardized laboratory testing of site specific rock samples is essentially required by the EPC Contractor for the assessment of rockmass parameters for detailed design.

5.4 ADDITIONALLY PROPOSED FIELD AND LABORATORY INVESTIGATIONS

Apart from this limitation of previous stage, for detail design by the EPC Contractor shall require drill holes at the sites of missing structures as well as to drill additional holes at already investigated sites for refining/ conforming the information for use in EPC design. Considering the previously available data, and the project nature, the deep subsurface investigations vertical and inclined both are essentially required by the EPC Contractor for the detailed design. It is recommended to perform the additional geotechnical investigations following the terrain characterization of the project area and prior to the finalization of the project layout plan.

In order to complete the set of data obtained from the ground surveys drilling of some additional boreholes to evaluate the geotechnical and geological conditions at the project components are recommended for Detail design studies by EPC Contractor. These boreholes should be located on the right bank of the weir where no previous borehole present, water intake, surge tank and penstock alignment, in the present powerhouse location (can be used for tailrace structure) and new proposed location of powerhouse.

The locations of the proposed boreholes is given in Table 5.9 and shown in Figure 5.4 and Figure 5.5. This proposal is subject to finalization of project layout during detail design studies.

Table 5.9 Coordinates and Locations of Proposed Boreholes for Detail Design

Sr. No	Borehole	Northing	Easting	Depth (m)	Location
1	BH-7	4018180	484021	60	Weir Site (Right abutment)
2	BH-8	4017758	484168	90	Power Intake
3	BH-9	4017615	481944	70	Surge Tank
4	BH-10	4017720	481859	80	Penstock
5	BH-11	4017836	481762	60	Proposed Power House Area
6	BH-12	4017961	481648	60	Power House Area

It is proposed to perform laboratory tests in order to define the grain size, permeability and grain size of the slopewash material during the opening of the borehole on the right bank of the weir location.

Lugeon tets should be done during the drilling of boreholes in the intake and surge tank. Also UCS with elastic moduli, unit weight, porosity, point load, UCS and triaxial tests should be perfomed on samples taken from these boreholes.

Menard pressuremeter test is proposed in the location of the penstock and powerhouse.

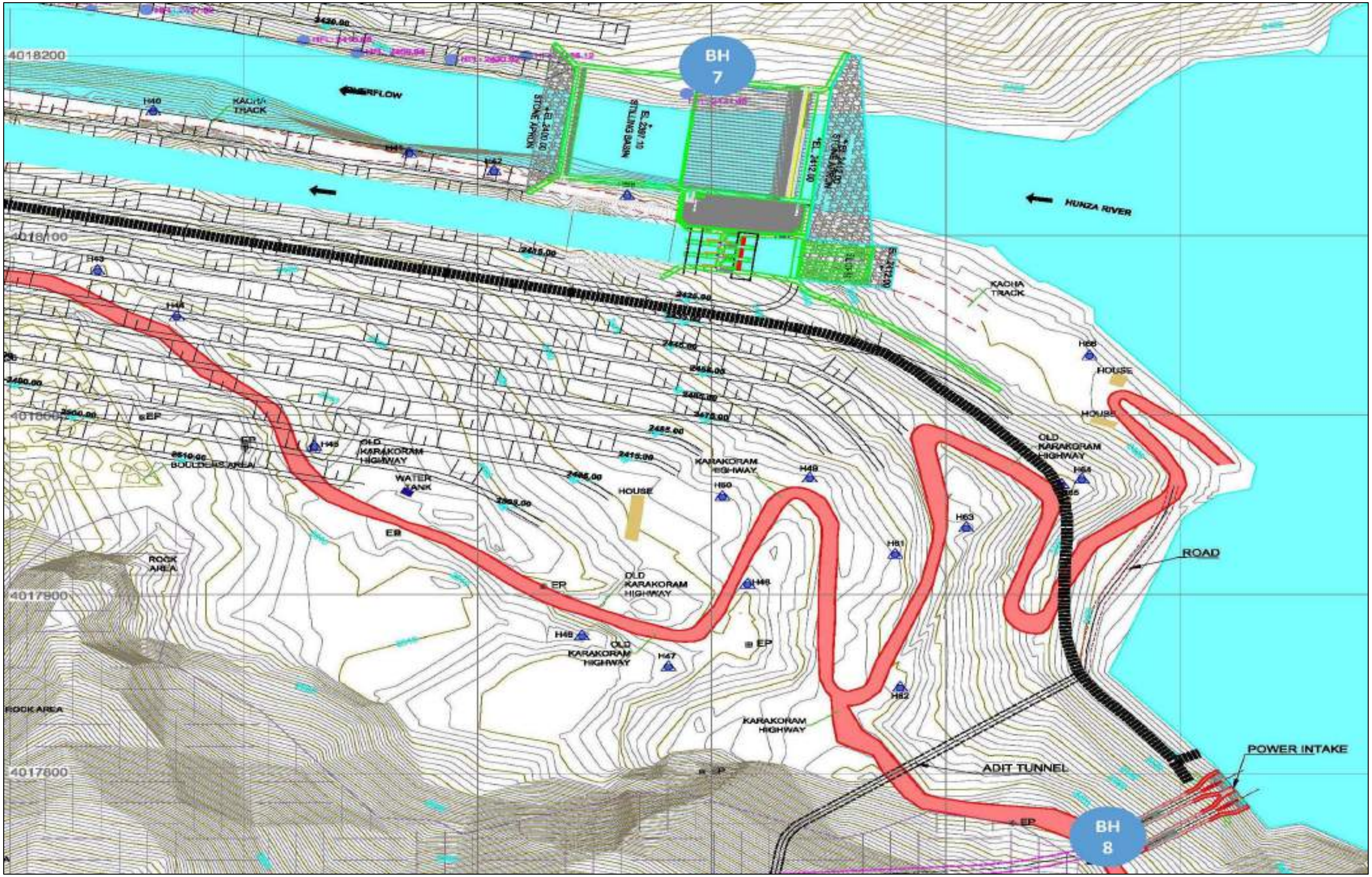


Figure 5.4: Proposed Borehole Locations of the Weir and Intake Area

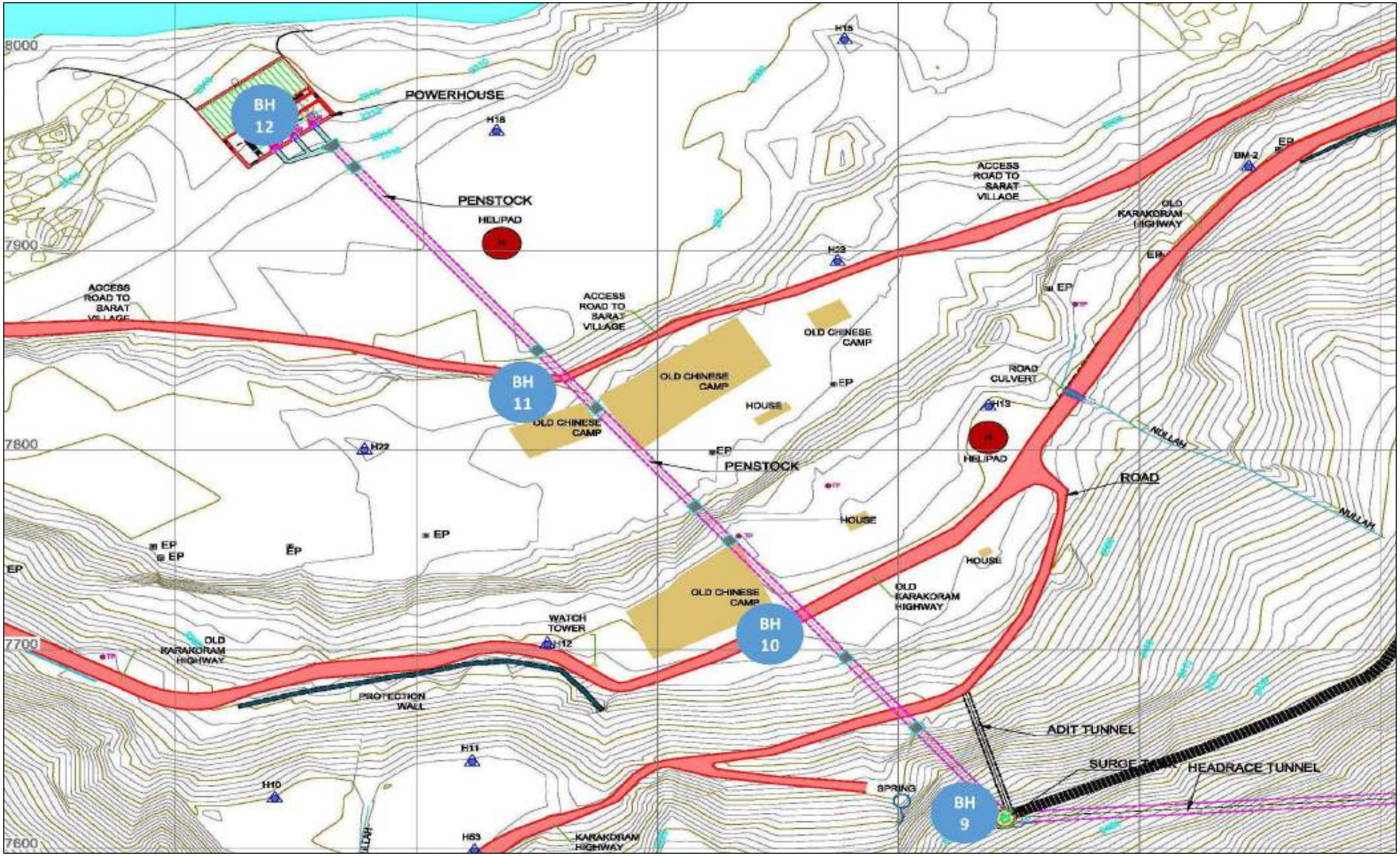


Figure 5.5: Borehole Locations between Surge Tank and Powerhouse

5.5 REVIEW OF FOUNDATION EVALUATION

The available documentation reviewed in this regard includes in-situ foundation evaluation as reported under Sub-Section 5.11and further foundation treatments and specific considerations under Sub-Section 10.11, both from Volume-I: Main Report of the Feasibility Study 2021. The FEM models and concept have also been presented in figures as included in Volume-VII: Appendices to the Feasibility Study. Additionally, the project drawings from Volume-II studied for the review include the surface geological cross section of weir and under sluice area as shown in drawing AFS-GE-313 along with the arrangement, plan, cross-section and longitudinal section of these structures as presented in Feasibility study 2021 drawings AFS-GE-408 to AFS-GE-411.

5.5.1 Review Findings

The following aspects have been noted from the available documentation:

- **Weir and Under sluice Area**

- BH-1, BH-2 and BH-3 drilled in the slided mass at the location of these structures show that the underlying strata consists of loose, highly pervious ABGM (slided mass), having large size boulders and cavities with no rock till the depth of 90m.
- The foundation of weir is proposed to be laid on this slided mass after foundation treatment.
- It is recommended in Feasibility study to replace a maximum of 14m deep slided mass material with stabilized soil layer as weir foundation treatment.
- Silty clay hauled from the powerhouse area mixed with 2% cement will be placed as foundation material under 60m wide weir.
- Moreover, 2 cutoff walls were also proposed beneath the main structure of the weir and under sluices.
- Permeability of weir area is very high as BH-01 drilling is showing 100% water loss and water head was not maintained/built during the permeability test.
- Geological rock units with which weir has to be abut, includes granodiorite, intruded by pegmatite veins at right side. On left side, weir has to be abut with ABGM formed by slided mass (DWG No: AFS-GE-297 & 298). This statement given in section 5.12 of Attabad Lake HPP Main Feasibility Report is contradictory to the geological maps and geological cross sections.

- **Intake**

- Surface geological section of intake is presented in drawing AFS-GE-314. It is proposed to be excavated in the granodiorite rock which is strong to moderately strong. Granodiorite rock is exposed at steep angle where it is hard, massive, strong to moderately strong, and sparsely to moderately jointed. But at higher elevation, the rock is highly jointed and sheared, having vertical open joints which must be treated for stability and safety.(DWG No: AFS-GE-295 & 298). Intake area slopes alongwith Tunnel Portal slopes stability analysis was not conducted during the Feasibility study.

- **Power tunnel**

- Geological cross-section of power tunnel is missing from the Volume: II Drawings.
- It is proposed to be excavated through grandiorite with subordinate Granitic gneiss.
- BH-4 shows that the rock has high RQD values with low permeability which seems good for tunnel excavation.

- Data obtained during the feasibility study for tunnel design is not upto the mark for the safe design of the Power Tunnel.

- **Surge Tank**

- It is proposed in moderately hard, highly jointed and sheared granitic gneiss rock.
- Borehole location map AFS-GE-316 shows that no borehole has been drilled at the proposed location.

- **Penstock**

- The Penstock will be supported by piers according to Section 5 “Geology and Geotechnical Studies” whereas according to Section 15 “Construction Planning” the penstock will be laid over concrete in the excavated trench. Moreover, in the “Penstock Plan and Longitudinal Profile” DWG NO. AFS-ST-430, piers and its foundations cannot be observed instead anchor blocks can be observed at the joints.
- The foundation of piers is proposed to be laid on overburden material (sandy gravel and boulder) after removal of clayey silt.

- **Powerhouse**

- The powerhouse layout, plan, cross-section and longitudinal section are presented in Dwg. No. AFS-PH-(431, 432, 435 and 436) respectively.
- BH-05 and BH-06 had been drilled in the proposed power house area. According to the field log BH-05 detects boulders and gravel material at a depth of 30 m whereas BH-06 detects similar material at a depth of 15 m.
- It is proposed to remove the whole thickness of clayey silt layer due to high swelling potential and then lay the foundation of powerhouse on gravels and boulders.

- **Cofferdam**

- The layout and cross-section of coffer dam are provided in drawing AFS-WE-412 and AFS-WE-413.
- The staged construction sequence of cofferdam will comprise the construction of upstream coffer dam along with under sluice channel in stage-I, afterwards, Stage-II cofferdam will be constructed on the upstream of weir site till the completion of weir construction. Diversion flood is selected as 1750 cumecs corresponding to 20-year return period.
- Coffer dam is proposed to be made of silty sand, sandy silt and gravelly sand available at river bed and slided mass. The large boulders present in the slided mass are proposed as riprap material. The foundation of coffer dam will consist of impervious cutoff off provided by intersecting columns of jet grouting.

5.5.2 Technical Considerations for Foundations

The following aspects necessitate specific technical considerations regarding foundations of the project structures as described above:

- **Weir and Under sluice Area**

- The longitudinal sections shown in drawing AFS-GE-410 are not clearly consistent with the cross-sections shown in drawing AFS-GE-411, in depicting the proposed treatment of stabilized clay blanket and the depth of cutoff walls. The concept proposed and indicated through the Figures 2.11 and 2.17 along with the respective FEM Models for analysis as shown in Figures 2.12 and 2.18 are also in contrast with the drawings.
- The specific properties of silty clay material from the powerhouse area in-situ and treated have not been lab tested for comparative assessment of utility. Tests like dispersion,

swelling, shrinkage shear test parameters etc. must be evaluated prior to a recommendation of such a proposal.

- The strata at the weir location seems very critical due to presence of loose ABGM, therefore any excavation through large boulders seems impractical and likely to slide at or during the time of execution.
- Secondly, the clayey blanket is proposed to be placed on the highly pervious and loose slided mass. As clay is always placed in the form of thin compacted layers. The placing and compaction of clay on the slided mass material also seems hazardous and impractical as finer material particles of the clay have the potential to migrate or erode in absence of adequate filters over its foundation. The presence of large cavities further complicates this issue.
- Structural stability of the weir has not been addressed to evaluate its safety in bearing, sliding and overturning scenarios.
- The type of cutoff wall is not finalized rather both options of cement slurry and plastic piles have been indicated. The equipment requirements, execution methodologies and material specifications for each of these options are different and necessitate specific deliberations in context.
- The slide slopes are proposed to be stabilized through benching which seems impractical in such loose material having boulders as large as 25m. The estimated volume and extents of the excavation has the potential to increase considerably out-of-proportion. Due to the critical nature of strata, the excavation is hazardous and attempts to excavate may possibly trigger further land sliding. Moreover, the care and handling of excavation equipment in such area is not feasible.

- **Intake**

- Since, no borehole is drilled at this site so it needs further subsurface investigations.
- It is also reported that the rocks at higher elevation are highly jointed and sheared having vertical open joints which needs to be analyzed and treated for safety and stability.

- **Power Tunnel**

- It is mentioned in Volume-I, the rocks at end of the tunnel are highly sheared and the rock units are crushed which needs to be analyzed and evaluated for the stability of the proposed power tunnel.

- **Surge Tank**

- Surge Tank Plan and Section DWG No. AFS-ST-428 Section A-A shows that Surge Tank is almost 61 meters embedded below the NSL. Due to absence of investigative boreholes the subsurface strata are unknown at this location. Subsurface investigations shall be conducted before evaluating the stability of Surge Tank structure.

- **Penstock**

- Section 5 “Geology and Geotechnical Studies” contradicts with DWG NO. AFS-ST-430 “Penstock Plan and Longitudinal Profile” as piers are proposed to support the Penstock alignment in prior whereas anchor blocks can be observed in later.
- Subsurface investigations along he proposed Penstock length are important for analysis.

- **Powerhouse**

- The underlying strata of powerhouse consists of clayey silt material followed by sandy gravels and boulders. The layers of deposited material may vary as the field logs show different depth of clayey silt material.

- Cross Section for Powerhouse DWG. AFS-PH-435 shows the depth of the base of the Powerhouse 16 m below the natural surface level. Removal of the whole thickness of clayey silt layer thereby also implies depths greater than the powerhouse base.
- No further details related to powerhouse foundation have been specified in the feasibility report. As such the complete removal of the cover layer (clayey silt) and the proposed foundations for the powerhouse require re-consideration in terms of further clarity.

- **Cofferdam**

- It is reported in the feasibility study that the coffer dam will be constructed for 2-year period but the stability and protection of cofferdam in such slide prone area is not considered in feasibility study. Further, the provision of grouting columns in the slided mass material seems impractical due to presence of large cavities. Also, the quantification of material available at the indicated sites for the construction of dam body is not specified.

5.5.3 Recommendations for Foundation Evaluations

- **Weir**

- The EPC Contractor shall perform the detailed engineering geological mapping during detail design stage with a scale of 1:500 to capture the ground information for safely hosting the weir. Moreover, additional geological and geohazard studies shall also be carried out focusing geomechanical properties, terrain characterization and rockmass categorization.
- Additional design stage geotechnical investigations should be performed by EPC Contractor on both left and right abutments and also into the middle part of the Weir foundation/gabion mattress.
- Permeability tests in soil and rock, collection of samples from overburden strata and rock coring for appropriate laboratory testing, must be done for these boreholes.
- A comprehensive cut slope stability analysis should be performed by the EPC Contractor for detailed design of cut slopes in both soil and rock conditions. Suitable slope stability measures shall be then adopted according to the results of analyses including shotcrete, wiremesh, and rock bolts etc.
- Permeability parameters for seepage analysis must be evaluated for near to actual depiction of site conditions in numerical model so that seepage control measures can be adopted. This shall be done by means of appropriate geotechnical investigations and permeability testing in river bed by EPC Contractor.

- **Intake**

- Accurate location of the Intake shall be finalized after terrain characterization and geohazard studies
- Scanline survey and geomechanical properties of the rockmass shall be evaluated by the EPC Contractor during the detailed design stage.
- For stability assessment and detailed design of cut slopes, appropriate geotechnical investigations shall be performed by EPC Contractor at the location of Intake.
- Specialized laboratory testing for evaluating relevant shear strength constitutive model for stability analysis and Uniaxial Compressive Strength (UCS) tests
- Detailed design of bedrock slopes, cut slopes and tunnel portals/ intake area shall be prepared by the EPC Contractor. Selection of type/ layout of protection/ stabilization Measures shall also be prepared by the EPC Contractor.

- **Power Tunnel**

- The EPC contractor should perform the geologic mapping during detail design stage with a scale of 1:500 along the power tunnel alignment corridor to capture the ground information. Moreover, additional geological and geohazard studies shall also be carried out focusing geomechanical properties, terrain characterization and rockmass categorization.
- Accurate alignment of the tunnel should be finalized after terrain characterization, geological and geohazard studies
- Scanline survey and geomechanical properties of the rockmass shall be evaluated by the EPC Contractor during the tunnel alignment mapping in detailed design stage
- Appropriate geotechnical investigations with vertical and inclined boreholes shall be conducted during the tunnel design.
- Laboratory testing of selected samples should be conducted for evaluating the relevant geotechnical parameters.
- Detailed discontinuity survey/Scanline survey is required across the tunnel alignment for the assessment of rock quality.
- Assessment of rock quality is required by the empirical methods like Rock Mass Rating and Q-Method and evaluate the support across the power tunnel alignment.
- Support of power tunnel alignment is required to check through numerical modeling in computer programs.

- **Surge tank and Penstock**

- The EPC Contractor should perform the geologic mapping during the detail design stage with a scale of 1:500 along the Surge Tank and Penstock alignment to capture the ground information. Moreover, additional geological and geohazard studies shall also be carried out focusing geomechanical properties, terrain characterization and rockmass categorization.
- Any alternations/ adjustments in the locations of Surge Tank and Penstock alignment shall be finalized after terrain characterization and geohazard studies
- Piers will support the penstock alignment, as proposed. In this section, the piers are laid down on overburden material and the presence of clayey silt may create problem and might be excavated prior to establish the foundation for piers.
- Appropriate confirmatory geotechnical investigations shall be performed by EPC Contractor for Surge Tank and Penstock location to assess the strength and other geotechnical parameters of soil and rock.
- For Surge tank Structure, detailed foundation design analysis is required for checking the stability.
- Slope stability analysis is required for the slopes of Surge tank proposed area.

- **Powerhouse and Tailrace Channel**

- The EPC Contractor shall perform the geologic mapping during the detail design stage with a scale of 1:500 in power house area to capture the ground information. Moreover, additional geological and geohazard studies shall also be carried out focusing terrain characterization.
- More refinements, in the proposed and optional locations of Power house should be considered after detailed terrain characterization, geohazard studies and on the basis of the Design stage Investigations if required.
- Appropriate geotechnical investigations and laboratory testing is required for the determination of geotechnical parameters.

- Detail foundation design analysis are required during the detailed design stage by the EPC Contractor.

5.6 REVIEW OF CONSTRUCTION MATERIAL STUDIES

Appropriate selection of the material sources for construction of a project put major impacts on its durability, performance, cost and quality. In the context, a systematic investigation will be required to identify suitable sources for various construction materials. Sources of construction materials will be identified and confirmed for its suitability as per project requirements in the closest vicinity of the project to save time and cost.

Feasibility studies of the proposed hydro power project was carried out in June 2021. Construction materials study conducted during the Feasibility stage including test pit logs and laboratory test results etc. are reviewed and found generic and lacking in many aspects.

5.6.1 General Review

- Quantification of construction material suitable sources is not done. It is very important to quantify the suitable sources with respect to the material requirement in the project.
- For qualitative analysis, few mandatory tests for finalization of the suitability of construction materials are missing which are mentioned below in material specific review comments.
- To conclude whether any source has ASR potential, accelerated mortar bar test (ASTM C1260) along with Petrographic examination (ASTM C295) should be done.
- No established rock quarries in the vicinity are studied in this report. Neither any information regarding available established rock quarries is documented. Previous studies of the materials utilized in the construction of three tunnels (left bank of Attabad Lake) can also be useful for the current project.
- For cement, steel, bitumen, concrete admixtures and others manufactured materials, general information is given in the report. Specific studies related to the availability of production of cement types, steel grades-wise and others, are missing in the report and is recommended to be carried out before the preparation of BOQs.
- Attabad Lake itself is the biggest possible source of mixing water used in concrete, which is also not investigated through detailed testing in accordance with ASTM C1602, in this report.
- Fill material requires more investigation and testing to check and finalize the sources. Selection of fill material keeping in view the permeability requirement with respect to each component, is of significant importance. To prevent from seepage, different soil types are required and should be studied separately and sources should be located accordingly in the closest vicinity.
- At least one location map should be prepared showing all the construction material sources discussed in the relevant section.

5.6.2 Material Specific Review

- **Fine Aggregate**

Refer section 7.2.1 of the Feasibility report. One sand quarry with only one test pit (TP-1) is studied at Feasibility stage. This is a river bed quarry located in the tail of Attabad Lake. Quantification was not done. Petrography was also not done. Without doing petrographic test, Alkali Silica Reaction potential cannot be concluded. Sieve analysis was done showing percentage of Sand, Silt and Gravel (USCS classification). No fineness modulus was mentioned which is a mandatory requirement for sand size confirmation. Gradation analysis was given in the form of graph and sieve analysis details was not given. Soundness test and other deleterious tests (guidelines of ASTM C33) are also missing in the Feasibility Report.

- **Silt and Clay Quarry**

Refer section 7.2.2 of the feasibility report, 3 test pits were done to analyze the fill materials availability in the project area. TP-2 & TP-3 were done in helipad/powerhouse area and TP-10 near Kiln area, near Ganesh Bridge. These natural soils were not classified as per fill material type requirement. Quantification was also not done.

- **Coarse Aggregate**

Refer section 7.3 of the feasibility report, three sources (Slided mass, power house area/river bed and Passu areas) were selected and sampling and testing were carried out accordingly during Feasibility stage for coarse aggregate. Quantification is not done for above three sources.

Passu area quarry is located 35 km upstream of Attabad Lake near Passu village. Rock is marble and have economic importance and generally not permitted to be used as coarse aggregate for concrete (need to check the laws of mineral department, Govt. of Gilgit Baltistan). Petrographic result shows no ASR potential. Remaining two exhibit ASR and cannot be used with ordinary Portland cement. Both sources have significant quantity of fine materials which are not noted. More than 20% deleterious quartz as indicate in their respective petrographic examination make it highly susceptible to ASR potential. Slided Mass (TP-7) shows higher Loss Angeles Abrasion and cannot be used as coarse aggregate in concrete.

5.6.3 Recommendation for Construction Material Studies

- Missing testing is recommended for Fine Aggregate to be done with more samples collection from the source to finalize the suitability of the said source. New sand quarries are also recommended to be located for fine aggregate utilization in the project.
- It is recommended to classify silt and clay soils as per ASTM soil classification system.
- As already discussed, that established rock quarries should be studied and documented accordingly. More rock quarries in the vicinity are recommended to be investigated to finalize the suitability of the coarse aggregate.

6 REVIEW OF NEOTECTONICS STUDY AND SEISMIC HAZARD ANALYSIS

6.1 GENERAL

A review of the available report/data related to seismological studies was carried out, the following report/data have been reviewed in this regard;

- "Feasibility Report-2021, Volume-IV, Neotectonic and Seismic Hazard Analysis, Attabad Lake Hydropower Project, WAPDA".
- Additional data via email dated September 22, 2022 titled "Attabad HPP: Data requirement for Review of Neotectonics & Seismic Hazard Study".
- Letter with additional data titled "Attabad LHPP-clarification regarding review of Seismic Hazard Study dated October 11, 2022".

6.2 REVIEW FINDINGS

On the basis of review of the above mentioned reports/additional data, the findings are as under;

- As a part of Feasibility Study (2021), the seismic hazard evaluation was carried out for selecting the seismic design parameters. The following seismic design parameters were recommended;
    i. Safety Evaluation Earthquake (SEE) = 0.44g for Dam/Weir & 0.38g for Spillway
    ii. Operating Basis Earthquake (OBE)= 0.24g

Overall the work done (seismic design parameters) during feasibility stage needs to update/finalize before or during Detail Design stage due to the following reasons;

- No Ground Motion Prediction Equation (GMPE) for the deep seismic source zone (Hindukush) is used in calculation of seismic design parameters.
- Above mentioned selected seismic design parameters should be comparable with the findings of PSHA (Total Hazard Curves, Response Spectra) and/or DSHA (Table 6.2-1 & 6.2-2 of Feasibility Study).

6.3 RECOMMENDATIONS

It is recommended to carry out following surveys/studies before or during Detail Design stage;

- Detailed site specific seismic hazard assessment should have been conducted to authenticate & finalized the seismic design parameters.
- Geologic mapping of faults is also recommended, specifically to confirm the local faults as mentioned in the report.
- Necessary geophysical surveys in support of seismic hazard assessment, where required.

7 REVIEW OF PROJECT LAYOUT STUDIES

7.1 GENERAL

Two alternate layouts were studied at Feasibility stage 2021. One was with project components on left bank and one on right bank. A comparative study was done at Feasibility stage for these alternates. Both studies alternates are described in following sections.

7.2 ALTERNATIVE-I: PROJECT LAYOUT ON THE LEFT BANK

In the Feasibility study, it is stated that the left bank alternative with Weir and Under sluices, 2.3 km headrace tunnel, surge tank, embedded penstock and a surface power house is favorable alternate and considered suitable to propose a Project. Review of Feasibility study implies that selecting the left bank option is a legitimate decision in terms of topographic conditions and ease of access.

7.3 ALTERNATIVE-II: PROJECT LAYOUT ON THE RIGHT BANK

Alternative-II, which is the right bank option, is a less favorable option for Attabad Lake HPP in terms of access, stability of slopes, and ease of construction and same assessment was done during Feasibility study.

The water intake structure planned in alternative-II is far away from the Weir body. For this reason, it is not possible to flush the sediment that will be deposited in front of the intake.

7.4 SELECTED LAYOUT AT FEASIBILITY STAGE 2021

The project layout studies were refined during Feasibility study after selection of left bank alternate and further following two alternates were studied on left bank option in detail.

a. Alternate-1A, Headrace Tunnel Option.
b. Alternate-1B, Headrace with Embedded Pressure Pipe.

The selection of the Project Layout (Alternative-I A) on the left bank with identified weir site, 2.3 km long headrace tunnel, 550 m embedded penstock and surface powerhouse in the Feasibility study was more beneficial layout plan for this project in terms of geology, cost/kWh for design discharge, ease of construction and technical reasons.

The selected Project Layout (Alternative-I A) from Feasibility study is reproduced in Figure 7.1.

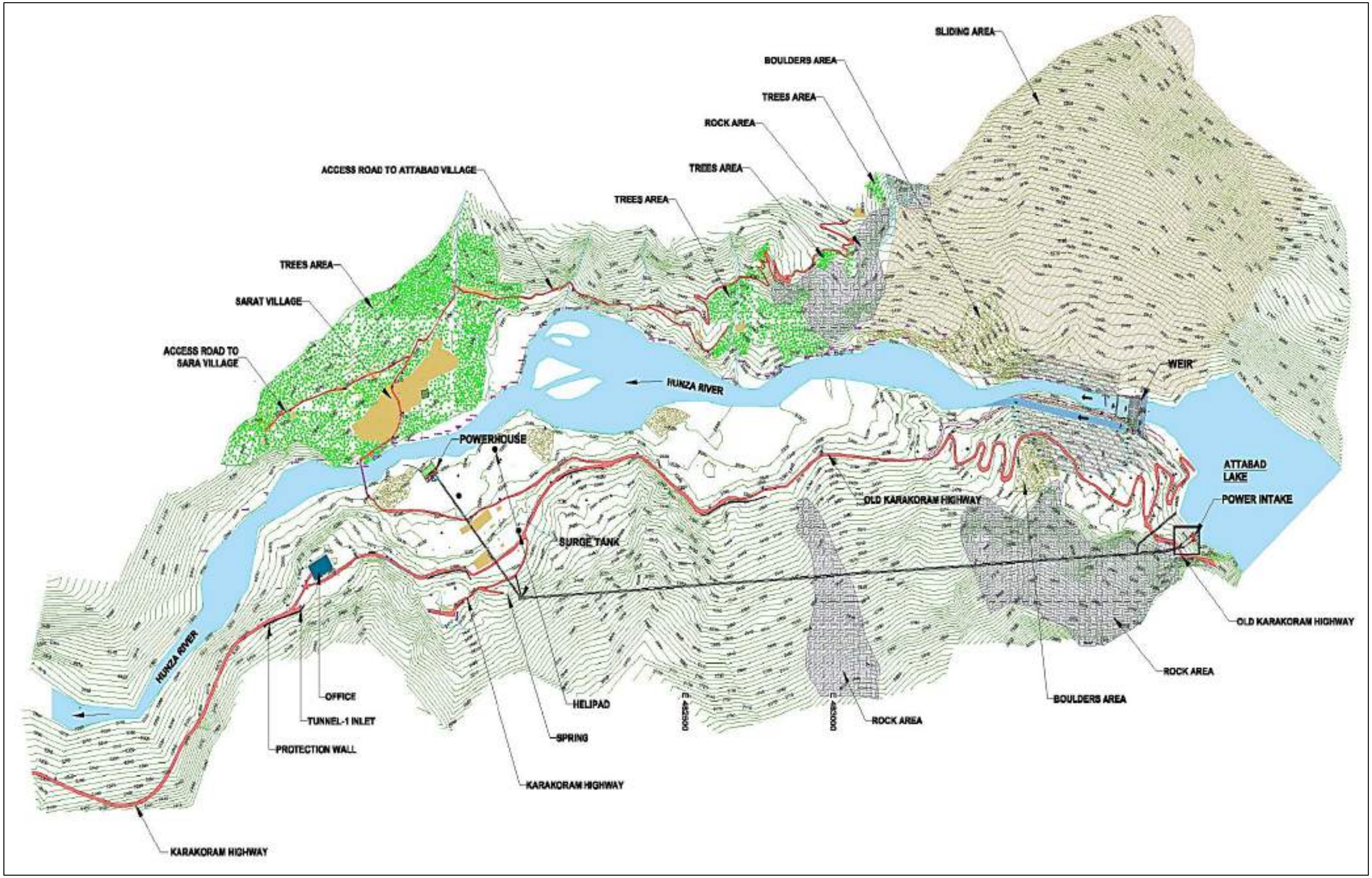


Figure 7.1: Feasibility stage Project Layout Alternative-1A

7.5 PROJECT LAYOUT STUDIES AT REVIEW STAGE

The natural Dam created due to landslide resulted in formation of a Lake behind it at Attabad. It not only possess a threat but at the same time an opportunity for making use of the head due to this Earthen Dam. Due to lake formation, there is potential drop of more than 100 m in 2.5 km river reach. The mean annual flow of Hunza River at Attabad Lake is 204 m^3/s. The Project Layout is to be configured considering the power demand and demand pattern of regional grid. Attabad Lake storage can be used for daily peaking. A design discharge with peaking storage facility is to be selected to generate the installed capacity 95% time of the year in peak hours. From the available flow data and estimated flow series at Project Area, a discharge of 60 m^3/s has been taken as design discharge.

The Management Consultants have carried out comprehensive review of Project layout and have thoroughly reviewed the technical aspects of all Project components proposed for the Feasibility study 2021 Alternate-I A. Reconnaissance site visit was also conducted to get acquainted with the proposed sites and to give recommendations for EPC Contractor detail design.

7.5.1 Site Visit Observations and Feasibility Review Findings for Project Layout

Following sections describe the site visit observations of proposed sites of project components. These site observations are supported by Review findings of Feasibility study 2021.

- **Weir / Spillway and Under Sluice Area**

The slided mound area proposed for the main Weir / Spillway and Under Sluice structures was visited. Landslide material was observed on both banks. Feasibility stage Geotechnical Investigations in this area did not report the bedrock even as deep as 90 m. The overburden was reported to comprise of ABGM of slide mass (in BH-01 and BH-02) or sandy gravels (in BH-03). Although the description may also vary as gravels, cobbles and boulders embedded in silty sandy matrix of mix origin, the physical presence and extent of this strata especially the boulders was realized with this site visit. Excavation for abutments and foundations is risky under such conditions and might be in higher volumes than proposed in Feasibility.

Feasibility stage Weir and Under Sluices layout and longitudinal section is reproduced in Figure 7.2 and Figure 7.3.

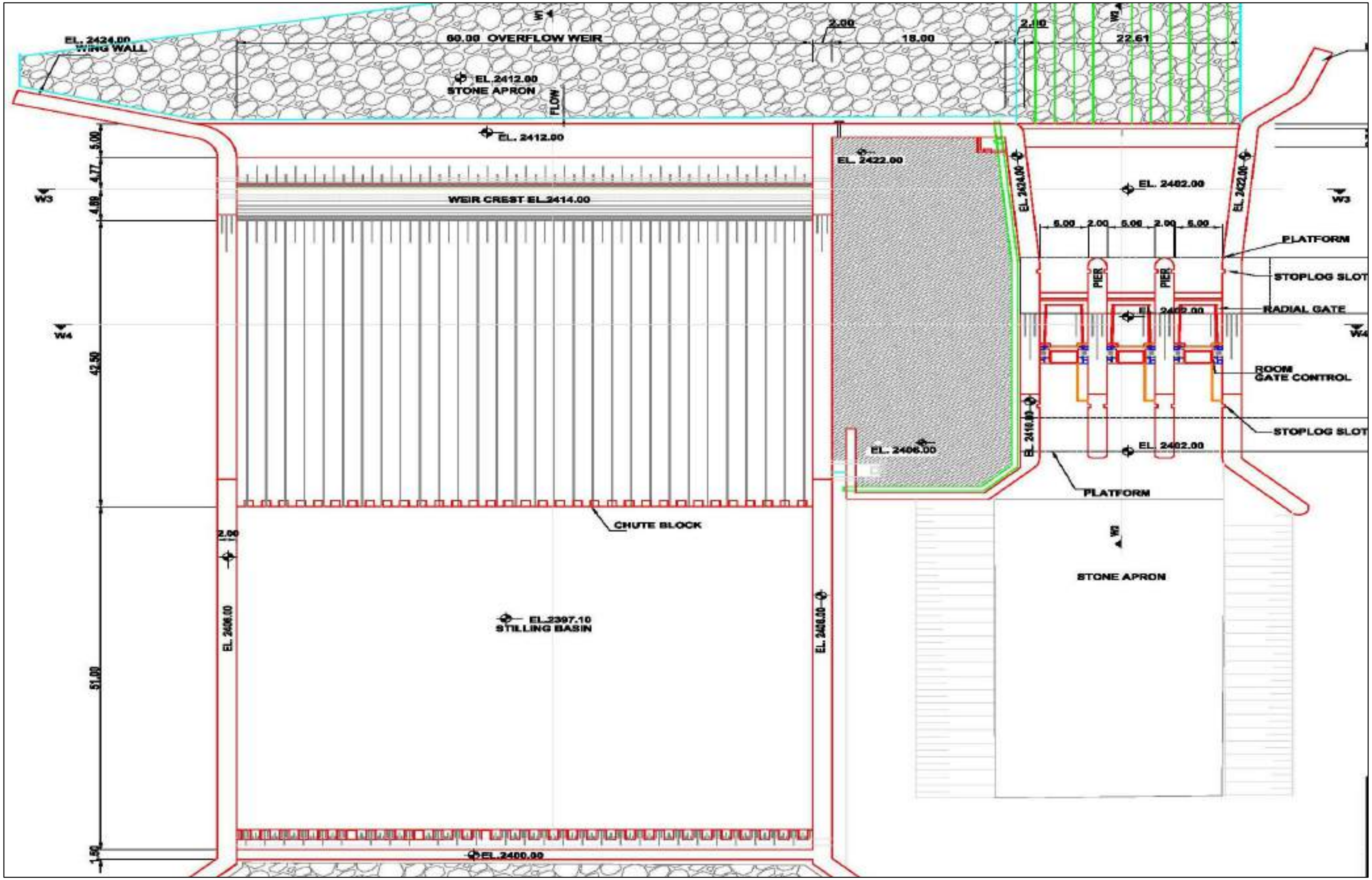


Figure 7.2: Weir and Under Sluices layout plan from Feasibility Study

![AHMC logo]

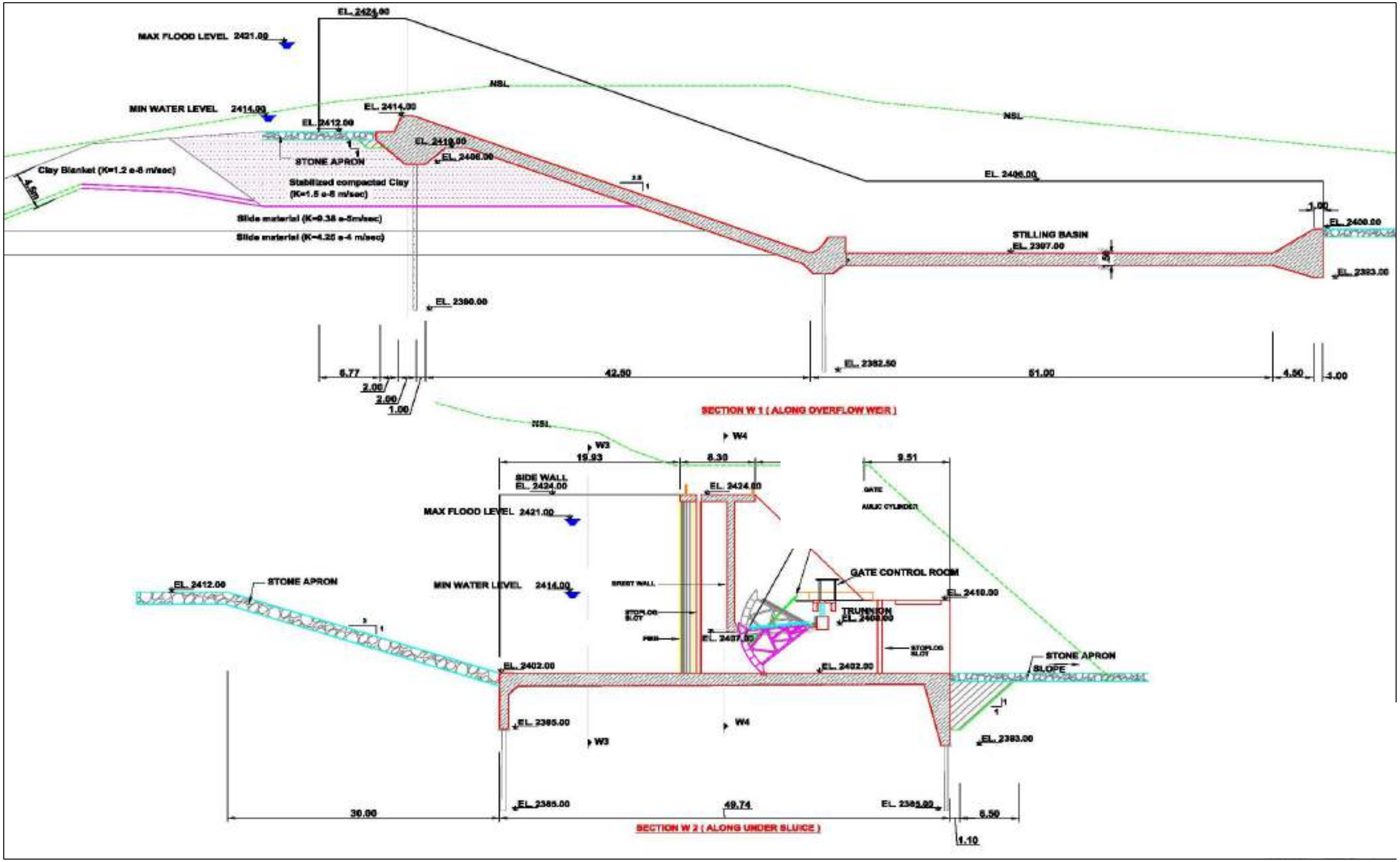


Figure 7.3: Weir and Under Sluices longitudinal section from Feasibility Study

Although, the location of the crest of the proposed Weir and the approach channel were visible, the spillway chute area was inaccessible due to collapse of a portion of the track that runs along the edge of the spillway excavated by FWO. This only reinforces the need for stabilizing the embankment, particularly, in the region of proposed weir.

Weir and Under sluices area needs special care on foundation treatment and seepage under the structures as well as through the banks / abutments. Specific considerations in this regard include the requirement and effectiveness of measures to be adopted for seepage control, practicality of the methods to be utilized for required excavations in slopes and foundations, measures required for stabilizing and / or reinforcing the required excavations, along with the equipment mobilization and installation constraints. Construction of piles / seepage cutoff proposed in Feasibility might be difficult due to presence of boulders. Bentonite mixtures for excavation stability might also be ineffective. Concrete diaphragm or sheet piles will also be difficult. For these reasons, MC initial proposal for ensuring the stability and the seepage of the Weir site is the consolidation grouting using sand mixed grouting with hardening accelerating admixtures. The depth and pattern of the consolidation grouting to be defined in the next design stage (by EPC Contactor).

River bed also needs stabilization to avoid differential settlements of the proposed gated structures. The variability of slide material along with large size boulders present in the slide mass also necessitate a sensitivity / back analysis for justification of the geotechnical parameters to be considered.

The slow erosion of fines from the slide mass from both sides, in contact with the flowing waters of the river was another point of concern. The muddy water was observed due to migration of fines on edges reflected inherent instability of the loose mass to an extent which was alarming while considering this stratum for founding the sensitive structures with hydro-mechanical fixtures like gates, slots etc.

Some of the photographs of Feasibility stage proposed Weir and Under sluices site as taken during reconnaissance site visit are shown in Figure 7.4 through Figure 7.10.

Figure 7.4: Weir site view from Attabad Lake

Figure 7.5: View from Just Upstream of Weir Axis

Figure 7.6: Upstream View from Weir Axis, Muddy water & migrated fines on banks

Figure 7.7: Excavated Spillway view from Weir Axis

Figure 7.8: Right Abutment view from Left Abutment, Slide Mound of ABGM

Figure 7.9: Left Abutment view on side track, Large size Boulders from slide material

Figure 7.10: Left Abutment, Fresh Slide area and completely damaged walkway / path towards chute area

- **Intake Area**

The intake area is inaccessible except by boat. The intake appears to be located in relatively sound rock (as good as any seen in the area) and the location is geologically appropriate. However, its construction should pose significant challenge. It is anticipated that there are likely to be structural issues regarding the design of coffer dam and the access chamber for intake gate, trash racks and stop logs. Further, the Hydrology, Sedimentation and Hydraulic model data is being reviewed to assess these structural issues. Photograph of Feasibility stage proposed Intake site as taken during reconnaissance site visit is shown in Figure 7.11.

Figure 7.11: Intake Area view from Upstream of Weir, Sound Rock Exposed

- **Power Tunnel and Surge Shaft Area**

Geology of Power Tunnel and Surge Shaft area seems appropriate with Granitic Gneiss as also presented in the surface geological maps from Feasibility. The location of surge shaft was flagged to assist in visualizing the alignment of the Penstock to the Powerhouse. The access to surge shaft and headrace tunnel can approached through old KKH.

Four (04) highway tunnels exist in the general area in which the Power Tunnel is to be constructed, so its construction might not pose any problem. The details for the existing highway tunnels of KKH, i.e. the actual alignments marked on plan and profile, the geological mapping information during their construction phase, etc were requested by MC to Client to support and validate geological conditions along the proposed Tunnel. Client arranged construction and as-built drawings of these existing KKH tunnels with alignment, profile and geological data. These support documents were studied by MC to validate the alignment and geological conditions.

The closest part of the headrace scheme to the KKH Tunnel is at surge tank. At this point, the surge tank bottom elevation is 2385 m, whereas the KKH tunnel invert elevation is 2376 m. The perpendicular distance between the outer walls of the structures is 65 m. Since the geological formation is defined as moderately weathered gneiss in the as-built drawings of the KKH tunnel, the interaction between the structures is regarded as safe.

The measures to be taken at the fault zone that is assumed to be present at the middle section of tunnel during site visit which the low pressure tunnel is passing through shall be clarified and incorporated in the tender design. On the other hand, the as-built drawing of KKH tunnel no 2 states that the section that is parallel to the low pressure headrace tunnel is completely in moderately weathered gneiss and a fault zone is not mentioned.

Photograph of Feasibility stage proposed Surge shaft site as taken during reconnaissance site visit is shown in Figure 7.12.

Figure 7.12: Flagged Surge shaft Area, Sheared and Jointed Rock Exposed

- **Penstock**

A single Penstock leads from the surge shaft to the Powerhouse in the feasibility study proposed by WAPDA. Except for the upper reach, the proposed Penstock alignment traverses through relatively easy terrain

In the upper reach, there may be some slope stability issues, although, it appears that the rock line is near the ground surface. Further towards lower elevations, the rock line appears to recede away from the natural ground with fine material on surface with sand and gravel lenses.

It is proposed to drill two boreholes under the anchor blocks in order to have information to be used for the design of anchor blocks. These boreholes can be used also for the powerhouse.

- **Powerhouse Area**

The Powerhouse, as proposed in the Feasibility report, is located on flat terrace along left bank of Hunza River. The site was flagged and marked so it could be identified clearly quite adjacent to the left bank of the river Hunza.

It looks as if rock may not be available under the Powerhouse footprint. However, the area looks satisfactory for construction of the Powerhouse and other appurtenant structures. There also appears to be no slope stability issues at the proposed site.

Photographs of Feasibility stage proposed Powerhouse site as taken during reconnaissance site visit are shown in Figure 7.13 and Figure 7.14.

Figure 7.13: Marked Power House Area near Left Bank Hunza River, Overburden visible along the Penstock Alignment

Figure 7.14: Flagged Power House Site

7.5.2 Proposal for Detail Design Regarding Project Layout

A modification in Feasibility stage Alternative-I A layout is proposed by MC based on described review findings and site visit observations. The layout modified by MC is “Modified Alternate-I A” and is presented in figure 7.15. The modifications are to be checked for technical and economic viability during Detail design stage by EPC Contractor. Following are components of this proposal and recommendations for Detail Design are also given.

- **Weir Body**

The naturally formed Attabad Lake dam body due to the landslide has been surviving since 2010. The prevailing scenario of overflow indicates that existing conditions are naturally stabilized. Attempts to excavate side slopes for enlarging the area and/or deepening to place structural foundations has the potential to destabilize the present in-situ conditions. Keeping this fact in mind, MC is proposing to utilize the naturally formed dam body in order to minimize the total cost of the project and to avoid aggravating any further potential geotechnical problems. The naturally formed embankment is proposed to be covered with “flexible gabion mattress” in order to take extra precaution to defend against any further surface erosions due to floods. The gabion mattress offers a prime advantage that the structure can be changed in height and size simply by building up or removing courses of gabions on the existing formation. After a period of operation, the shape of the structure can be adjusted according to requirements, and progressive adjustments can be made thereafter. This way of design will eliminate the necessity of any concrete structure on the Dam axis reducing the cost significantly.

Generally, gabions offer the most convenient solution than most other materials in such situations when substrata problems of permeability, erodibility, and bearing capabilities are faced. Moreover, in addition to their flexibility, which allows them to deform while remaining structurally sound, other advantages which gabions offer are relatively low cost and simplicity of construction. This is specifically significant for this site, where stone is locally available, sophisticated plant and equipment are unnecessary, Skilled craftsmen are not needed, and labour can be fully trained within a matter of days to carry out assembly and erection of the units in the required manner.

However, design of the gabion mattress should involve required hydraulic and structural stability considerations for detailing the geometry, size and fillings. Moreover, numerical and physical model should be undertaken.

The originally proposed 19 m wide under sluice is to be excavated on the left side of an overflow weir / ungated spillway, which is 60 m wide. This under sluice section is proposed primarily for drawdown of the lake level below El. 2400 masl to allow commencement of construction works on the power intake structure. Canceling the previously proposed flushing part of the Weir in the Feasibility Study would make dewatering of the construction site of the power intake and newly proposed flushing tunnel difficult. This is an important disadvantage of the newly proposed layout that should be kept in mind. Conversely, the rock face at the power intake location can also be used to advantage of this alternate proposal. A downstream rearrangement of the intakes (both power and flushing) would allow commencement of construction works on these locations behind a retained natural mound of the rock face acting as a natural plug for the upstream lake levels. The interim construction access can be managed from the downstream around and over the slided mass area to allow uninterrupted construction activities, independent of the lake water levels. The natural upstream plug can later-on be removed near completion of all works by pre-splitting and controlled blasting techniques.

Construction stages should be considered and discussed thoroughly in the next design phase with **tentative Construction Methodology** as follows.

During low flow season i.e. December, January, February, March and April, the Contractor can work on Weir site by diverting water in two stages. In first stage, local diversion Contractor shall laydown gabion mattress on one side and then remaining portion can be laid down during second stage diversion.

- **Flushing Tunnel**

Since the Under sluices that will be utilized for sediment flushing are being avoided in the proposed alternative layout due to geotechnical issues, a tunnel at a lower level than the power intake structure is proposed to flush out the sediment that is likely to be deposited in front of the intake structure.

The flushing tunnel underneath the intake area is proposed at distance of 7m away in the plan view and flushing tunnel outlet will be after the most disturbed area. In this case the life of Powerhouse can be increased by many years. Only risk involve in this proposal will be outlet portion of the flushing tunnel which will lie in overburden area. Cut and cover conduit or even open channel can be given in this area after a transition. In worst conditions if some damage happens to outlet structure, it can easily repair any time because flushing will not be done throughout the year. One more advantage will be available to us that power house will be running continuously during repair time of this outlet structure.

It is foreseen by MC that a flushing tunnel with a diameter of 5.50 m and a length of approximately 1100 m might be constructed beneath the power intake structure. This flushing tunnel will be capable of flushing about 250 m^3/s discharge with a velocity of approximately 10 m/s which will be enough to convey the sediment downstream without any deposition. The velocity during the operation should not be less than 4 m/s in order not to have sedimentation inside the flushing tunnel. It is recommended to control the flushing tunnel with a gate located at the entrance, as the approach of the access tunnel to the valve chamber remains within the landslide and poses difficulties. Hydromechanical equipment, gate and gate slots must be designed with the corrosive effects of sedimentary water in mind.

The proposed flushing tunnel inlet base elevation is recommended to be 2380 m leaving a distance of approximately 2 x D between the headrace tunnel and flushing tunnel which is thought to be enough for this type of rock formation. The rock formation at the inlet area is classified as Granodiorite and is suitable for tunnel construction. The slope of the flushing tunnel might be selected approximately as 0.5%. The exit elevation of the flushing tunnel will be approximately at 2375 m.

The water velocity is limited (nearly 10 m/s) in the proposed flushing tunnel. Therefore, there is no need for an extraordinary energy dissipater structure at first glance. But riprap or gabion mattress should be placed at the outlet of the flushing tunnel for surface protection. It is also recommended to study a suitable energy dissipater structure in the next design stage. But the detail designer has to consider the possible deposit of sediment at the outlet of the flushing tunnel (inside the proposed energy dissipater structure).

The downstream part of the tunnel will pass through the landslide (about 250 m long from the outlet), and it is difficult to do tunnel excavation in this region. Therefore, the excavation in this region can alternately be open-cut and the system in this region might be an open channel system. However, if there is a probability of a landslide forming in the open cut slopes of this channel, then a closed conduit might be constructed to avoid the probability of blocking the tunnel outlet and the channel by landslide.

Regarding the **tentative Construction Methodology,** Contractor will approach the intake area of power and flushing tunnels trough an adit from the existing road leaving 3 to 5m plug area of existing rock. Once construction of power intake and flushing tunnel is completed then contractor can remove this rock plug.

Since the sedimentation and flushing of the reservoir depend on lots of variables, the flushing tunnel diameter, discharge capacity, effectiveness of the system and intake level must be determined with a hydraulic numerical model study / physical model study by EPC contractor. The EPC Contractor should carry out the detail design of the tunnel in such a manner that any damage (abrasion, cavitation, etc.) to the concrete cover or rock mass in the flushing tunnel can be prevented during flushing operation.

Following are the recommendations regarding hydrological and sedimentation aspects for proposed flushing tunnel.

- Flushing tunnel should run periodically to remove silts deposited in front of power intake or nearby power intake vicinity.
- Flushing tunnel is purposed to be adopted for removal of certain estimated capacity of sediments, not for flushing maximum sediment volume.
- Flushing tunnel should be operational by considering reservoir storage capacity in balance position.
- Water availability, Reservoir operation for power generation and Flushing tunnel action to be linked in such a manner to keep project parameters and objectives functional.
- Flows study indicated that after utilization of 60 cumecs design discharge, water releases / spill periods are (May to October) with maximum releases in July (679 cumecs) and August (649 cumecs), respectively. Lean period from (Nov to April), where shortages are reduces by utilization of reservoir live storage water.
- Selection period for flushing is to be in July and August. The water release volume would be used for flushing purpose, which keep the reservoir storage capacity in balance. Sediment study also reflected, maximum sediment deposition in July and August. So, July and August are ideal periods for flushing tunnel operational, which would enhance project life and meet the objectives in a successful manner.

- **Power Tunnel Design Aspects**

Power Tunnel alignment seems appropriate. However, surface overburden is present along the Power Tunnel alignment as presented in the surface geological maps from Feasibility study which might be an indication of less rock cover. This aspect is to be studied during Detail Design studies by EPC Contractor. Moreover, the surge shaft / end portal of the Power Tunnel appears to end in sheared and jointed rock in terrain susceptible to external slope stability failure. This has to be given due consideration in Detail Design studies, particularly in view of the fact that the area is prone to earthquakes of large magnitudes.

- **Powerhouse**

Surface Powerhouse as proposed in Feasibility study is located adjacent to main river on an alluvium terrace. To minimize cost of protection works from maximum flood and for its increased safety, the Powerhouse may be moved to a slightly higher ground. It would reduce the Penstock length, at the cost of some extra excavation being proposed on an elevated surface profile for Powerhouse. The excavation may also additionally safeguard against structural stability issues. However, the primary measures of foundation design might remain similar to the pervious conceptual location, due to similarity in the strata. At the proposed site, surface powerhouse can be constructed. Surface powerhouse, tailrace channel and bifurcated penstock can be placed at required elevation by excavating the alluvium.

Another possibility, which can be explored, is underground powerhouse. In case of underground length of steel liner will become minimum length while length of tailrace channel will become longest. In this case head losses will become minimum. The project area is located in zone-4 regarding seismic activities. It is evident from past experiences that underground power houses behave much better as compared to surface Powerhouses during any seismic hazard activity. If economics allow, technically this option will be more stable safe with minimum O & M requirements.

7.5.3 Future Prospects of Modified Layout

A schematic drawing showing the proposed modifications in Feasibility study Alternative-I A layout is presented in Figure 7.15 as “Modified Alternate-1A layout”.

If in case all slide material is washed away at Weir site then a temporary coffer dam can be added in front of intake area in a shorter duration which will restore power house and then a new / fresh Weir/ Dam in this area can be constructed.

If intake structure is moved to Weir location and in case all slide material washes away then in this scenario considerable time will be required for restoration of Weir/Dam for generation of power.

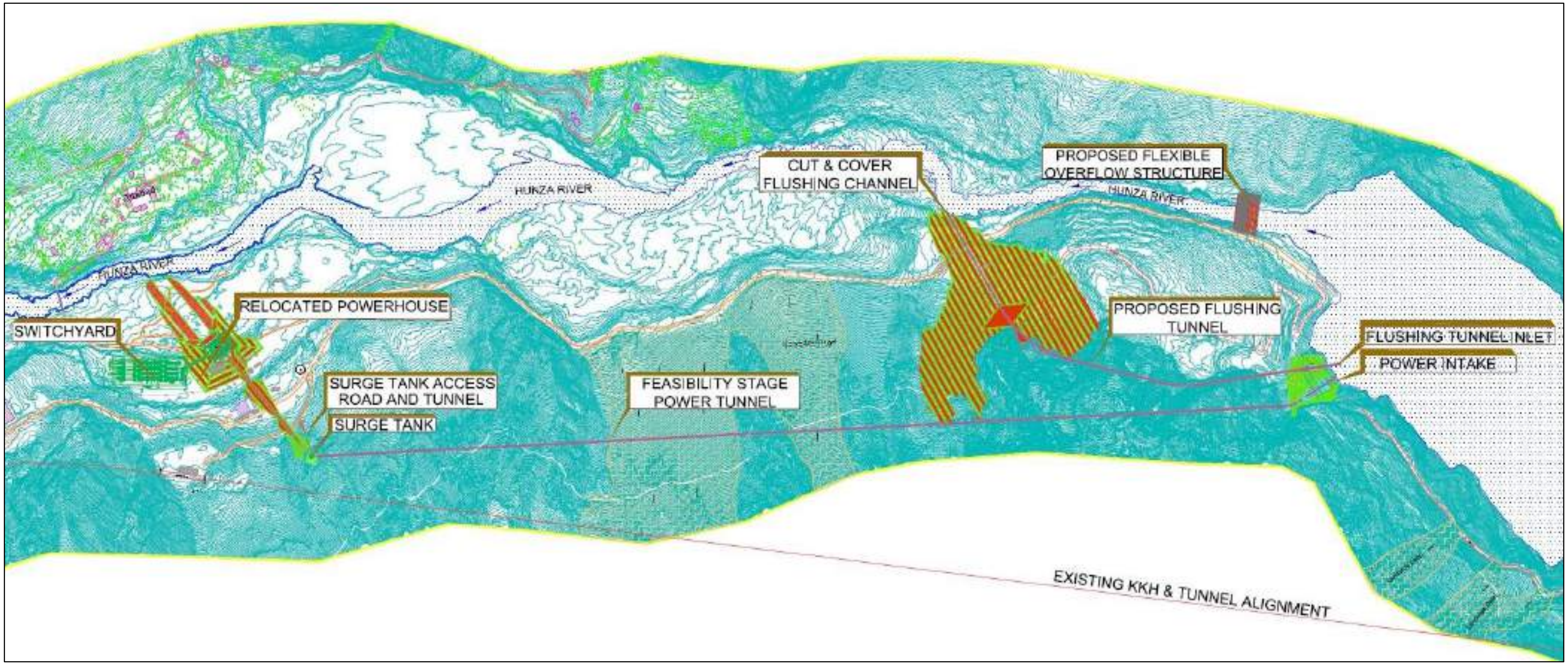


Figure 7.15: Proposed Modified Alternate-1A layout

8 REVIEW OF POWER POTENTIAL STUDIES

8.1 GENERAL

The demand for electricity varies seasonally. In lower areas of the country demand reaches to its maximum in summer season and remains lower in winter. But in northern area of the country especially in Gilgit-Baltistan (GB) demand for power and energy is maximum in winter season and remains within lower ranges in summer. Electricity demands also vary during the same day, people use more power in the morning and in the evening. Therefore, many powerhouses are managed to tackle such peak times. For this purpose, temporary storage is done in the lake/pond/forebay and releases during peak hours. Such projects have small storage as compared to large dam's storage.

As the lake had already been developed so reservoir capacity has not been enhanced than present to avoid inundation of upstream areas due to increase of water levels, however, flushing will be carried out to retain the present capacity of the lake. Area Capacity Curve from Feasibility study is shown in Figure 8.1.

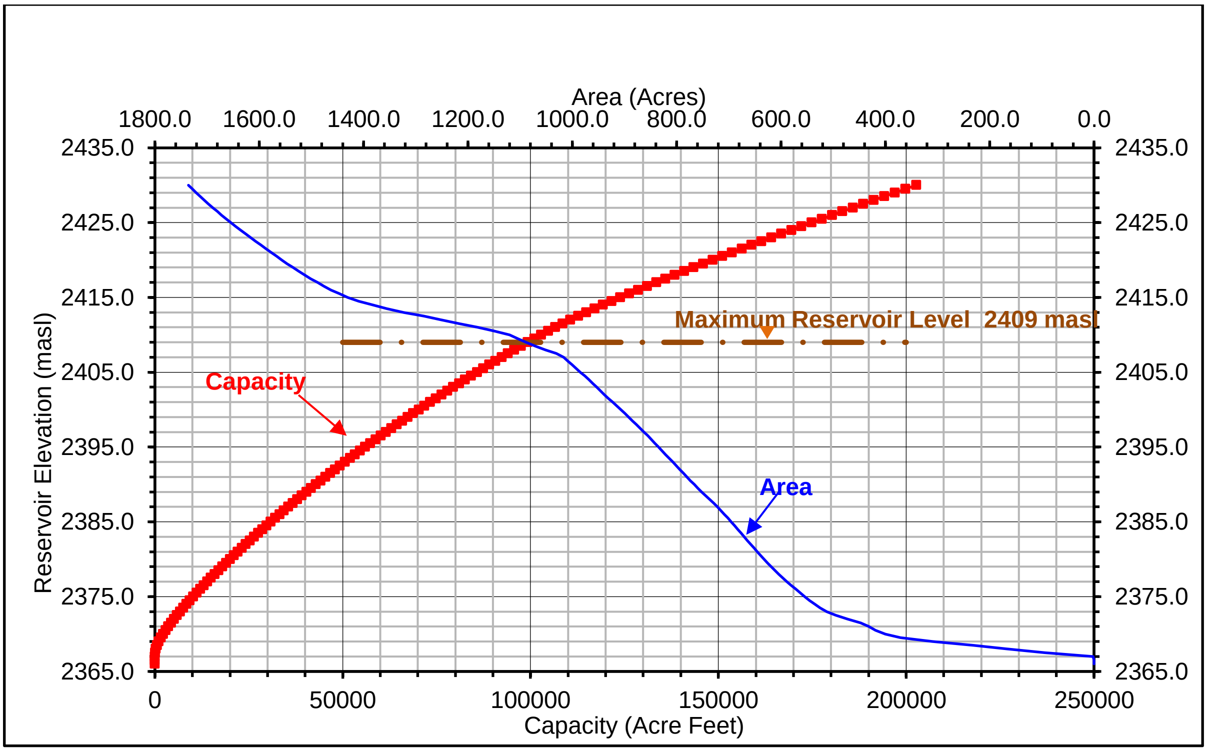


Figure 8.1: Area Capacity Curve

8.2 WATER AVAILABILITY AT RESERVOIR

Mean annual flows are calculated for the period 1966-2015. Average value for mean annual flow has been calculated as 208.36 m^3/s. Flows are generally maximum in July and August almost touching 700 m^3/s. Whereas, flows are low from December to April each year when snow falls in the catchment. The graph shows that design discharge of 60 m^3/s is available for about half year (almost 6 months). Graph also shows that flows are exceeding the design discharge from June to mid-December which shows the potential of hydropower generation. Graphical presentation of Mean Annual Flows is shown in Figure 8.2 and graphical presentation of Mean Monthly Flows is shown in Figure 8.3.

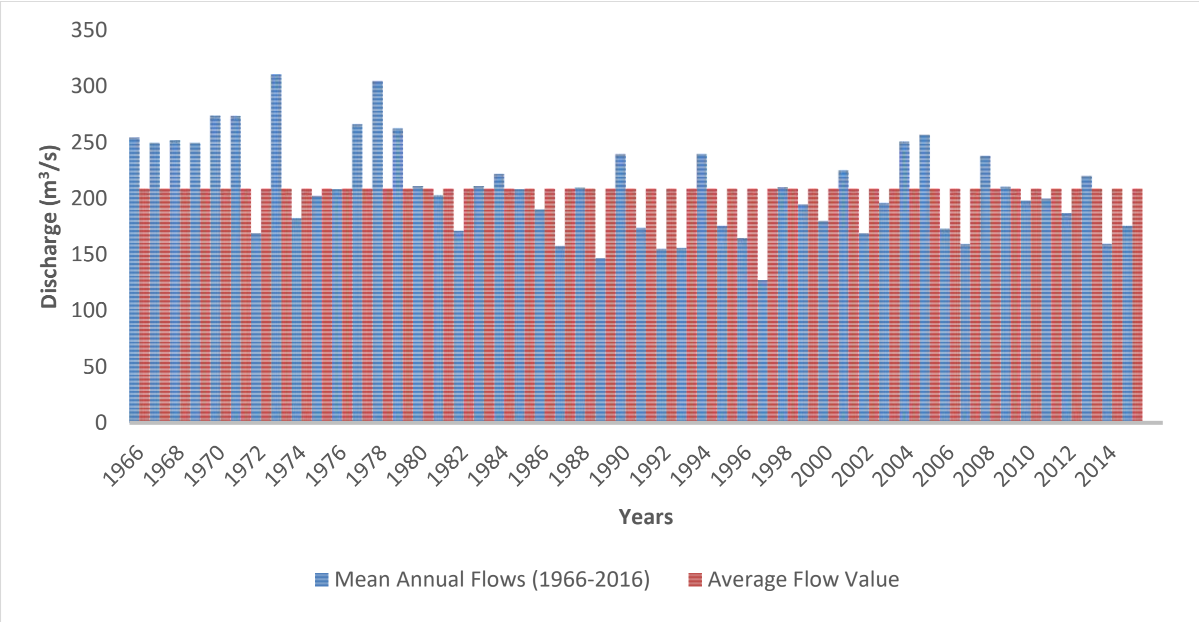


Figure 8.2: Mean Annual Flows

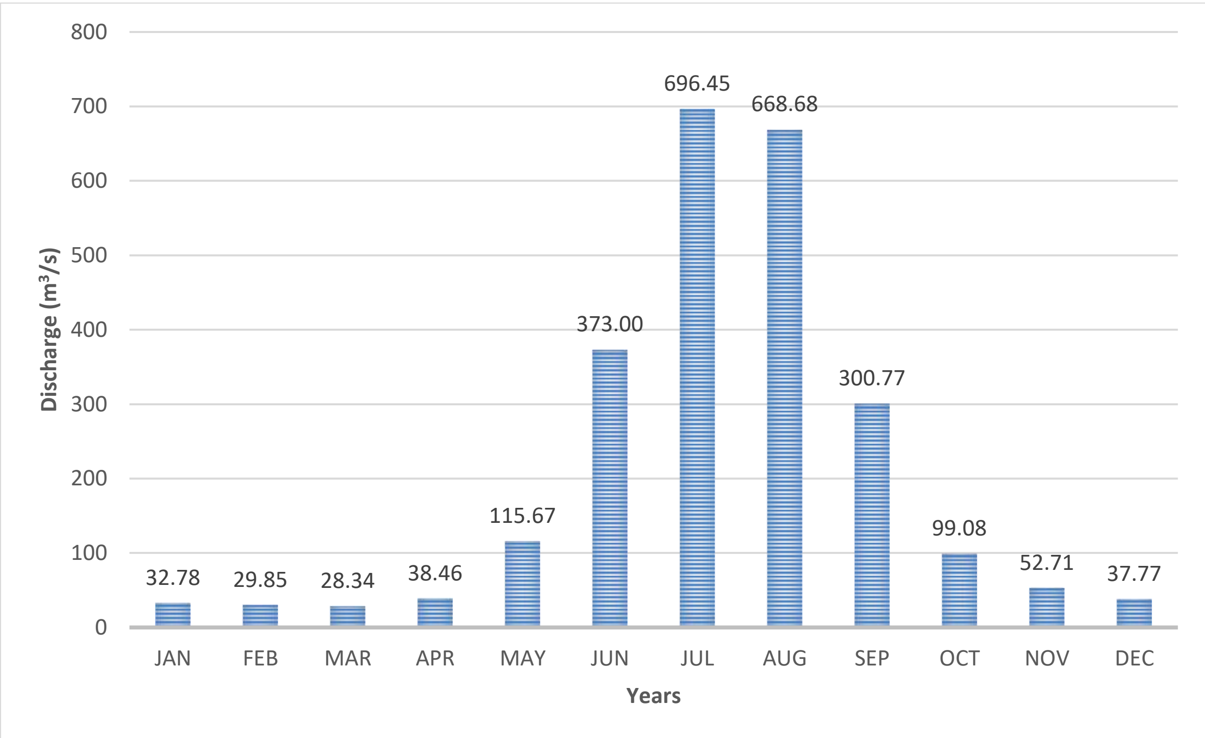


Figure 8.3: Mean Monthly Flows

Maximum, Mean and Minimum daily flows for the period 1966-2015 are also plotted in the Figure 8.4 to depict the flow pattern at Attabad Lake site.

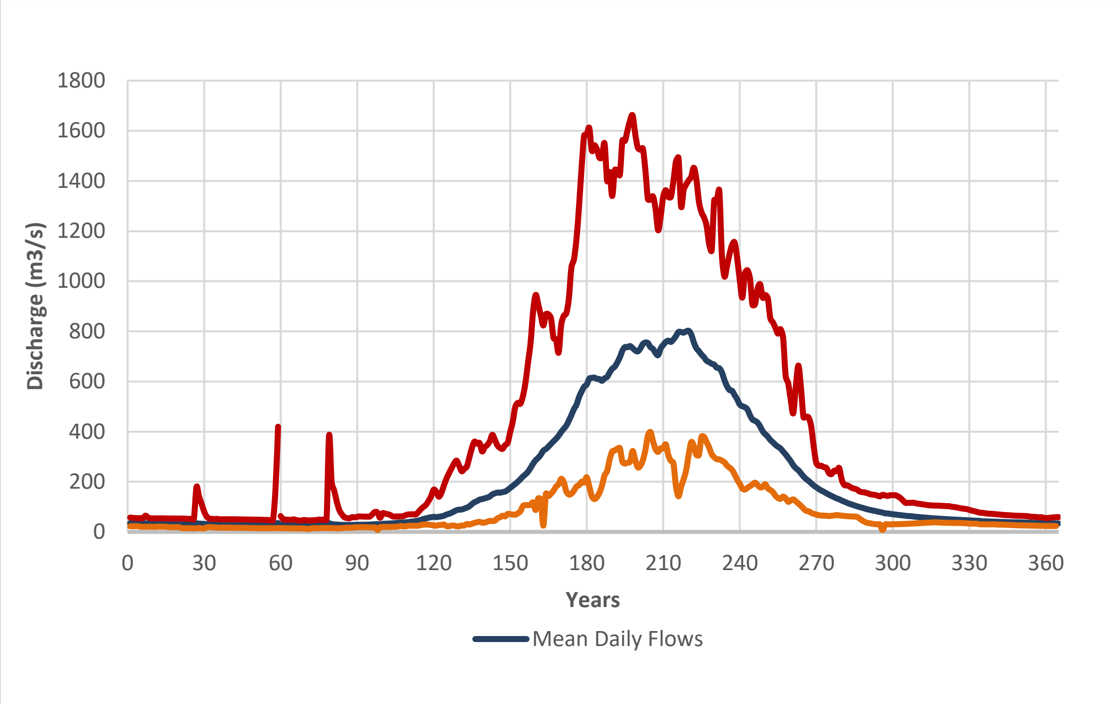


Figure 8.4: Mean, Maximum & Minimum Daily Flows

8.2.1 Flow Duration Curve (FDC)

Flow Duration Curve (FDC) for Attabad Lake HPP as plotted on the basis of almost 51 years (1966-2016) is shown here in Figure 8.5. Flow duration curve results are shown in Table 8.1. From FDC it is evident that design discharge of 60 m^3/s is available for almost 50% of the time. Therefore, design discharge of 60 m^3/s is justified for the generation of 54 MW power.

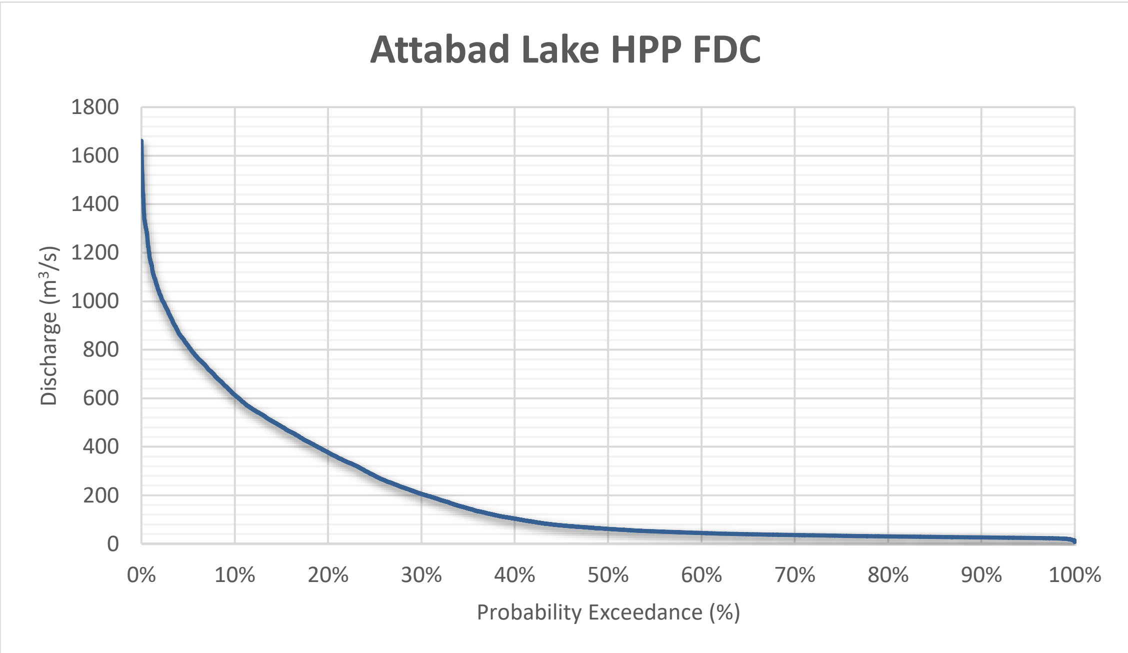


Figure 8.5 Attabad Lake Flow Duration Curve (FDC)