REMIT HYDRO LIMITED
GHUNSA KHOLA HYDROELECTRIC PROJECT (GKHEP)
(77.5 MW)

BIDDING DOCUMENT
FOR
CONSTRUCTION
OF
LOT NO. 1 – CIVIL AND HYDROMECHANICAL WORKS

(Under EPC Contract)

INTERNATIONAL COMPETITIVE BIDDING (ICB)

[Single Stage Two Envelope Bidding Procedure]

VOLUME 4 of 5
Basic Project Report

January 2026

Invitation for Bids No.:	**GKHEP/ICB-081/82/EPC-1**
Employer:	**Remit Hydro Limited**
Country:	**Nepal**

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PREFACE

This volume is part of the Tender Documents for Lot 1 – Civil and Hydro-mechanical Works of Ghunsa Khola Hydroelectric Project under EPC contract. The complete set of Tender Documents consists of the following volumes.

Volume 1	Bidding Procedures Section I - Instructions to Bidders (ITB) Section II - Bid Data Sheet (BDS) Section III - Evaluation and Qualification Criteria (EQC) Section IV - Bidding Forms (BDF) Section V - Eligible Countries
Volume 2	EMPLOYER'S REQUIREMENTS Section I – Scope of Works Section II – General Technical Requirements Section III – Standard Technical Requirements Section IV – Particular Technical Requirements Section V – Activity Schedule (ACS)
Volume 3	Conditions of Contract Section I - General Conditions of Contract (GCC) Section II - Special Conditions of Contract (SCC) Section III - Annexures Section IV - Contract Forms (COF)
Volume 4	**Basic Project Report** **Part A: General Information** **1 – Introduction** **2 – Key Features of the project** **3 – Power and Energy Studies** **4 – Salient Features of Employer's Concept Design** **Part B: Project Description** **5 – Topographic Survey and mapping** **6 – Hydrology and Meteorology** **7 – Sedimentology** **8 – Geology and Geotechnics** **9 – Seismicity**
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Abbreviations

AC	Alternative Current
B	Breadth
Ch	Chainage
DC	Direct Current
EIA	Environmental Impact Assessment
EL.	Elevation
EM	Electromechanical
EMS	Environmental Management System
EOT	Electrical Overhead Travelling
EPC	Engineering, Procurement, and Construction
FIDIC	International Federation of Consulting Engineers
FS	Factor of Safety
FSL	Full Supply Level
GCC	General Conditions of Contract
GIS	Gas Insulated Substation
GoN	Government of Nepal
GWh	Gigawatt hours
H	Height
HM	Hydromechanical
HOC	Headworks Operation Center
HRT	Headrace Tunnel
HV	High Voltage
HVAC	Heating Ventilation and Air Conditioning
Hz	Hertz
ICB	International Competitive Bidding
ICI	Imperial Chemical Industries
ILO	International Labor Organization
JSA	Job Safety Analyze
kg	kilogram
km	Kilometer
kV	Kilovolt
LCC	Local Control Cabinet
LED	light Emitting Diode
LV	Low Voltage
LV	Low Voltage
m	meter
m^3/s	Public Procurement Regulations
masl	meter above sea level
MDDL	Minimum Draw Down Level
MDE	Maximum Design Earthquake
MV	Megavolt
MVA	Megavolt Ampere
MW	Megawatt
NDT	Non-Destructive Test
NEC	NEA Engineering Company
OPGW	Optical Fiber Composite Overhead Ground Wire or Optical Ground Wire
OSH	Occupational Safety and Heath
PCC	Particular Condition of Contract
PGA	Peak Ground Acceleration
PMF	Probable Maximum Flood

PPE	Personnel Protective Equipment
QAP	Quality Assurance Program
RCC	Roller Compacted Concrete
SCADA	Supervisory Control and Data Acquisition
SF_6	sulfur hexafluoride
TDS	Total Dissolved Solid
W	Width
W/C	Water Cement Ratio

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Abbreviations & Acronyms

amsl	Above Mean Sea Level
CITES	Convention on International Trade in Endangered Species
FSL	Full Supply Level
GKHEP	Ghunsa Khola Hydroelectric Project
GoN	Government of Nepal
INPS	Integrated Nepal Power System
KCA	Kanchenjunga Conservation Area
KCAMC	Kanchenjunga Conservation Area Management Committee
RHL	Remit Hydro Limited
RM	Rural Municipality
RoR	Run-of-River
TWL	Tailrace Water Level

Units and Symbols

%	Percent
ºC	degree Celsius
GWh	Gigawatt hour
km	kilometer
km^2	square kilometer
kV	kilo Volt
m	meter
mm	millimeter
MW	Mega Watt
s	second

1. INTRODUCTION

Ghunsa Khola Hydroelectric Project (GKHEP) is proposed on Ghunsa Khola in Taplejung District, Province No. 1, Nepal (Figure 1). The project is being developed by Remit Hydro Limited (RHL), a company promoted by Nepal Electricity Authority (NEA), Hydroelectricity Investment and Development Company Limited (HIDCL) and Vidyut Utpadan Company Limited (VUCL) under the license for production of electricity issued by the Department of Electricity Development, Ministry of Energy, Water Resources and Irrigation, Government of Nepal (GoN).

Figure 1: Project location

2. PROJECT SETTINGS

Location

The GKHEP project area lies in the northeastern part of Nepal, between latitudes 27°31'46" and 27°34'00" N and longitudes 87°48'20" and 87°51'18" E. The project area is located in Phaktanglung Rural Municipality (RM) of Taplejung District. The project's diversion structure spans between Ward No. 5 of the RM on the left riverbank and Ward No. 6 on the right. All other components of the project are located on the right riverbank in Ward No. 6 of RM.

Access

The project area can be accessed through two routes on the national highway network. These routes are shown in Figure 2 and in *Volume V, Employer's Concept Drawings (Dwg No. GK-PA-02)* .

The project site is accessible via two principal routes from the national highway network.

Access Route-I:

This route originates at Charali, a town situated approximately 11 km north of Bhadrapur where the East-West Highway (HO-1) intersects with the Mechi Highway (HO-7). From Charali, the route follows a 226 km stretch of HO-7 heading northward to Phungling, the administrative headquarter of Taplejung District. Along this route, the towns of Budhabare, Ilam and Phidim are traversed. From Phungling, the initial point of the project area located at the confluence of Ghunsa Khola and Tamor River can be accessed via an approximately 59 km section of the North-South Olangchung Gola Road, passing through Ghatte Khola and Ranipul. Alternatively, the same initial point can be reached from Phungling via a route passing through Bahanande and Tawa, which includes the settlements of Handrung, Mitlung and Siwa along the way.

Access Route-II:

The alternative route to the project area begins at Itahari, a town located about 25 km north of Biratnagar, at the junction of HO-1 and the Koshi Highway (HO-8). From Itahari, a 53 km paved section of HO-8 extends northward to Mulghat. From Mulghat, 84 km segment of the Mulghat-Panchthar Road leads northeast and eventually intersects HO-7 north of Phidim. From this point, the project site can be reached via Phungling following HO-7 and subsequently the North-South Olangchung Gola Road via Ranipul.

The Contractor shall be solely responsible for conducting a detailed assessment of the load-bearing capacity and structural adequacy of all roads, bridges and related infrastructure along the proposed access routes to the project site. This assessment shall be carried out in coordination with the relevant government authorities.

The Employer shall bear no responsibility or liability for any damage, degradation or structural deficiency of roads or bridges that lie outside the designated boundaries of the project area. The Contractor shall, at his own expense, undertake any required upgrading, strengthening or rectification works necessary to facilitate the safe and efficient transportation of construction equipment, materials and heavy machinery to the project site if required and substantiated after detailed assessment of the routes. Such work shall be carried out in compliance with applicable laws and without causing undue disruption to public infrastructure.

During the 2025 flood event, the Bailey Bridge over the Tamor River on the North–South Highway, located few kilometers downstream of the Ghunsa–Tamor confluence was washed away. The loss of bridge caused a temporary interruption of road access for Lelep, Sekathum and other communities that rely on this section of the highway. The Department of Roads has allocated funds for the reconstruction of a Bailey Bridge at this location to restore and maintain uninterrupted access on the North–South Highway. Reconstruction works are currently underway and Bailey Bridge construction will be completed in the near term.

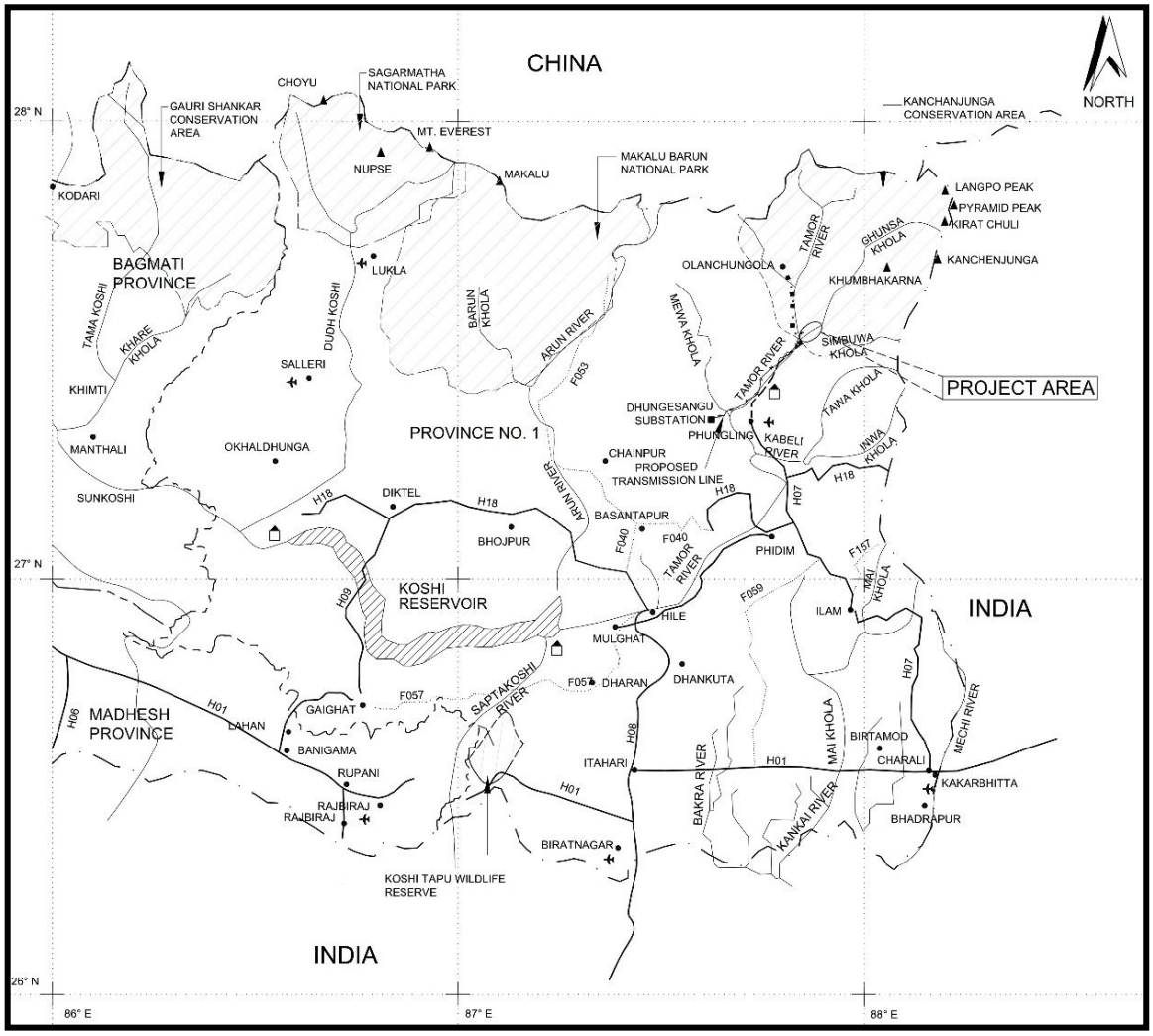

Figure 2: Access routes to project site

The nearest airport to the project area is at Suketar located about 8.5 km east of Phungling. Flights to this airport are scheduled from Kathmandu and Biratnagar; however, they are irregular and unreliable, mainly

due to weather conditions. Other major airports near the HO-1 access points to the project area are located at Bhadrapur and Biratnagar.

Indo-Nepal border points nearest to the project area are located at Kakarbhitta on the eastern border and at Biratnagar on the southern border. The distance from Kakarbhitta to the project area via Charali is 296 km while that from Biratnagar via Itahari is 308 km.

Topography

The GKHEP project area lies in the Himalayan region of Nepal, with elevations ranging from 1,500 m near the powerhouse area to 1,950 m in the headworks area. Its topography is characterized by steep rocky terrain, sharp-crested ridges, fairly steep to mild slope hills and deep and narrow river valleys. Forests with a few patches of cultivated terraces and scattered settlements are present along the river in the project area.

Climate

The climate of the project area varies from temperate to alpine. Most areas along the hill mid-slopes are warm, but the upper slopes are cool. May, June and July are the hottest months of the year. The average maximum temperature of the project district ranges from 16.8 °C to 26.1 °C, while its average minimum varies between 3.9 °C and 18.3 °C. The average annual rainfall in the Ghunsa catchment is about 1,818 mm, of which 75 to 80% occurs during the monsoon season (June to September).

Hydrology

Ghunsa Khola is a glacier-fed perennial river that is supported by monsoon rains, springs and snowmelt. About 80% of the annual flows in the river occurs during the monsoon season (June to September), while the remaining 20% occurs during the non-monsoon months, i.e., between October and May.

A major part of the Ghunsa Khola basin lies above 5,000 m and is characterized by permanent snow cover and glacial lakes. Portions of the basin lying between 3,000 m and 5,000 m witness occasional snowfall, while its areas below 3,000 m are affected by the monsoon.

Geology

Geologically, the GKHEP project area lies within the Higher Himalayan Zone of Nepal. It is located about 25 km north of the Main Central Thrust that is exposed along Tamor River.

The dominant rock type of the project area is gneiss; however, marble and quartzite are also observed in varying quantities. Colluviums are developed in the headworks and powerhouse areas, while alluviums are developed along the foothills close to these areas. Residual soils are generally found in small amounts at a few locations.

Forests and Vegetation

The project area lies within the Kanchenjunga Conservation Area (KCA) which is managed by Kanchenjunga Conservation Area Management Committee (KCAMC). It has various types of forests and vegetation with low to moderate crown coverage. The forests in the area are managed by different forest user committees formed under the management of KCAMC.

Wildlife

The project area is reported to have diverse species of mammals, birds, reptiles and amphibians; however, the occurrence of these species in, and the frequency of their visit to, the core project area is rare. In addition, the project area is reported to have multiple classes of phytoplankton, multiple orders of Zooplanktons, plenty of species of aquatic insects and few species of fish.

Due to its steep exposed rocks, sparse vegetation, settlements and human activities, the core project area does not serve as a good habitat for important and large mammals. A unique presence of high waterfall in the river downstream of the headworks area obstruct the upstream migration of fish.

Multiple mammal and bird species reported in the project area are listed under the Convention for International Trade of Endangered Species (CITES) categories. Likewise, the leopard cat and crimson-horned pheasant are protected under the National Parks and Wildlife Conservation Act 1973.

Demography and Ethnicity

The project area is scarcely populated. Different castes/ethnic groups live in the area. Limbu is the major ethnic group in the area, followed by Sherpa, Rai and Gurung. Chhetri, Brahmin, Dalit, Tamang and Newar are the other ethnic groups present in the area.

Language

The major languages spoken in the project area are Limbu, Sherpa, Nepali, Bantawa (Rai), Gurung and Tamang. Limbu is the dominant spoken language in the area, followed by Sherpa. however, most people residing in the area can communicate in the Nepali language.

Culture and Religion

Because of its ethnic and population diversity, the project area is home to a variety of cultures and religions. Traditionally, the Limbu culture, with its own social structure, is predominant in many places. However, several areas are inhabited by different ethnic groups with mixed cultures. People of all castes and/or religions respect each other's culture, values and norms.

Literacy and Education

The literacy rate in the project area is below the national average. However, this rate varies widely within the ethnic groups and between residents of different localities. Apart from formal education, residents have access to informal education programs like child, adult and female education programs that are conducted by different agencies on a limited scale every year.

Economy

Agriculture is the main occupation of the project area residents. A majority of them earn their livelihood from agriculture, which is largely of subsistence nature. The major cereal crops of the area are maize, millet and wheat, and the major cash crop is cardamom and potato. Other occupations practiced by the locals are animal husbandry, pottering, business, labor and portering.

Major Settlements

There are several scattered settlements in the project area, the major settlements in the area include Sekathum, Japantar, Ghaiyabari, Lawagua, Jongim and Amjilosa. Of these, Amjilosa is located near the headworks site while Sekathum and Japantar lie near the powerhouse site. Ghaiyabari, Lawagua and Jongim fall along the headrace tunnel alignment.

Major Market Places

Lelep and Tapethok are the nearest marketplaces to the project area. These markets serve the daily needs of the local people and play an important role in supporting their livelihood. A large market is located at Phungling, the district headquarters. Several hotels and lodges are present along the trekking routes in the project area.

Infrastructure

The project area has different types of infrastructures and facilities related to education, health, security, post, agriculture and veterinary services, etc. It has access to mobile telephone however the internet services isn't reliable presently. The area is not connected to the national power grid, but electric supply is available in some areas through micro hydropower stations and solar panels.

3. PROJECT SCHEME

GKHEP is being developed as a 77.5 MW run-of-river (RoR) scheme on Ghunsa Khola. The project's general arrangement is shown in Figure 3 and in *Volume V: Employer's Concept Drawing*

The major components GKHEP are arranged on the right bank of Ghunsa Khola. Its diversion weir is located at Lamatar, nearly 7 km upstream of the confluence of Ghunsa Khola with Tamor River. This structure, consisting of an ungated spillway weir and a gated undersluice, diverts the river water to a side intake located on the right bank. From the intake, a pair of connecting tunnels leads the water to two underground desilting chambers. At the end the desilting chambers, the desilted water enters the water conveyance system through a power intake. The conveyance system, comprising a pair of link tunnels off taking from the power intakes, a 4.208 km long headrace tunnel, a 56.4 m high, 6 m diameter restricted orifice surge shaft and a 854 m long, 2.5 m diameter pressure shaft consisting of three horizontal tunnels interconnected by two vertical shafts, transports the water to an underground powerhouse located in the ridges on the right bank of the river near Japantar, about 850 m upstream of the confluence of Ghunsa Khola and Tamor River. Water exiting the powerhouse rejoins Ghunsa Khola through a 660 m long tailrace tunnel.

4. MAJOR FEATURES OF PROJECT

The major features of the permanent structures of GKHEP envisaged in the Employer's design are listed in Table 1. Details of these structures are shown in *Volume V: Employer's Concept Drawing.*

5. POWER AND ENERGY

GKHEP is designed with an installed capacity of 77.5 MW generated through a gross head of 432 m and a design discharge of 21.28 m^3/s. Its annual average energy is estimated at 459.04 GWh, consisting of 138.529

GWh of six-month dry season energy generated between December and May and 320.511 GWh of six-month wet season energy generated between June to November, at an annual load factor of 67.6%.

6. POWER EVACUATION

The output from the GKHEP is proposed to be evacuated to the Integrated Nepal Power System (INPS) via a dedicated transmission infrastructure extending from the project's Takeoff Yard to the Margen Substation. For the downstream transmission of bulk power from the Tamor River basin, including GKHEP, RHL as an equity stakeholder, has participated in the establishment of a Special Purpose Vehicle (SPV), Tamor Dhungesanghu Power Transmission Company Limited. This SPV has been mandated to implement and operate a 220 kV double-circuit (D/C) transmission line from Margen Substation to Dhungesanghu Substation, forming a critical segment for power evacuation and facilitating grid integration of multiple hydropower projects in the Tamor Corridor.

7. CONSTRUCTION PLANNING AND SCHEDULING

The construction planning and scheduling for GKHEP was performed considering the entire scope of the civil, hydromechanical and electromechanical works required for the project. All requisite factors, such as volumes and nature of the works, effective work periods, material properties, the required equipment types and their characteristics and interfaces between civil, hydromechanical and electromechanical works were considered in the planning. The Employer's Concept for Project Execution and Construction Planning is provided to the Contractor for information purpose only herewith this Project Information. Based on these considerations, the entire project is planned to be completed in 48 months, i.e.,1460 days.

Table 1: Salient features of GKHEP

Feature	Particulars
River	Ghunsa Khola
Location	Phaktanglung RM, Taplejung Districts, Province No. 1, Nepal
Layout	Right bank of Ghunsa Khola
Catchment area	726.75 km^2
Type of scheme	Run-of-River (RoR)
Full supply level (FSL)	1,947 m amsl
Gross head	432 m
Rated head	420.74 m
Design discharge	21.28 m^3/s
Design flood discharge	880 m^3/s (1 in 100-year flood)
Diversion structure	32 m wide ungated spillway weir, two-bay undersluice with 3.5 m wide, 4.5 m high openings with radial gates, 52.5 m long horizontal stilling basin and vehicular bridge. Height from riverbed level: 6.75 m
River intake	Two side intakes, each 3.1 m wide and 3.1 m high, fitted with trash racks, trash rack cleaning machine, a maintenance gate and two service gates
Bed load sluices	Four sluices, two with 1 m and two with 0.8 m square openings, fitted with 800 mm steel pipes and gate valves, integrated in the intake structure
Gravel trap	Four traps, each with size varying between 0.8 m × 0.5 m at inlet and 0.8 m × 0.8 m at outlet and fitted with 800 mm steel pipes and gate valves, integrated in the intake structure
Connecting tunnels	Two tunnels, each 3.9 m high and 3.9 m wide, 127.8/ 101.54 m long
Desilting chambers	Two underground basins, each 62 m long,11.5 m wide and 14.65 m high, with continuous flushing through 223.62 m long, 3.9 m high, 3.9 m wide silt flushing tunnel fitted with four 1 m × 1 m silt flushing gates housed in a 13 m × 10 m × 10 m flushing gate chamber
Power intake	Two intakes, each 3.9 m high and 3.9 m wide with vertical lift gates operated from 6.5 m × 5.25 m × 8 m gate operating chambers accessed through a 162 m long, 4.5 m high, 4.5 m wide gate-cum-ventilation tunnel
Link tunnels	Two tunnels, each 80.8 m long, 3.9 m high and 3.9 m wide, connecting power intake with headrace tunnel
Headrace tunnel	4.208 km long, 3.6 m × 3.6 m D-shaped in shotcrete-lined sections and 3.2 m diameter horseshoe shaped in concrete-lined sections, with three construction Adits of lengths 159 m, 205 m, 318 m and 89 m

Feature	Particulars
Surge shaft	Open to sky, 55.6 m high and 6 m finished diameter, restricted orifice type
Valve chamber	10 m long, 13 m wide, 17 m high cavern with 3 m diameter butterfly valve
Pressure shaft	2.5 m finished diameter steel lined and 854m long
Unit penstock	Three 1.4 m diameter steel lined tunnels, 35 / 30 m lengths
Powerhouse	Underground cavern, 84 m long, 16 m wide and 36 m high
Transformer gallery	Underground cavern, 75.6 m long, 16 m wide and 10 m high GIS cavern,15.8 m in length, 13 m in width and 14.95 m in height.
Bus duct tunnels	Three, 4 m high, 4 m wide and 32 m long tunnels
Main access tunnel	312 m long, 7m finished dia
Cable-cum-ventilation tunnel	186.5 m long, 4.5 m high and 4.5 m wide
Tailrace tunnel	660 m long, 4.5 m high and 4.5 m wide with 28 m long outfall structure
Installed capacity	77.5 MW
Turbine	Three vertical shaft Pelton turbine, 500 rpm
Transmission line	33 km long, 220 kV double circuit transmission line to NEA's Dhungesanghu substation
Internal access road	13.62 km
Construction period	4 years

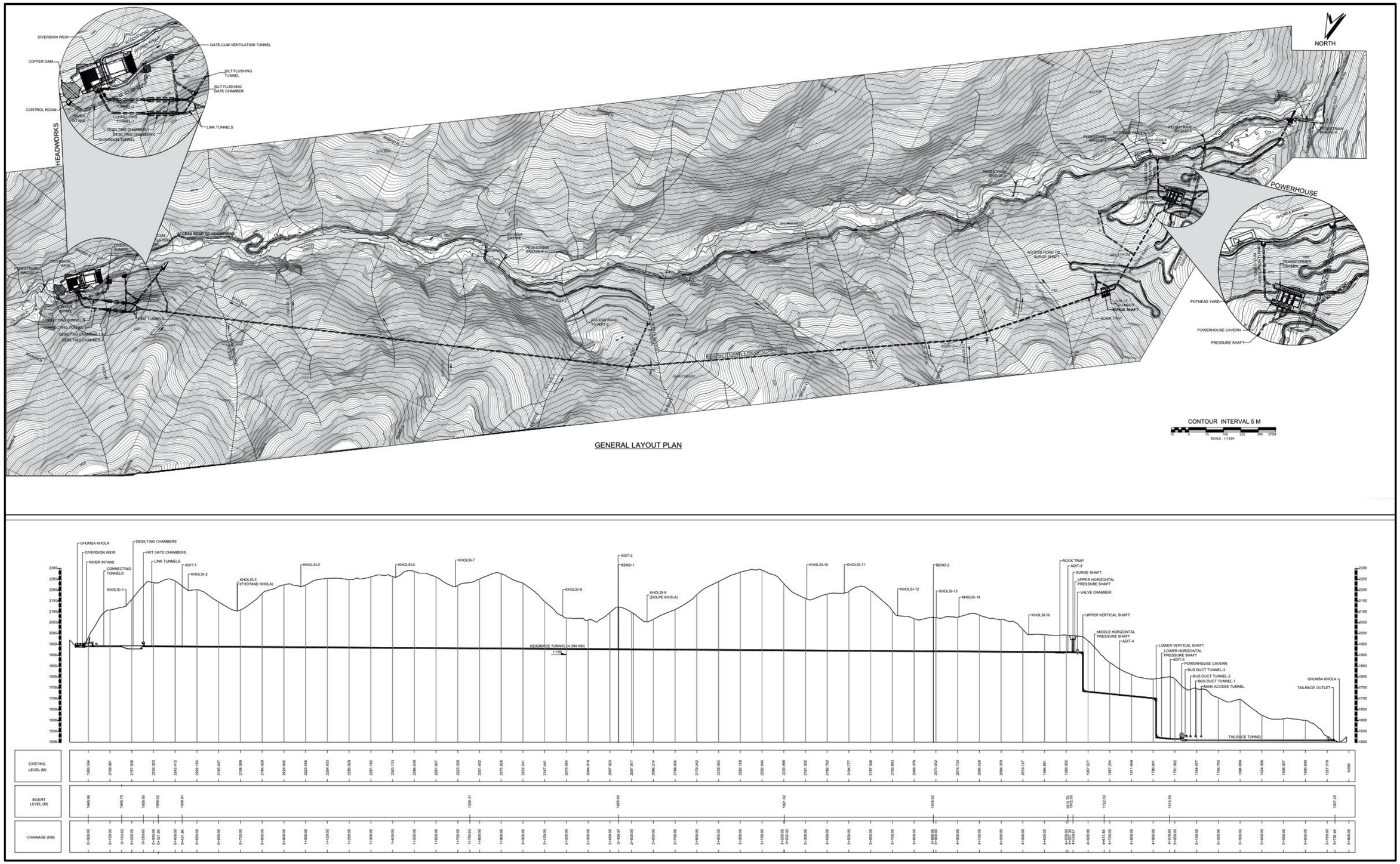

Figure 3: General arrangement of GKHEP

VOLUME IV: BASIC PROJECT INFORMATION

SECTION II: GENERAL INFORMATION

SECTION A: TOPOGRAPHICAL SURVEY

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Abbreviations & Acronyms

amsl	Above Mean Sea Level
DGPS	Differential Global Positioning System
GKHEP	Ghunsa Khola Hydroelectric Project
GNSS	Global Navigation Satellite System
GoN	Government of Nepal
RHL	Remit Hydro Limited

Units and Symbols

ha	hectare
km	kilometer
km^2	square kilometer
m	meter
m^2	square meter
mm	millimeter

1. INTRODUCTION

A comprehensive topographic survey of the project area was executed in early 2019, encompassing a sufficiently extensive stretch from the confluence of the Ghunsa and Tamor rivers to the upstream extent of the proposed headworks. The survey was undertaken with a spatial density adequate for the generation of precise topographic maps, longitudinal and transverse profiles and cross-sections necessary for the detailed engineering design of the project infrastructures. Complementing this, a high-resolution LiDAR (Light Detection and Ranging) survey was carried out in February 2025 to delineate baseline geospatial and terrain conditions in advance of the commencement of main civil works. It is imperative to underscore that the accuracy and applicability of all geospatial datasets and outputs generated from these surveys shall be subject to verification and validation by the successful EPC Contractor as part of their design obligations.

2. METHODOLOGY

Topographical surveys of the project area commenced with desk studies in which the following available information on the project-area topography were collected and reviewed:

a. 1:50,000 scale map of the project area with a contour interval of 40 m (Map No. 2787 08) published by the Topographic Survey Branch, Survey Department, GoN.

b. Details of trigonometric stations on the national geodetic network established near the project area by the Geodetic Survey Branch, Survey Department, GoN.

c. Digital data for topographic sheets relevant to the project area, published by the National Geographic Information Infrastructure Project, Survey Department, GoN.

Following the review, a reconnaissance visit of the project site was conducted in November 2018. During the site visit, the areas to be surveyed were delineated and the areas requiring additional topographic detailing for design purposes were marked. The resulting survey boundaries were marked on the 1:50,000 topographical map to guide the field surveys.

Topographical field surveys of the project area were performed between December 29, 2018, and February 22, 2019. These surveys consisted of control surveys, tacheometric detailing of the headworks, surge shaft and powerhouse areas, strip survey of the tunnel alignment, river profile survey, river cross-sectional surveys and road alignment surveys.

3. CONTROL SURVEY

3.1 Establishment of DGPS Control Points

Control surveys were conducted to establish the basic framework for topographic surveys in the project area. For this purpose, five differential global positioning system (DGPS) control stations were established in the project area using Global Navigation Satellite System (GNSS) technology[1]. Details of these control stations are listed in Table 1.

Table 1: DGPS control stations established in project area

SN	Station	Location	Coordinates (m)			Remarks
			Northing	Easting	Elevation	
1	GK1	Sekathum	3046447.170	579758.631	1576.709	Permanent point (200 × 200 × 500 mm pillar)
2	GK2	Japantar	3046721.402	580110.057	1552.134	
3	GK3	Lai Gyap Bheer	3046871.672	580081.383	1702.028	
4	GK4	Phedi, headwork area	3049158.663	584099.818	1973.549	
5	GK5		3049256.536	584160.254	1976.633	

3.2 Establishment of Permanent Control Points

Permanent control points were established in the project area by running a closed traverse from the DGPS control points established in the powerhouse area to those in the headworks area (Table 1). In this traverse, the planimetric positions and elevations of the permanent control points were respectively established through the horizontal and vertical control survey procedures.

[1] Fourth-order trigonometric stations on the national geodetic network established by the Survey Department could not be traced on the ground. Further, the elevations of the trigonometric stations were not available.

Horizontal Control Survey

Horizontal control survey was conducted using a high precision total station with least count of 1". Four sets of horizontal readings (face left and face right) were observed to minimize collimation error. The readings were observed at different zeros to minimize the graduation errors. All traverse angles were measured on both left and right faces, and observations for directional controls were made and checked at regular intervals.

Vertical Control Survey

Vertical control survey was conducted through triangulation leveling using a 1" precision total station. Four sets of vertical readings (face left and face right) were observed to minimize index error. The readings were observed at different sets to minimize graduation errors.

Details of Permanent Control Points

Using the procedures described above, seven permanent control points were established in the project area, including two in the powerhouse area, three in the headworks area and two in between (Table 2). The locations of the control points are shown on Description Cards (D-Cards) and on the traverse chart in *Volume IV, Part 2: Topographic Maps*.

Table 2: Coordinates of permanent control points

SN	Station	Location	Coordinates (m)			Remarks
			Northing	Easting	Elevation	
1	TP1	Lai Gyap Phedi	3046844.244	580167.952	1630.708	Permanent points on boulder
2	TP2	Vhedi chowk	3047537.052	581037.711	1754.235	
3	TP3	Jongim	3048099.609	581938.565	1757.370	
4	TP4	Jaubari	3048368.889	582793.149	1837.888	
5	TP5	Lamatar	3048664.548	583477.072	1917.174	
6	TP6	Lamatar	3048754.864	583632.177	1924.419	
7	TP7	Amjilosa Phedi	3049305.876	584136.797	2001.430	

Accuracy

The closed traverse for establishing the horizontal control points was 45029.488 m long, and it was performed with a linear accuracy of 1:47,050. This accuracy is within permissible limits. Likewise, the accuracy for the vertical control points was within the limiting value of $12\sqrt{k}$, where k is the traverse length in km.

Monumentation

Permanent monuments were set up to preserve the planimetric positions and elevations of the control points. The DGPS control points were monumented with 200×200×500 mm concrete pillars with embedded steel bars, and their names were marked on the top surface of the pillars with enamel paint. Likewise, the permanent control points established on the closed traverse were monumented on large, permanent boulders with steel nails and enamel point.

4. TOPOGRAPHIC SURVEY OF PROJECT COMPONENT SITES

Tacheometric procedures were used to map the topography of the headworks, portal, surge shaft, powerhouse and tailrace areas. For this purpose, horizontal and vertical distance measurements were made with a total station through radial surveys either from the control points or from setup points established through traverses run with reference to the control points. During the survey, the locations of different field features were recorded using a coding system to facilitate data processing using mapping software.

The surveys were conducted in sufficient detail to permit preparation of topographic maps at the required scales. A total of 33.19 ha was surveyed in the headworks area, 83.41 ha in the powerhouse and surge shaft areas and 20.15 ha in the tailrace area.

5. STRIP SURVEY ALONG TUNNEL ALIGNMENT

A strip survey of the headrace tunnel alignment, covering a corridor of at least 130 m from the tunnel center line, was conducted. Detailed sections of cross drainages lying across the alignment were also surveyed. The survey was 4.08 km long and covered an area of 53.04 ha.

6. LONGITUDINAL PROFILE OF RIVER

The longitudinal profile of Ghunsa Khola was surveyed from about 690 m upstream of the weir axis to about 460 m downstream of the tailrace axis. The total length of the profile was about 6.9 km.

7. RIVER CROSS-SECTION SURVEYS

River cross-sectional surveys were performed along the weir axis and at 50 m intervals from 700 m upstream to 500 m downstream of the axis. Likewise, these surveys were carried out at the tailrace axis and from 540 m upstream to 510 m downstream of it, at similar intervals. Additional cross-sections were surveyed between the headworks and tailrace sites at 50 m intervals to acquire data for flood modeling. The surveys covered a width of more than 20 m on both sides of the river centerline.

8. ROAD ALIGNMENT SURVEY

Road alignment survey was conducted from the junction of the North-South Olanchungola road and the road to Sekathum village up to the headworks area through the powerhouse area. The survey was generally carried out along the existing trail that connects the confluence of Ghunsa Khola and Tamor Khola to Phedi at the headworks site, covering a corridor of 20 m on either side of the trail centerline. Detailed surveys of cross drains and other important features along the alignment were also performed.

9. MAP PREPARATION

After completion of the field surveys, the field data were compiled and reduced. Using these data and digital data where needed to complement the reduced field data, the following maps, profiles and cross-sections were prepared:

- Overall traverse chart at a scale of 1:15,000.
- General plan of the project area at a 1:7,500 scale with a contour interval of 5 m.
- Topographic maps of the headworks, desilting basin, surge shaft, pressure shaft, powerhouse and tailrace areas at a scale of 1:500 with a contour interval of 1 m.
- Topographic map of the headrace tunnel alignment at a scale of 1:5000 with a contour interval of 5 m.
- River profiles in the headworks area at horizontal and vertical scales of 1:500 and 1:250, respectively.
- River profiles in the tailrace area at horizontal and vertical scales of 1:500 and 1:100, respectively.
- Longitudinal profile of Ghunsa Khola at horizontal and vertical scales of 1:10,000 and 1:5,000, respectively.
- Cross-sections of Ghunsa Khola at the headworks and tailrace sites at horizontal and vertical scales of 1:1000 and 1:1000, respectively.

SECTION B: HYDROLOGY AND SEDIMENTOLOGY

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Abbreviations & Acronyms

amsl	Above Mean Sea Level
DHM	Department of Hydrology and Meteorology
FDC	Flow Duration Curve
FSL	Full Supply Level
GKHEP	Ghunsa Khola Hydroelectric Project
GLOF	Glacial Lake Outburst Flood
GoN	Government of Nepal
HFL	High Flood Level
HSC	Hydro-meteorologically Similar Catchment
KCA	Kanchenjunga Conservation Area
MWL	Maximum Water Level
RHL	Remit Hydro Limited
WEC	Water and Energy Commission

Units and Symbols

%	Percent
km	kilometer
km^2	square kilometer
m	meter
m^2	square meter
mm	millimeter
s	second

1. INTRODUCTION

Detailed hydrological and sedimentological studies were performed on Ghunsa Khola to establish the hydro-meteorological and watershed characteristics of the river basin, to assess its long-term hydrology, flood hydrology and low-flow hydrology at the GKHEP intake and powerhouse sites and to estimate the long-term sediment yield and concentration of Ghunsa Khola at the GKHEP intake. The results of these studies are summarized below.

2. HYDRO-METEOROLOGICAL CHARACTERISTICS OF GHUNSA KHOLA

Ghunsa Khola is a major tributary of Tamor River, which in turn is one of the major tributaries of the Sapta Koshi River that drains eastern Nepal. The river's headwater originates from the base of Mt. Kanchanjungha, at an elevation of about 8,586 m amsl. Several high peaks and glaciers are present within the river watershed. More than 60% of the river's catchment lies above an elevation of 5,000 m with permanent snow cover.

The hydrology of the Ghunsa Khola basin is governed by snowmelt, glacial flows, annual rainfall, precipitation (i.e., rain or snow) and their variability in time and space and the response of the basin to them. The basin response depends on a host of factors, the dominant ones being the basin shape and size, physiography, topography, slope, land use, drainage pattern, etc.

The long-term average annual and monsoon rainfall of the Ghunsa Khola basin, based on the nearest rainfall station (Station No. 1403 at Lungthung) established at the Department of Hydrology and Meteorology (DHM), GoN, are 2,257 and 1,761 mm, respectively. These figures stand at 1,818 mm and 1,363 mm, respectively, when other surrounding rainfall stations (Station Nos. 1401 and 1402) are also considered.

The primary sources of flow in Ghunsa Khola are glaciers present on both banks of the river in its upper catchment. Even during the dry months of February to May, the river has runoffs of 8 to 9 m^3/s that are primarily contributed by snowmelt. Hence, Ghunsa Khola is perennial in nature.

3. WATERSHED CHARACTERISTICS

The Ghunsa Khola watershed is characterized by rugged topography with high peaks, sharp-crested ridges, narrow gorges and mountains with fairly steep to mild, medium slopes. The river valley is generally narrow, with some small stretches of terraces along the river. The slopes in the upper parts of the catchment are steep and barren; however, in the lower parts, they are covered with forests along with a few patches of cultivated land and scattered settlements.

The Ghunsa Khola course is deeply incised and steep. Several medium and high falls are present along its course, creating turbulent flows in the river. Most of the river's tributaries meet it at right angles, and they are moderately to deeply incised and have sub-parallel drainage patterns.

The catchment area of the Ghunsa basin at the GKHEP intake is estimated at 726.75 sq. km. A large part of the catchment is located on the northeast side of the river course. However, its upper part extends northward in the form of permanent snow cover and glacial lakes. Elevations of the catchment vary from 8,586 m amsl (Mt. Kanchanjungha) to 1,950 m amsl at the GKHEP intake.

A major part of the Ghunsa Khola catchment area at the intake, about 446.75 sq. km, lies above 5,000 m and is permanently under snow cover. About 255.8 sq. km of the area lies between 3,000 and 5,000 m, and only 24.2 sq. km is located below 3,000 m.

4. LONG-TERM HYDROLOGY

As an adequately long period of flow data is not available for the river, the long-term flows for Ghunsa Khola at the GKHEP intake site were estimated through transposition of the following data:

1. Long-term flow data of seven regional DHM stations, viz. Station Nos. 600.1 on Arun River at Uwagaun, 602 on Sabhay Khola at Tumlingtar, 602.5 on Hinwa Khola at Pipaltar, 604.5 on Arun River at Turkighat, 606 on Arun River at Simle, 684 on Tamor River at Majhitar and 690 on Tamor River at Mulghat, using their long-term flow records till 2014.
2. Long-term flow data of DHM stations in two Hydrologically Similar Catchments (HSC), viz. the Balephi Khola and Likhu Khola catchments, having similar catchment areas and characteristics as those of the Ghunsa Khola catchment at the GKHEP intake.
3. Daily flow data of the DHM's Station No. 690 at Mulghat on Tamor River.

Based on a careful study of long-term flows derived from the above transpositions, the long-term mean monthly flows derived from the transposition of flows from DHM's Station No. 690 on Tamor River at Mulghat, which represents the mother catchment of Ghunsa Khola, were selected for the GKHEP intake.

The resulting mean monthly long-term flow series and the flow durations for different percentiles are listed in Table 1 and Table 2, respectively, and the corresponding hydrograph and flow duration curve are presented in Figure 1 and Figure 2, respectively.

Table 1: Mean monthly flows at GKHEP intake

Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Ave.
8.4	6.9	6.8	10.1	17.8	44.6	88.0	96.2	69.4	32.0	15.3	11.6	*33.9*

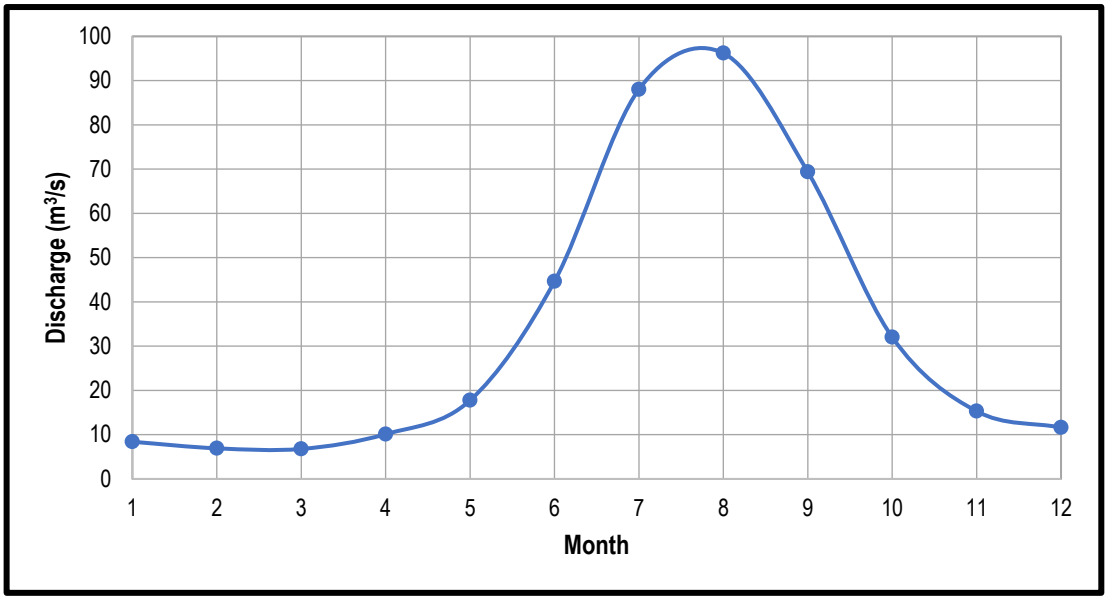

Figure 1: Recommended long-term monthly flows at GKHEP intake

Table 2: Flow duration at GKHEP intake without environmental release

% time	5	10	15	20	25	30	35	40	45	50
Q, m^3/s	97.25	92.47	82.24	72.55	57.46	46.12	31.99	24.69	19.01	15.82
% time	**55**	**60**	**65**	**70**	**75**	**80**	**85**	**90**	**95**	**99**
Q, m^3/s	13.70	12.45	11.53	10.11	8.56	7.87	7.30	6.95	6.59	6.40

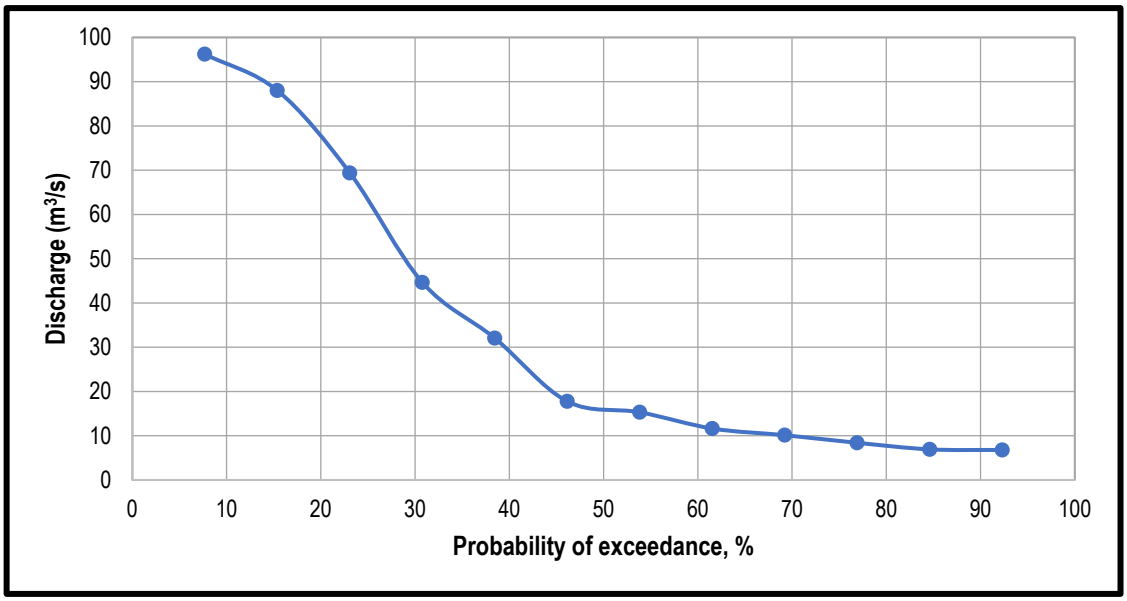

Figure 2: Recommended long-term monthly flow duration at GKHEP intake

4.1 Mean Monthly Design Flows

As the GKHEP project area lies within the KCA, an environmental release of 25% of the minimum flow is mandatory. Accordingly, the mean monthly design flows at the GKHEP intake were derived from the long-

term series in by deducting 25% of the minimum monthly flow (6.78 m^3/s in March, Table 1), i.e., 1.7 m^3/s, as environmental release. The resulting mean monthly design flow values for GKHEP are listed in Table 3.

Table 3: Mean monthly design flows after environmental release (m^3/s)

Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Ave.
6.7	5.22	5.08	8.42	16.0	42.94	86.3	94.5	67.68	30.3	13.6	9.93	32.23

5. FLOOD HYDROLOGY

Due to lack of stream gauging and rainfall stations in the Ghunsa Khola catchment, the floods for Ghunsa Khola at the GKHEP intake were estimated through the following approaches using flood data available till 2013:

1. Frequency analysis of transposed average floods of Tamor River recorded at DHM's Station No. 690 at Mulghat.
2. Intensity-based approach (Hourly Rainfall Intensity Method or the PCJ 1996 Method).

Floods of different frequencies at the GKHEP intake, estimated from these methods, are listed in Table 4. On comparing the fits of the various distributions to the flood data, floods from the Gumbel distribution were selected for the GKHEP intake.

Given limited local gauging, the Contractor during its own design shall broaden the hydrological database and design discreetly in consideration of the safety margins in hydrological analyses and headworks design so that the structure remains resilient under plausible extreme flood conditions.

Table 4: Estimated floods of different frequencies at GKHEP intake

Methods	Return periods, years						
	5	10	50	100	500	1000	10000
Frequency analysis of transposed flows							
Log Excel	501	621	900	1020	1299	1419	1818
Gumbel EV	*501*	*593*	*795*	*880*	*1076*	*1161*	*1442*
Log Normal	499	591	794	881	1088	1180	1501
Log Pearson III	496	595	829	936	1204	1330	1800
Intensity based (PCJ 1996)		329	777	936	-	-	-

5.1 Probable Maximum Flood

The probable maximum flood (PMF) at the GKHEP intake was estimated based on a general rule of thumb that the PMF is approximately twice the 10,000-years flood. Accordingly, for the 10,000-year flood of 1,442 m^3/s from Gumbel distribution (Table 4), the PMF was estimated at 2,884 m^3/s.

5.2 Selected Design Floods

The GKHEP diversion structure is a simple weir with limited impounding capability. Due to its inherently simple configuration and ease of operation, the structure is envisaged to conveniently pass high floods without severe damage, i.e., without damage that could severely impair the continued operation of the project. Furthermore, the consequences of failure of the weir in terms of loss of human life and property are minimal because of the remote location of the weir and the sparse settlements on its downstream.

Considering the above factors, a design flood frequency of 100 years was selected for the GKHEP intake. Accordingly, the design flood at this site was chosen as 880 m^3/s.

5.3 Diversion Flood at GKHEP Intake

Owing to severe climatic conditions in the project area during the winter (especially December and January), adequate time for construction of the GKHEP headworks during the dry season may not be available, and the construction may well run into the rainy season. As such, the 5-year return period instantaneous flood, equal to 501 m^3/s (Table 4), was selected as the construction diversion flood for the GKHEP intake.

5.4 Design Floods at GKHEP Powerhouse

The design flood for the GKHEP powerhouse was derived from the design flood at the intake using catchment area transformation. Accordingly, a 100-year design flood of 901 m^3/s was selected for the powerhouse.

5.5 Glacial Lake Outburst Floods

Although several glaciers are present in its upper reaches, Ghunsa Khola has no recorded history or evidence of Glacial Lake Outburst Floods (GLOF). However, the possibility of GLOF events in the future cannot be ruled out in view of the global warming phenomenon.

5.6 Water Surface Profiles and Rating Curves

Water surface profiles corresponding to floods of different return periods and rating curves for the headworks and powerhouse sites were derived using HEC-RAS, a computer program developed by the United States Army Corps of Engineers (USACE). The analyses were performed assuming supercritical conditions. Based on results of direct flow measurements, the water surface profiles at the headworks and the rating curve at the weir axis thus computed are shown in Figure 3 and Figure 4, respectively. Likewise, the water surface profiles in the tailrace area for different floods and the rating curve at the tailrace axis are shown in Figure 5 and Figure 6, respectively.

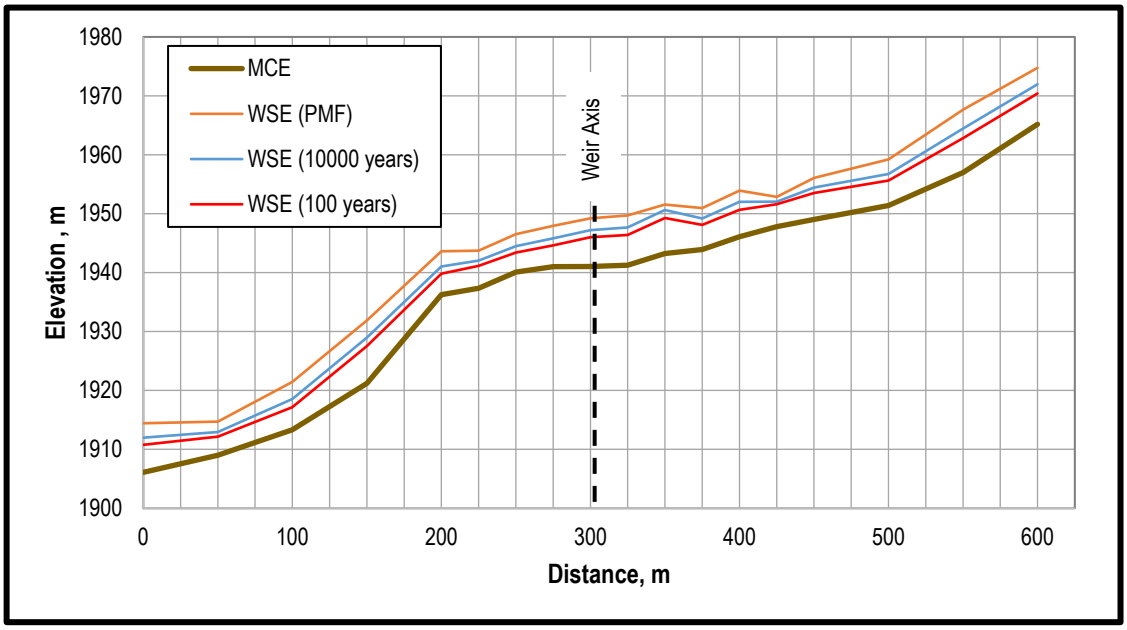

Figure 3: Water surface profile at weir axis for flood of different return periods

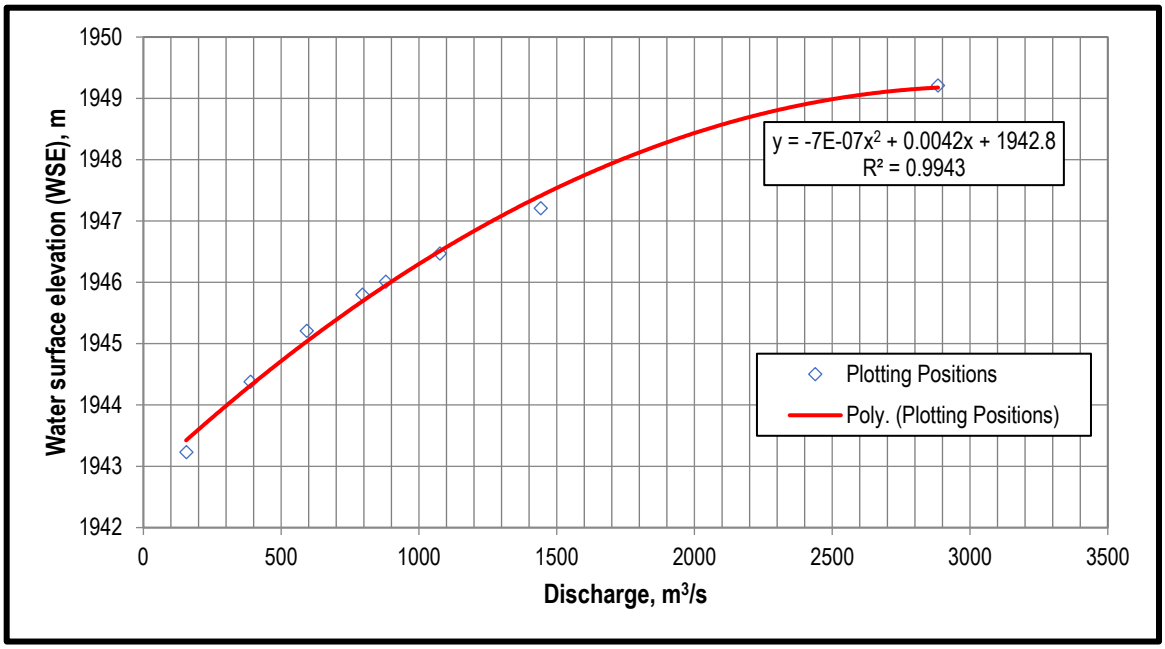

Figure 4: Rating curve at weir axis of GKHEP

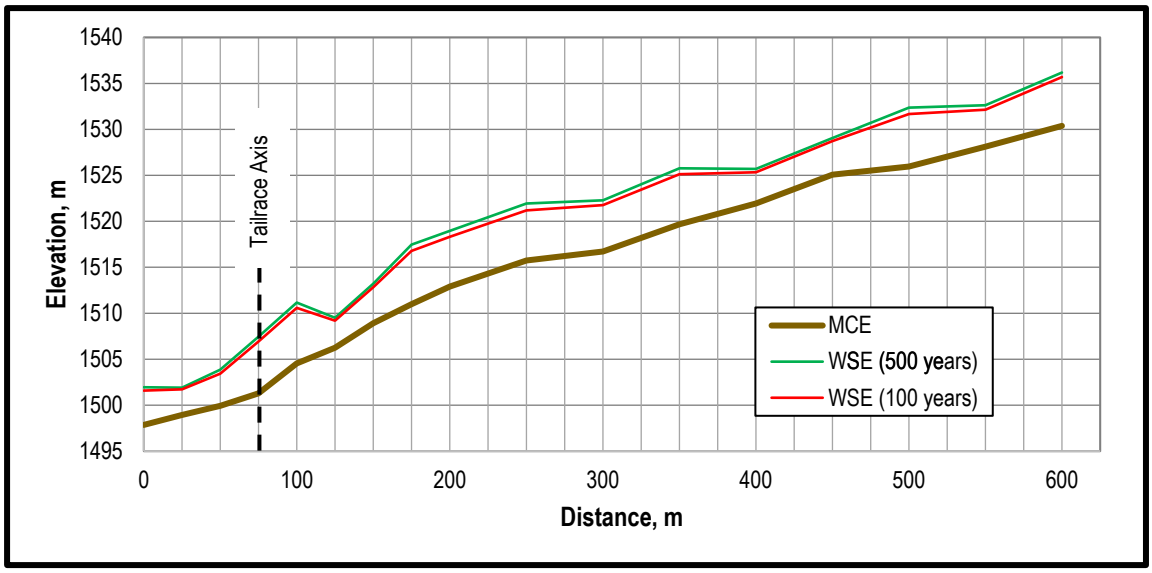

Figure 5: Water surface profile in tailrace area

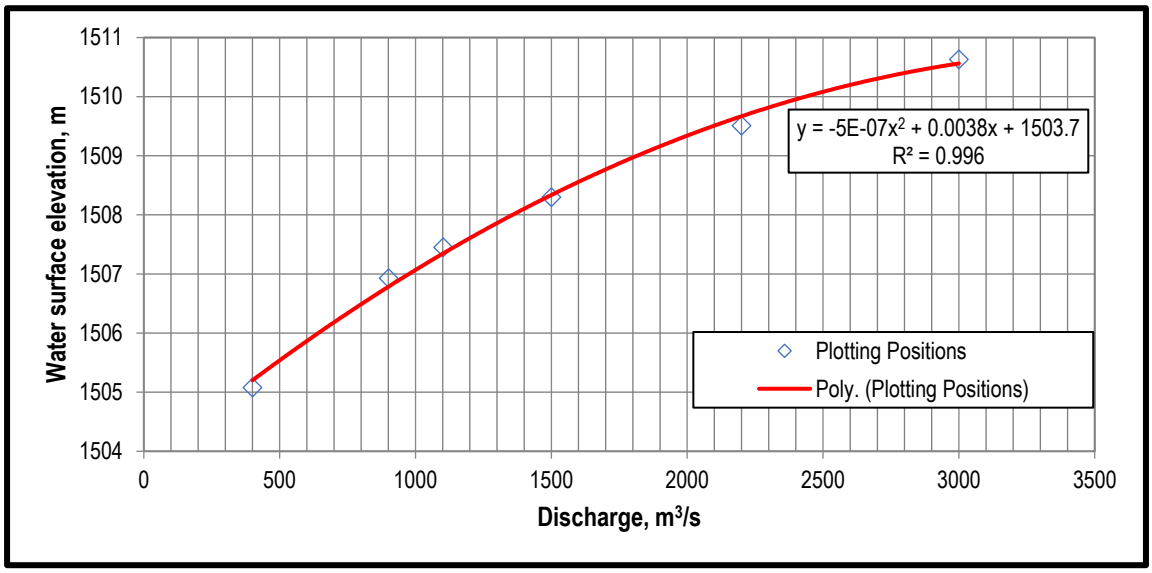

Figure 6: Rating curve at tailrace axis of GKHEP

6. LOW FLOW HYDROLOGY

Low flows at the GKHEP intake were estimated from the following approaches using data available till 2013:

1. Transposition of low flows of Tamor at Mulghat (St. No. 690).
2. WECS/DHM method.

Although the WECS/DHM results for low flows were slightly lower than those from the best fit to the transposed flows, the low flows from the best fit were selected for the intake considering the fact that these flows were based on low-flows of the mother catchment.

The design frequency for low flows is generally chosen considering draught conditions. Based on international practices and guidelines, the design low flow at the intake was selected as 2.18 m^3/s corresponding to the 20 years frequency.

7. SEDIMENTOLOGY

The sediment load of Ghunsa Khola is high because of its high gradients and alluvial banks, young geological formations along its course, deforestation, heavy and intense rainfall, etc. The bulk of the sediment transport in the river occurs during the monsoon period. Considerable variation in the sediment concentrations is generally witnessed during this period.

Owing to lack of availability of long-term sediment data, sediment studies on Ghunsa Khola were conducted based on regional and empirical approaches adopted in Nepal. Based on these studies, a sediment concentration of 3,000 to 3,500 ppm is recommended for design of the GKHEP intake and settling basin. The long-term average annual sediment yield from the Ghunsa catchment is estimated at 2.5 million T, and the expected bed load at the GKHEP intake may be adopted as 0.38 million T, equivalent to 15% of the average annual sediment yield (2.5 million T).

About 60% of suspended sediment comprises of particle size greater than 0.2 mm and will be removed in settling basin. Between 70 to 80% of sediments consist of quartz particles.

VOLUME IV : PROJECT INFORMATION

SECTION 3: GEOTECHNICAL INFOMATION & DATA

SECTION A: GEOTECHNICAL INFORMATION REPORT

DISCLAIMER
This Geotechnical Information Report (GIR) is the document that defines the subsurface physical conditions that are to be assumed to be encountered in the execution of the works through referenced statements for the foreseeable physical conditions of the ground between the Parties. The purpose of the GIR, is to provide the basis for establishing the ground-related risks in the design and construction for the execution of surface and underground works. The GIR presents and organizes the geological/ geotechnical information for use by the Owner, design team and construction contractor for the purpose of having a better understanding of the project and aid in preparation of bids to the Contractor. However, under no circumstances can this document be used for the purpose of preparing/ raising claims on account of variation in the geological/ geotechnical ground conditions as given in this document and actually encountered during the project construction and execution.

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Abbreviations & Acronyms

amsl	Above Mean Sea Level
ERT	Electrical Resistivity Tomography
GBG	Garnetiferous Banded Gneiss
GIR	Geotechnical Information Report
GDR	Geotechnical Data Report
GKHEP	Ghunsa Khola Hydroelectric Project
GQM	Quartz biotite gneiss, quartzite and marble
HFT	Himalayan Frontal Thrust
HT	Himalayan Thrust
MBT	Main Boundary Thrust
MCT	Main Central Thrust
MT	Mahabharat Thrust
MII	Modified Mercalli Intensity
NBCDP	National Building Code Development Project
NGI	Norwegian Geotechnical Institute
PGA	Peak Ground Acceleration
RBM	River Borne Material
RMR	Rock Mass Rating
RQD	Rock Quality Designation
STDS	South Tibetan Detachment System
USGS	United States Geological Survey

Units and Symbols

%	Percent
km	kilometer
km^2	square kilometer
l	Liter
Lu	Lugeon
m	meter
m^2	square meter
m^3	cubic meter
min	minute
mm	millimeter
s	second

1. INTRODUCTION

1.1 Purpose of Geotechnical Information Report

This Geotechnical Information Report (GIR) is issued as part of the Bid Documents for construction of GKHEP. It must be read in conjunction with the Geotechnical Data Report (GDR) and relevant drawings from *Volume: V Employers Concept Drawing*.

The primary objectives of the GIR are to:

- Provide a summary of the ground conditions, subsurface conditions, ground behavior and groundwater conditions in the project area in order to facilitate the understanding of the contractor's designer and the Owner's project management team regarding the geotechnical features and ground behavior to be addressed during bid preparation and during the design and construction of the surface and underground structures of the project.
- Establish anticipated geological and geotechnical conditions for the preparation of proposals, in bid submittals and in the design and the construction of the project structures.
- Assist in administering the provisions for differing site conditions contained in the Contract Documents by providing a basis for their evaluation.

1.2 Basis and Scope

The GIR is prepared based on a study of the regional and site geology of the project area and an evaluation and interpretation of the geological and geotechnical data and information obtained from various explorations and field tests carried out at the sites of different project components as well as laboratory tests carried out on samples obtained from the field.

The GIR describes the geological settings underground environment expected to be encountered during the execution of the underground works and other excavations for permanent works under LOT 1. It describes the ground conditions and basic surface and subsurface behavior. It should be noted that the surface and underground behaviors described in the GIR are based on assumptions regarding construction means and methods, and the actual behavior may differ if different means and methods are employed by the bidder.

Temporary structures, such as the diversion tunnel, upstream cofferdam and all treatment works for temporary slopes in overburden and rock, are not covered by the GIR.

1.3 Bidder's Obligations

The bidder must complete its own independent review and evaluation of the GIR, GDR and other geotechnical information and consider how its selected means, methods and designs will affect ground behavior and construction of project structures. It is important that the bidder understands the geotechnical aspects of the project, as these aspects represent a significant proportion of the project risk. If needed, the bidder should seek the opinion of specialist geological and geotechnical experts so as to fully understand the geological/ geotechnical issues related to the project. Failure to understand these issues will not be considered a reason for any additional compensation.

As mentioned in *Volume I, Part 3: Evaluation and Qualification Criteria* of the Bidding Documents, the bidder's team requires a qualified geotechnical engineer/ engineering geologist throughout the bidding, design and construction phases of the project to review and evaluate the GIR and its information, the GDR and other geotechnical information, to inspect geotechnical investigations and to consider the application of the geotechnical information to the designs and construction plans.

2. SOURCES OF GEOLOGICAL AND GEOTECHNICAL INFORMATION

The geological and geotechnical information presented in the GIR has been obtained through the following desk studies, field investigations and laboratory tests :

- Regional geological studies in and around the project area.
- Detailed engineering geological mapping of the entire project.
- Subsurface exploration including exploratory drilling, geophysical surveys and exploratory test pits.
- In-situ tests including permeability tests in overburden, water pressure tests in rock and dynamic cone penetration tests.
- Laboratory testing of different rock core and construction material samples to determine physical and mechanical properties of the overburden material and rock.

Details of the various field surveys, investigations and laboratory tests are presented in the GDR.

3. GEOLOGICAL SETTINGS OF PROJECT

3.1 Regional Geology

3.1.1 Nepal Himalaya

The Nepal Himalaya is situated in the central part of the Great Himalayan Arc (Figure 1), spanning 885 km from east to west with an average width of about 165 km. It is divided into five major tectonic zones, viz. Tethys Himalaya, Higher Himalaya, Lesser Himalaya, Sub-Himalaya and Indo-Gangetic/ Terai Plain (Figure 2). The major features of these zones are summarized in Table 1.

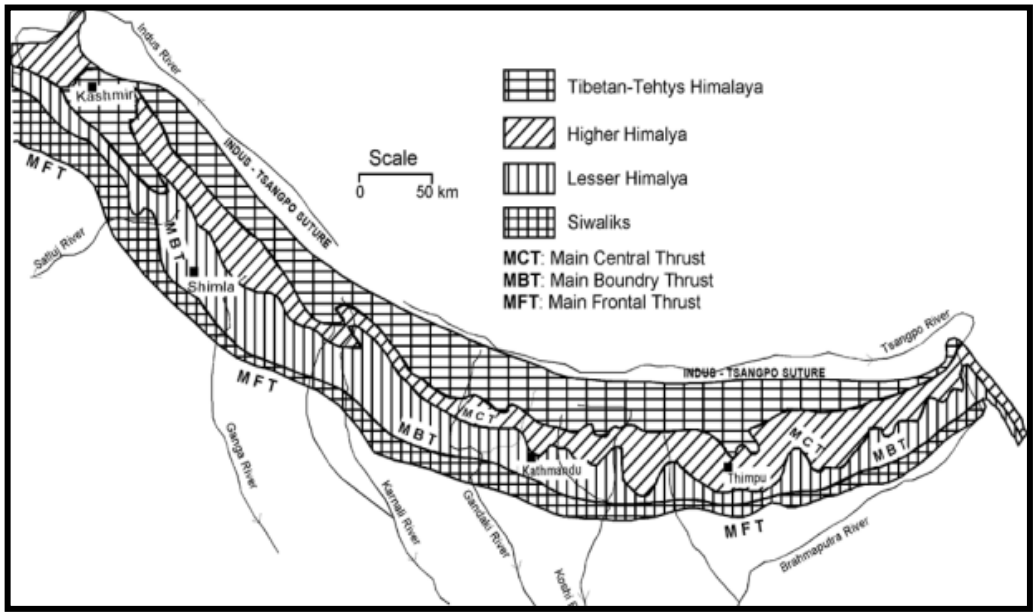


Figure 1: Geological division of the Himalaya (Ganser, 1964)

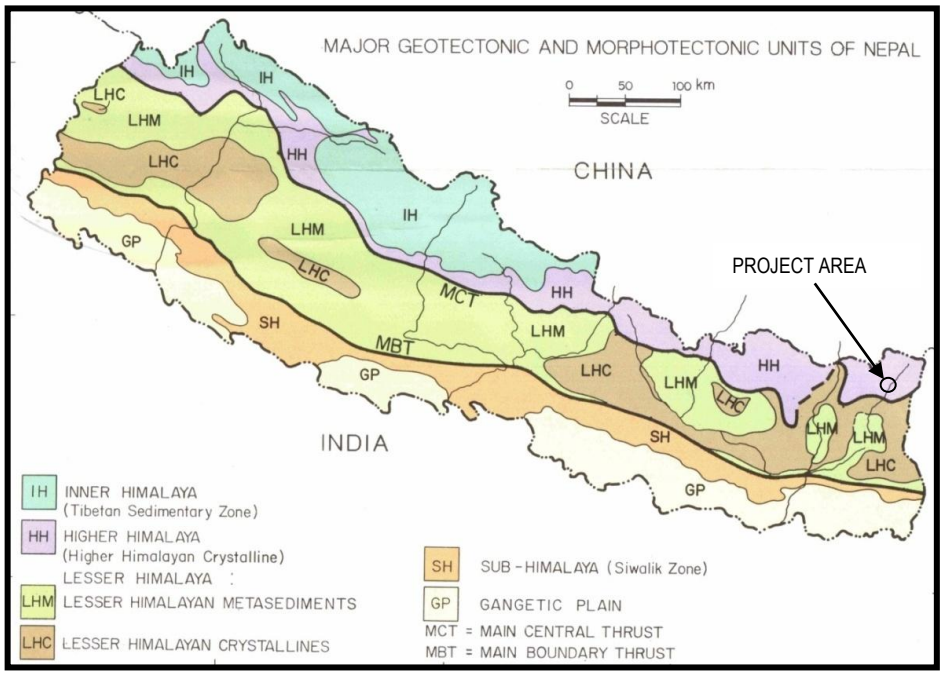


Figure 2: Simplified geological map of Nepal (UN, 1993)

Table 1: Geotectonic division of Nepal Himalaya (Sah, 2015)

Geological division	Stratigraphic units	Geological age
Tethys Himalaya	Paleotethys Group	Paleozoic (PZ)
	Neotethys Group	Mesozoic (MZ)
South Tibetan Detachment System (STDS)		

Higher Himalaya	Himal Group[2]		Pre-cambrian (P€)
Main Central Thrust (MCT) Zone			
Lesser Himalaya	Allochthonous succession	Phulchoki Group	Paleozoic (PZ)
		Bhimphedi Group	Pre-cambrian (P€)
	MT (Mahabharat Thrust)		
	Autochthonous succession	Midland Group	Pre-cambrian (P€)
		Gondwana Group	Late Paleozoic- Mesozoic (PZ_2-MZ)
		Tosh Group	Paleogene (p)
Main Boundary Thrust (MBT)			
Sub-Himalaya	Siwalik Group		Neogene (N)
Himalayan Frontal Thrust (HFT)			
Terai Plain	Quaternary Deposits		Quaternary (Q)

3.1.2 Stratigraphy

Ghunsa Khola flows through the eastern region of the Nepal Himalaya whose geological settings are shown in Figure 3. Stratigraphically, the rocks present in the Ghunsa region of the Nepal Himalaya are divided into four major successions, viz. Tethys Sediments, Higher Himalayan Crystallines, Lesser Himalayan Crystallines and Lesser Himalayan Metasediments.

3.1.3 Structure and Tectonics

The major structural elements dominating the tectonics of Nepal Himalaya are the MCT and the MBT. These structures have brought the Higher Himalayan crystallines onto the low-graded meta-sedimentary succession of the Lesser Himalaya, which in turn have been thrust over the molasse Siwalik sediments.

3.1.3.1 Major Tectonic Structures of Ghunsa Khola Region

The major tectonic structures of the Ghunsa Khola region are shown in Figure 4 and Figure 5. As indicated in these figures, the regional tectonic structures developed in and around this region are the Tamor Anticlinorium with Taplejung Window, MCT, MBT, HFT and other transverse and longitudinal faults.

The MCT is developed between the project area and Taplejung, crossing Tamor River about 25 km south of the former. Likewise, the MBT and the HFT are respectively located about 90 and 100 km south of the project site.

3.1.4 Geology of Ghunsa Khola Basin

Tectonically, the Ghunsa Khola basin is divided into two zones named Ghunsa Nappe and Tamor Nappe (Karmacharya & Kanskar, 1990). These zones are separated by a thrust plane dipping north and running slightly south of Silap and Thyangyan (Figure 6). The Ghunsa Nappe lies above the Tamor Nappe.

A few hundred meters northeast of Amjilosa, a sheared magmatite zone of about 25 m thickness is developed. The sheared plane appears to be gently dipping north-east. The rocks are highly distorted, and carbonaceous material is present in the sheared zone. Coarse-grained black biotite schists and quartzites are developed above the sheared zone, but quartz-feldspathic layers, reflecting the effects of migmatization, are present in places. The quartzose schists appear to be tectonized with crosscutting pegmatites in it.

The project area is located in the rock succession of Sekathum Formation belonging to the Tamor Nappe.

[2] Project area lies in this group

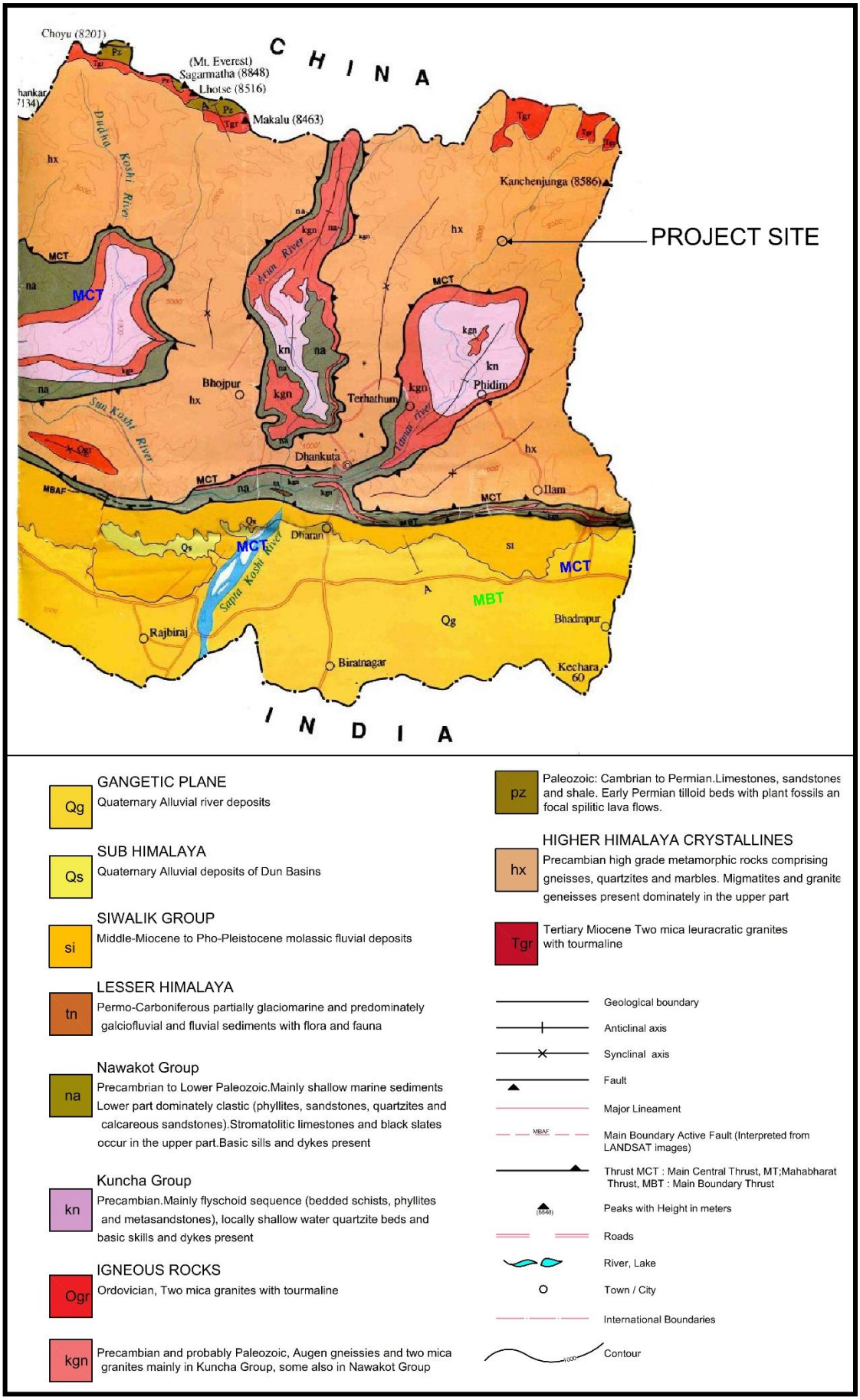
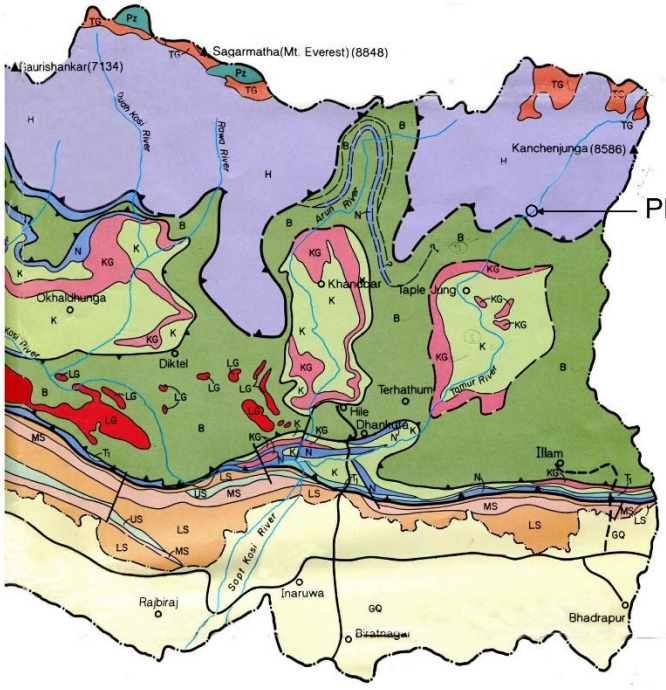


Figure 3: Geological map of eastern region of Nepal (Amatya & Jnawali, 1994)



Legend

GQ	Alluvial river deposits of Terai Plain
SQ	Sub Himalaya Alluvial deposits of Dun Basin
US	Upper Siwalik conglomerates with sandstone and shale
MS	Middle Siwalik : Sandstone and Shale
LS	Lower Siwalik : shales and sandstones
T1	Lesser Himalaya Permo-carboniferousn diamictite, shales and sandstones with coals
N	Nawakot Group: Precambrian slates, phyllites, sandstones quartzites stromatilitic limestones, basic sills and dykes.
K	Kuncha Group: Precambrian mainly flyschoid sequence of bedded schist, phyllites and metasandstones with basic sills and dykes
KG	Augun gneisses and two mica granites in Kuncha Group
KN	Nepheline Syenite in Kuncha group
P	Phulchowki Group: Lower Paleozoic limestone, phyllite, quartzite and siltstone with granitic intrusions
B	Bhimphedi Group: Precambrian schists, quartzites, marbles locally magmatised and gneissified and contain granitic intrusions
LG	Lower Paleozoic two mica cordierite bearing granites with tourmaline.
H	Higher Himalaya, Precambrian gneisses, quartzites and marbles. Migmatites and granite gneisses.
PZ	Paleozoic Tethys sediments
TG	Tertiary two mica lencocratic granites with tourmaline.
	Thrust deliminiting morpho-tectonic zones (MBT/MCT)
	Thrust deliminating lesser Himalayan crystalline thrust sheets.
	Fault
	Geological boundary
	Town/city
	Road
	River
	Mountain peaks

Figure 4: Geological map of eastern Nepal (UN, 1993)

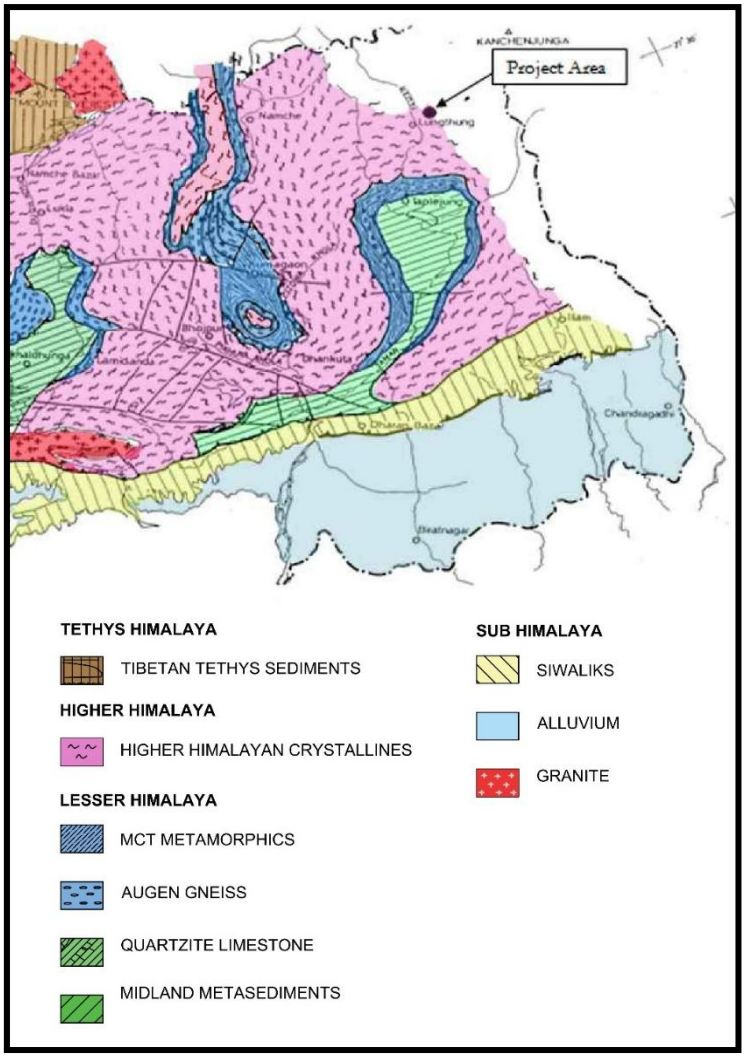


Figure 5: Tectonic map of eastern Nepal (Kizaki, 1987)

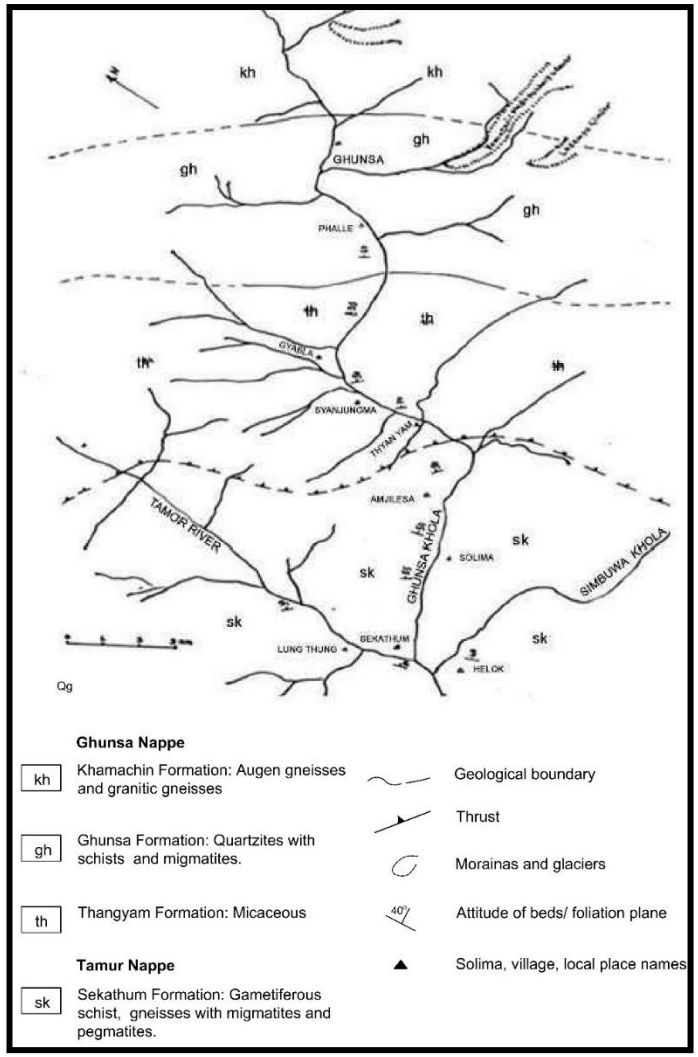


Figure 6: Geological map of Ghunsa Khola area (Karmacharya & Kanskar, 1990)

3.2 Seismicity

The Nepal Himalaya is recognized as a seismically active zone. Its seismic activity results from its presence in the Great Himalayan Arc formed by the collision of the Indian and the Eurasia Plates. As a consequence, all hydropower projects in Nepal, in general, are vulnerable to some degree of seismic risk. However, projects at some locations within the country are prone to specific seismic risks due to the presence of local active faults and thrusts.

Seismic studies for GKHEP focused on collection and review of available seismological data and information on the project area and the region, study of local geological and neo-tectonic settings and estimation of the ground motion parameters required for detailed design of the project components. The results of the studies are presented in the following sections.

3.2.1 Earthquake Sources

All major fault systems traversing Nepal, viz. the MCT, MT, MBT and HFT, are potential earthquake sources (Figure 7). In addition, there are 92 individual active faults and fault segments throughout the country that are potential earthquake sources.

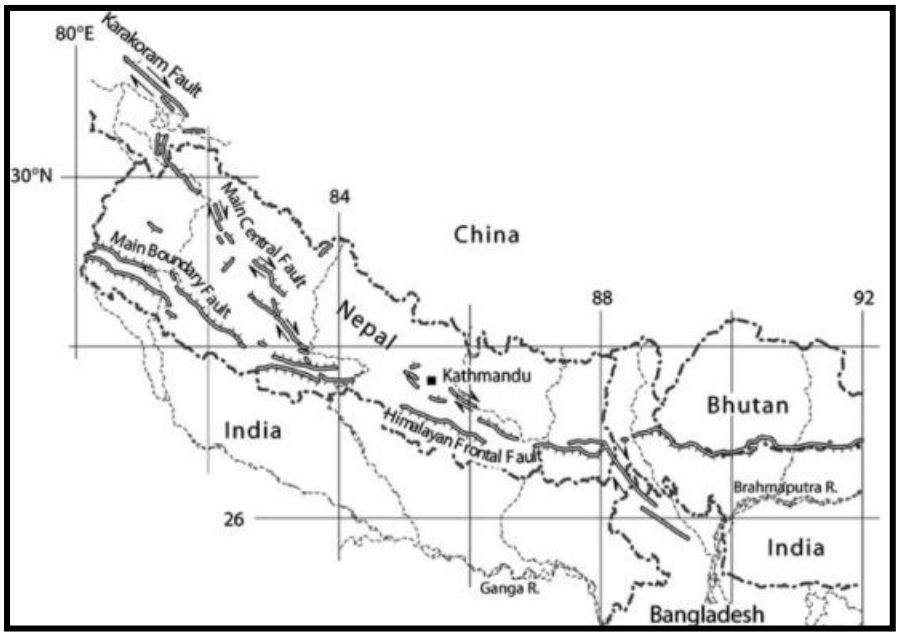


Figure 7: Active faults in and around Nepal Himalaya (Nakata & Kumahara, 2002)

Among the fault systems, MCT was active during the early phases of the Himalayan orogeny but is considered less active now. The largest recorded earthquake in the MCT zone between 1720 and 1982 is a 7.5 magnitude event that occurred on August 28, 1916.

The MT was also active during the early phase of the Himalayan orogeny but is less active now. However, the MBT is still active throughout the Himalaya region. The HFT is also considered active, with a maximum earthquake potential of 6.5 in magnitude.

3.2.2 Historical Seismicity

Over the years, Nepal has experienced several large earthquakes that have caused major destruction and loss of life and property (Table 2, Figure 8). Nepal's earliest recorded earthquake occurred in 1255, apparently near Kathmandu and had a maximum epicentral Modified Mercalli Intensity (MMI) of X, suggesting that its magnitude could have been around M 7.8. An earthquake with about the same magnitude occurred in this area in 1408, followed by a magnitude 7.0 earthquake in 1681. Earthquakes of at least magnitude 8 and 6.8 occurred near Kathmandu and to its northwest in 1833 and 1834.

The epicenter of the great 1934 Bihar-Nepal earthquake was located at Chainpur in far-eastern Nepal. The earthquake had a magnitude of 8.4, and it destroyed more than 80,000 homes and damaged another 104,000, mainly in Kathmandu valley and in the eastern Terai plains.

The 1988 Udaipur earthquake of magnitude 6.6 occurred in Udaipur District in eastern Nepal. It resulted in more than 700 deaths and destruction or damage to more than 65,000 buildings.

Table 2: Chronological listing of major earthquakes with magnitude greater than 6.5

Year	Location		Depth below ground (km)	Intensity	Magnitude
	Latitude (°)	Longitude (°)			
1720	28.70	77.20	NA	X	7 – 7.5
1803	30.30	78.20	NA	IX	6.5 – 7
1803	27.50	77.70	NA	IX	6.5 – 7
1833	28.00	85.00	NA	X	7 – 7.5
1833	27.00	85.00	NA	IX	6.5 – 7
1852	27.00	88.30	NA	IX	6.5 – 7
1869	28.00	85.00	NA	X	7 – 7.5
1905	33.00	76.00	NA	NA	8.00
1911	31.00	80.50	NA	NA	6.75
1916	30.00	81.00	NA	NA	7.50
1924	30.00	90.00	NA	NA	6.50
1930	25.50	90.00	NA	NA	7.10
1934	26.50	86.50	NA	NA	8.40
1934	31.25	89.25	NA	NA	7.10
1935	30.50	88.00	NA	NA	6.50
1935	28.75	89.25	140	NA	6.25
1936	28.50	83.50	NA	NA	7.00
1944	31.50	83.50	NA	NA	6.75
1944	34.00	84.00	NA	NA	6.75
1945	30.00	80.00	60	NA	6.50
1945	30.00	80.00	NA	NA	7.00
1945	32.50	76.00	60	NA	6.50
1956	28.15	77.67	NA	VIII	6.50
1958	31.00	85.00	NA	NA	6.63
1961	34.75	82.25	NA	NA	6.50
1966	29.90	80.20	100	NA	6.50
1982	31.69	82.24	33	NA	6.50
1988	26.70	86.60	57.94	NA	6.60
2011	27.72	88.06	19.7	NA	6.90
2015	28.255	84.07	15	VIII	7.90

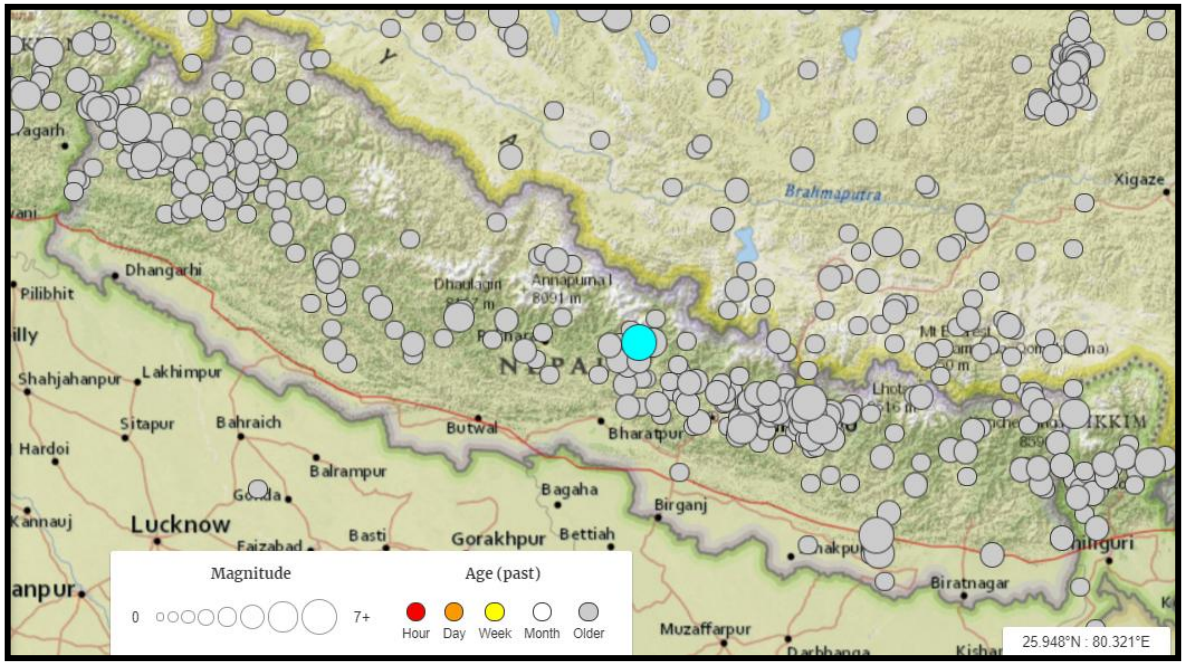


Figure 8: Seismicity of Nepal and surrounding areas (USGS, 2020b)

On September 18, 2011, an earthquake of magnitude 6.9, occurred in the Higher Himalaya region near Nepal's northeast border with India, with an epicentral distance of approximately 20 km (marked in blue in Figure 9). The earthquake caused substantial damage in Nepal, India and Bhutan, killing at least 108 people and injuring several hundreds. Several thousand buildings and many roads and bridges were destroyed, and electricity, telecommunication and water supply lines were disrupted. It also triggered landslides and mudslides in Sikkim of India, eastern Nepal and the Paro-Thimpu area of Bhutan. The preliminary focal mechanism of the earthquake suggests strike slip faulting.

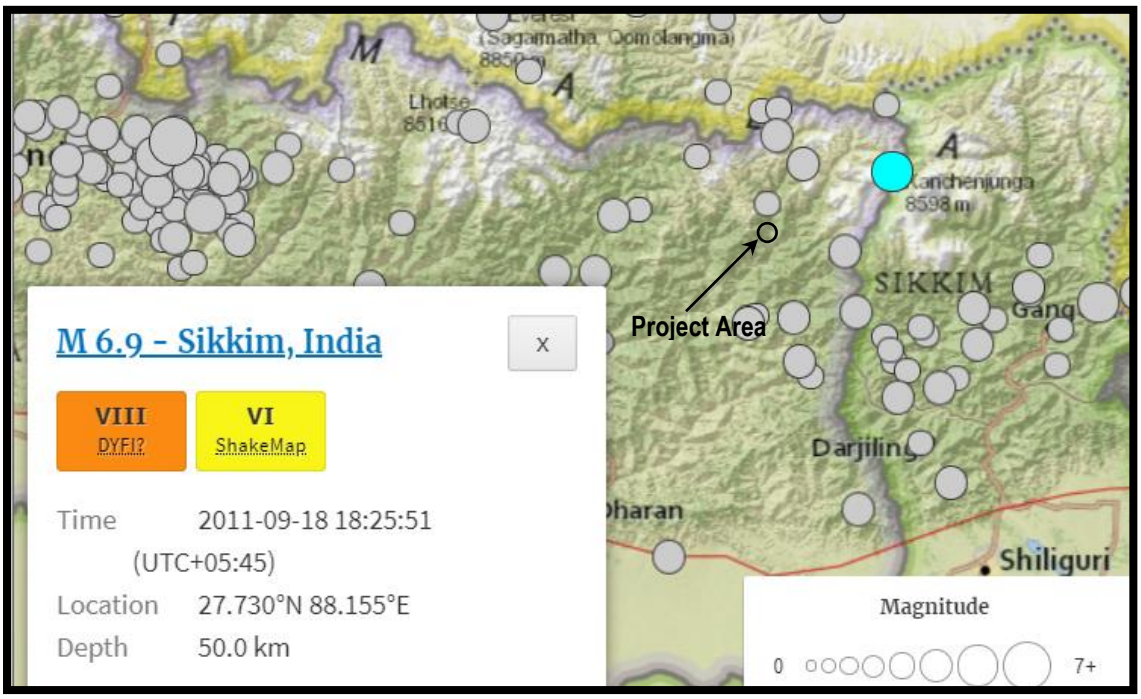


Figure 9: Location of Nepal – Sikkim earthquake of September 11, 2011 (USGS, 2020b)

On April 25, 2015, an earthquake of magnitude 7.9 with epicenter at Barpak in Gorkha District occurred, causing substantial damage in Nepal (marked in blue in Figure 8). About 9,000 people were killed and about 22,000 were injured. The earthquake damaged or destroyed around 60,000 houses and several roads and bridges. It triggered more than 4,300 landslides and ice avalanches, and it caused subsidence and liquefaction in Kathmandu valley. Substantial damage occurred in 11 surrounding districts as well.

It can be observed from Figure 8 that the seismic events recorded in Nepal have been mainly concentrated in the country's far-western, central and eastern regions. This figure shows that comparatively high seismic events have occurred around the Taplejung area in eastern Nepal, where the GKHEP project area lies. Figure 8 and Figure 9 further show that the recorded seismic events around the project area have generally been of magnitudes 2.5 to 4; however, the region has experienced relatively moderate seismicity in the past, with several earthquakes of magnitude 5 or greater occurring over the past 35 years within 100 km of the epicenter of the September 18, 2011, event.

3.2.3 Seismic Zoning of Nepal

Based on an analysis of seismic events and the tectonic structure of Nepal, the Seismic Hazard Mapping and Risk Assessment Component of NBCDP identified groups of earthquakes with major tectonic features, leading to identification and definition of seismic zones of assumed uniform seismicity in Nepal. The three seismic zones thus identified are shown in Figure 10. According to this zoning, GKHEP lies in Zone III.

3.2.4 Seismic Hazard Map

The seismic hazard map of Nepal prepared by DMG is shown in Figure 11. The map shows the peak ground accelerations (PGA) at bedrock with a 10% probability of being exceeded over 50 years.

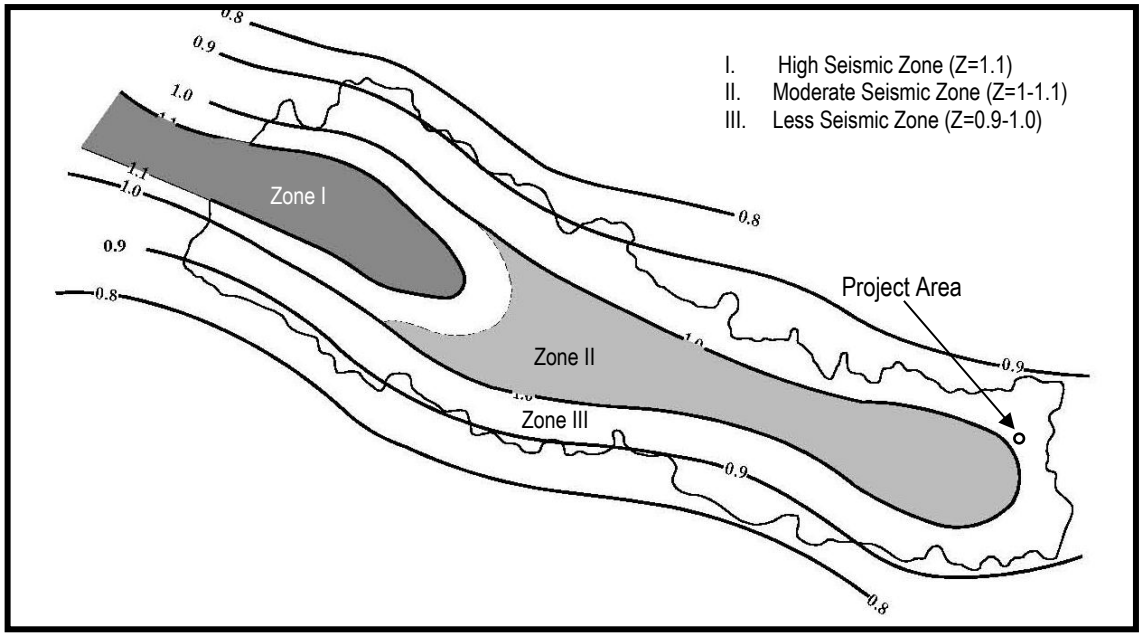


Figure 10: Seismic zoning map of Nepal (NBCDP 1994)

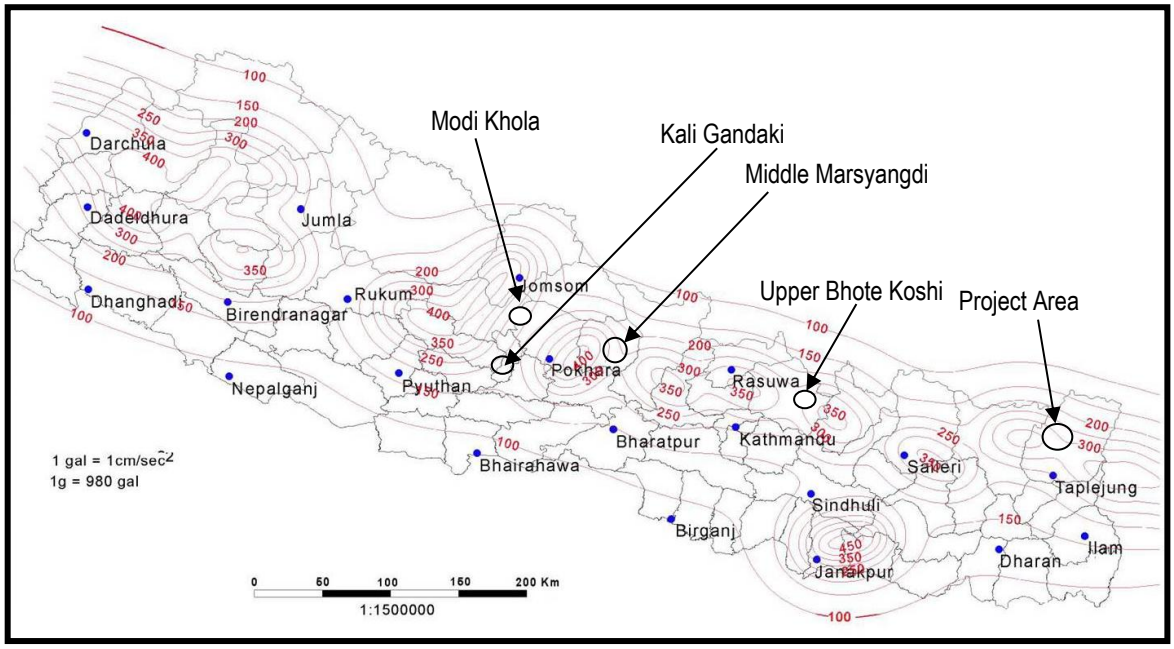


Figure 11: Seismic hazard map of Nepal (Pandey, et al. 2002)

3.2.5 Potential Earthquake Sources for GKHEP

Among the earthquake sources discussed in Section 3.2.1, the MCT is located around 25 km south of the project area. As such, the seismic risk for the project associated with the MCT should be taken into account despite the fact that the MCT is now considered inactive.

The MBT runs about 90 km south of the project area. Similarly, the HFT is also located very far away from this area. Therefore, despite being active, the MBT and HFT are not likely to pose significant seismic risks to the project structures.

The longitudinal fault developed across Tamor River below Taplejung, about 65 km south of the project area, is presently active. Movements along this fault could generate seismic events in the project area. Likewise, the local active faults developed north of Amjilosa and around Sekathum near the project area are also potential sources of seismic events in the project area.

3.2.6 Design Seismic Coefficients for GKHEP

The design seismic coefficients for the project components were estimated from relevant Indian and Nepali codes of practice.

3.2.6.1 Design Horizontal Seismic Coefficient

Indian Standard Code of Practice IS 1893: 1984

The Bureau of Indian Standards published its first code of practice for estimation of ground motion parameters, viz. IS 1893, in 1962, and it has since revised this code at regular intervals. One of the widely used versions of this code, viz. IS 1893: 1984, defined the design horizontal seismic coefficient, α_h, by the equation:

$$\alpha_h = \beta \times I \times a_0$$

where a_0 is the basic horizontal seismic coefficient, β is the soil foundation factor and I is the importance factor for the structure. According to the seismic zoning of India adopted in the IS 1893:1984 code, Nepal lies adjacent to the most severe seismic zone of India, i.e., Zone V, for which a_0 = 0.08. For the GKHEP project area, β can be adopted as 1 (medium soils) and I is 3 (for dams). Accordingly, α_h = 0.24 for the GKHEP site.

Indian Standard Code of Practice IS 1893: 2016

The latest version of the Indian standard, IS 1893: 2016 (Part I) deals with earthquake hazard assessment of buildings, liquid retaining structures, bridges, embankments, retaining walls and concrete, masonry and earth dams. As per this code, the design horizontal seismic coefficient, α_h, is defined by the equation:

$$\alpha_h = \left(\frac{Z}{2}\right) \times \left(\frac{S_a}{g}\right) \times \left(\frac{I}{R}\right)$$

where, Z is the seismic zoning factor, S_a/g is design acceleration coefficient for different soil types corresponding to the natural time period of the structure and R is the response reduction factor. Considering Nepal to be located in Zone V, Z = 0.36. As the concrete structures of the project area relatively stiff with short time periods, S_a/g can be adopted as 2.5. Likewise, adopting I = 1.5 for important structures (such as power stations) and R = 3 for ordinary structures, α_h = 0.225 for the GKHEP site.

National Building Code Development Project (1994)

According to the seismic design code of Nepal (NBCDP 1994), the effective design seismic coefficient a_{eff} can be computed as

$$a_{eff} = \frac{R \times a_{max}}{0.98}$$

where R is an empirical reduction factor between 0.5 and 0.65 and a_{max} is the maximum acceleration for the seismic zone under consideration. For seismic zone III, $a_{max} = 0.3g$, g being the acceleration due to gravity. For R values of 0.5 and 0.65, the above equation yields a_{eff} as 0.15 and 0.2, respectively. Thus, the estimated design horizontal seismic coefficient for the project ranges from 0.15 to 0.2.

Recommended Seismic Coefficients

For comparison purposes, the horizontal design seismic coefficients adopted for some of the major hydropower projects in Nepal are listed in Table 3 (see Figure 11 for locations of these projects). Of the projects listed in the table, the Kali Gandaki, Upper Bhote Koshi and Middle Marsyangdi Hydroelectric Projects has dam heights of 43 m, 22 m and 56 m, respectively, while Modi Khola Hydroelectric Project had a weir of 5.5 m height. Furthermore, the downstream risks to life and property associated with these projects were significantly higher than those for GKHEP.

Table 3: Horizontal design seismic coefficients for hydropower projects in Nepal

Project	Coefficient	Source
Kali Gandaki "A" Hydroelectric Project	0.3	Site-specific studies
Modi Khola Hydroelectric Project	0.1	Empirical
Upper Bhote Koshi Hydroelectric Project	0.2	Empirical
Middle Marsyangdi Hydroelectric Project	0.2	Site-specific studies

From the above calculations, it is clear the Indian standard gives slightly higher value of the design horizontal seismic coefficient compared to NBCDP. Furthermore, its coefficient is comparable with the design coefficients adopted in some of the major hydropower projects in Nepal listed in Table 3. As such,

considering the size and nature of the GKHEP structures and the minimal downstream risks of loss of life and property associated with their failure, a horizontal design seismic coefficient (α_h) of 0.25 is recommended for the project. Likewise, the vertical design seismic coefficient may be taken as two-third of the horizontal coefficient.

As per IS 1893: 2016 (Part I), the design horizontal acceleration value for underground structures and foundation deeper than 30 m shall be taken as half the value computed for surface. For structures and foundations placed between the ground and 30 m depth, the design horizontal acceleration value shall be linearly interpolated between α_h and 0.5 α_h.

3.3 Geology of Project Area

3.3.1 Geomorphology

Within the project area, Ghunsa Khola flows in a narrow, V-shaped valley (Photo 1), mostly with precipitous valley faces for which high escarpments are common on both banks. It descends from an elevation of 1,940 m at the weir site to 1,501 m at the tailrace outfall in a distance of about 5.77 km and thus has a steep gradient of 1 in 13 in this reach. An approximately 20 m high waterfall forms a prominent landscape along the river in the project area and, in a deviation from the general steep river gradient, is characterized by gentle river gradients both towards its upstream and downstream.

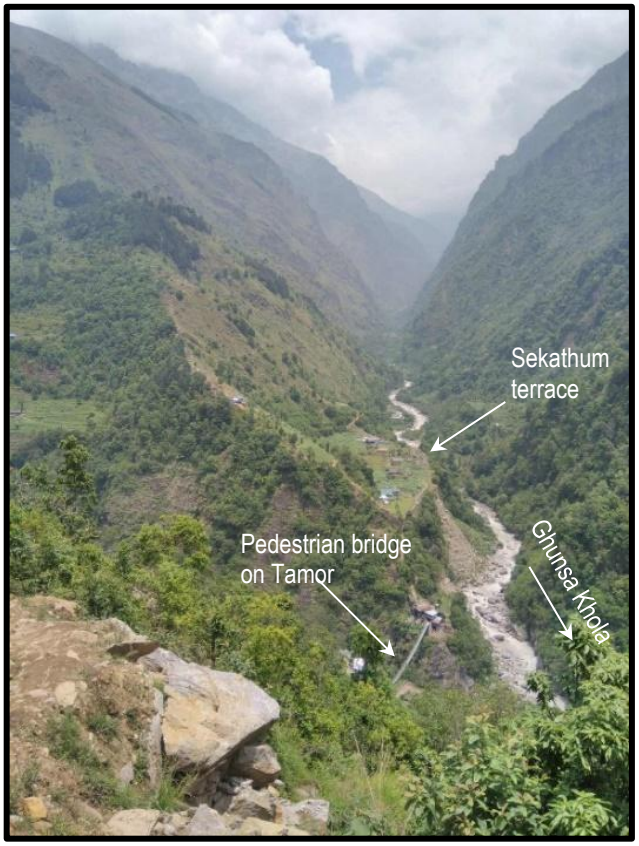


Photo 1: Narrow, V-shaped valley of Ghunsa Khola

Bedrock is conspicuously exposed on valley faces. A conspicuous geomorphic feature represented by a steep rocky scarp face in sharp contact with a major overburden covered flatter slopes (Photo 2), separated by a sudden break-in-slope is seen adjoining the surge shaft location.

The lower reaches of the valley close to the riverbed, however, are largely covered with colluvial deposits and occasional narrow river terraces. A conspicuous river terrace is found near the confluence of Ghunsa Khola and Tamor (Photo 1). Sekathum village and some of the main hotels and lodges on the trail to Kanchenjunga Base Camp are located on this terrace.

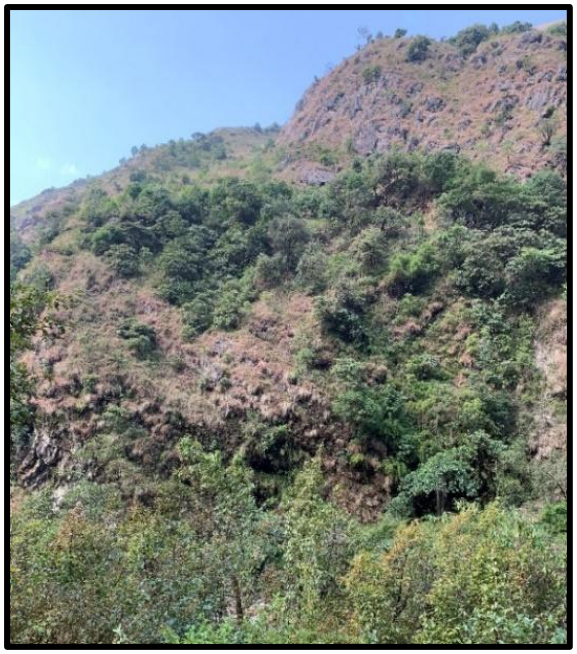


Photo 2: Scarp development along geological contact

Detailed engineering geological mapping of the project area has been performed, and geological maps of the project area and all major structures, along with sections and profiles, have been prepared at suitable scales. The maps, sections and profiles are included in Annex 1 of the GDR.

3.3.2 Lithostratigraphy

3.3.2.1 Lithology

The project area is located in a moderately upstream dipping rock succession of Precambrian Sekathum Formation and is divided into two distinct sequences termed here as the Lamatar Unit towards the upstream and Japantar Unit in the downstream (Figure 12) The rocks of Lamatar Unit predominate the project area and occupy the area from the headworks to the surge shaft upstream of the Japantar pedestrian bridge (Pedestrian Bridge 4). On the other hand, the Japantar Unit is restricted to the powerhouse area. The geological contact between the Lamatar and Japantar Units is concealed in the overburden covered reach close to the conspicuous geomorphic expression represented by an escarpment.

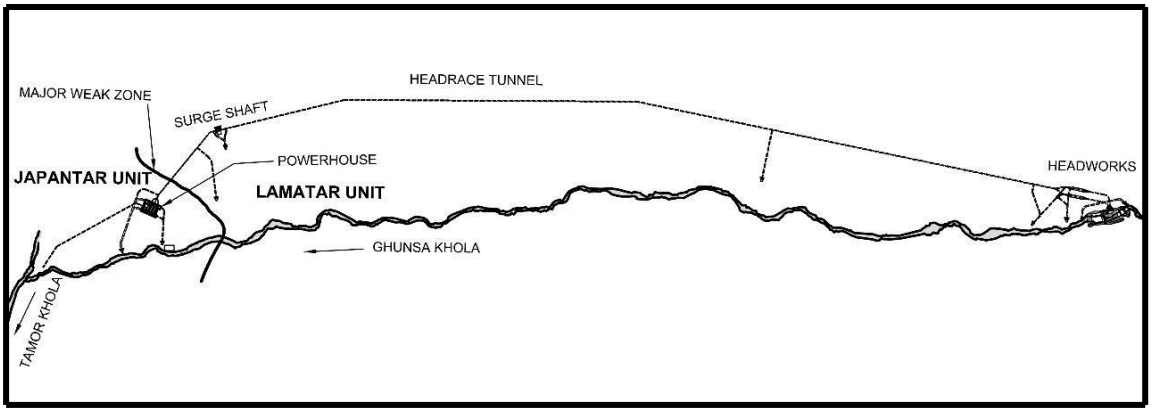


Figure 12: Disposition of Lamatar and Japantar units

Lamatar Unit

The Lamaar unit is dominated by thick beds of garnetiferous banded gneisses (Photo 3). The rock is principally composed of quartz, feldspar and biotite with occasional tourmaline, and it is characterized by the conspicuous presence of pink garnet.

Japantar Unit

The Japantar unit is represented by a mixed lithology of quartz biotite gneiss (Photo 4), quartzite and marble (Photo 5).

Photo 3: Garnetiferous Banded Gneiss (GBG) near headworks

Photo 4: Quartz biotite gneiss in powerhouse area

Photo 5: Marble outcrop near powerhouse site

The gneisses are fine to medium grained, well foliated and composed essentially of quartz, feldspar and biotite. Presence of calcite occasionally makes the rock calcareous. The quartzites are fine to medium grained, well bedded and consists essentially of quartz. The marble, on the other hand, is medium grained and saccharoidal in texture. It is mostly white in color. Bedding is not very well developed and is seen occasionally. Based on the observed mineral composition, the rock has been termed calcareous quartz gneiss, but the term marble has been retained for general engineering geological reference.

3.3.2.2 Overburden

Overburden of colluvium and alluvium is fairly widespread in the project area. However, the overburden of slide debris is limited.

Colluvial deposits are fairly well distributed all over the project area. They occur on gentler valley slopes and are particularly present at the base of valley faces. At places, the colluvial deposits are substantial in thickness and extent, such as in the surge shaft area. The colluvium consists of angular fragments of local rock types embedded in a silty matrix.

In the project area, besides the riverbed along Ghunsa Khola, alluvial deposits are found in the form of narrow river terraces and recent river deposits on the river. The most conspicuous river terrace of 60 m height is located at Sekathum village (Photo 6). In general, the alluvium comprises rounded to sub-rounded river-borne material (RBM) of various grades, viz. boulders, cobbles, pebbles, gravels, etc., that is set in a sandy or silty matrix.

Photo 6: Close up view of major river terrace at Sekathum village

Major landslides are uncommon in the project area. Minor dormant active landslides restricted to overburden are found more frequently.

3.3.3 Structural Discontinuities

3.3.3.1 Joints

In all, 520 joint records were collected and analyzed with the DIPS® software developed by Rocscience Inc. Of these, 118 records are from the headworks site, 221 from the headrace tunnel alignment and 181 from the powerhouse area. The stereographic plot of the discontinuities for the project area is given in Figure 13.

The rock mass is dissected by four sets of joints, viz., J1: N N037°/ 40° (foliation), J2: N235°/ 74°, J3: N N166°/ 69° and J4: N N300°/ 56°. Joint sets J1 to J3 uniformly figure in all the stereographic analyses for the headworks area, the headrace tunnel alignment and the powerhouse area; however, gentler foliation dips are found in powerhouse area. The joint set J4, on the other hand, is found only along the headrace tunnel alignment and the headworks area, indicating that these joints appear in the upstream area. In general, it is interpreted that the structural trends of the rock mass are fairly uniform in the project area (Photo.7). Mild steepening of the foliation dips by 4° to 5°, however, is discernible towards the headworks area. The engineering properties of the identified discontinuity sets are given in Table 4.

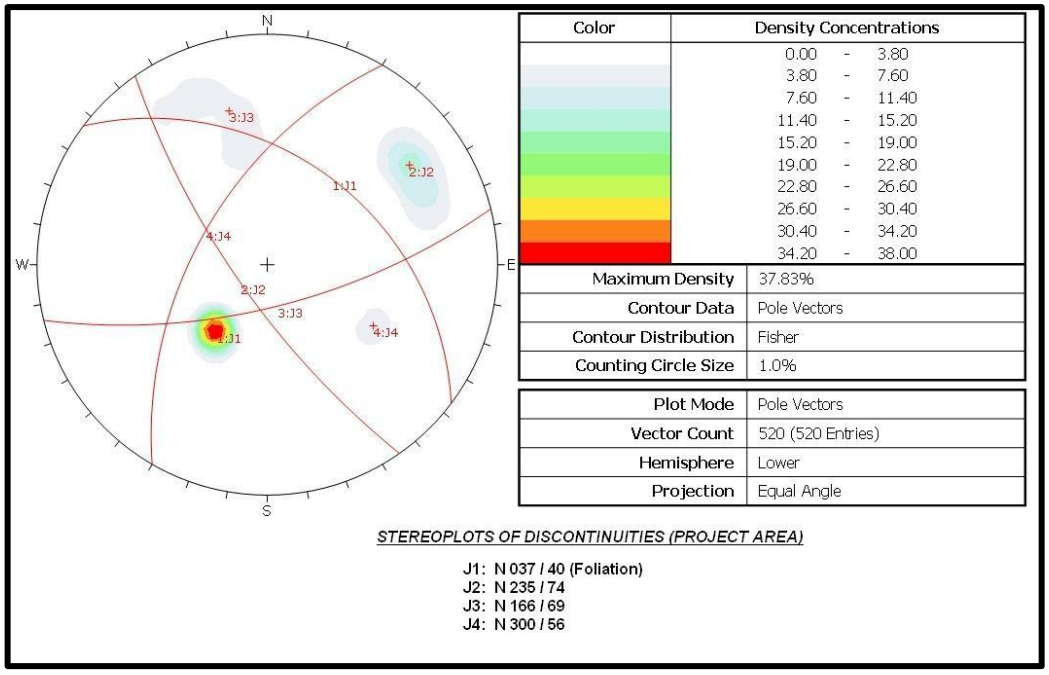


Figure 13: Stereoplot of discontinuities in project area (520 records)

Photo 7: Major joint sets in the project area (Sets J1, J2 andJ3)

Table 4: Discontinuity sets in project area and their engineering properties

Set No.	Feature	Dip (Dir./ amt.)	Persistence (m)	Spacing (m)	Aperture (mm)	Surface[3]	Filling
J1	Foliation/ Joint	N037°/ 40°	5 to >10	0.1 to >3.0	Tight to 0.2	RU	Nil to clay
J2	Joint (longitudinal)	N235°/ 74°	3 to >10	0.5 to >5.0	Tight to 0.1	RU to RP	Nil to clay
J3	Joint (transverse)	N166°/ 69°	3 to >7	0.5 to >5.0	Tight to 0.2	RU to RP	Nil to clay
J4	Joint	N300°/ 56°	5 to 7	1.0 to >5.0	Tight to 0.2	RU	Nil to clay

[3] RP: Rough planar; RU: Rough undulating.

3.3.3.2 Shear Zones

The observed shear zones in the project area are interpreted to have preferential orientation along joint set J1, i.e., foliation of the strata. The most prominent shear zone, however, is found oriented along joint set J2. The thickness of the shear zones is found to be restricted to 300 mm.; however, their persistence goes beyond 10 m and is considered significant.

3.3.4 Hydrogeological Conditions

Ghunsa Khola has a mean annual flow of 32.21 m^3/s and an estimated 1 in 100 years' flood of 880 m^3/s. The GKHEP project area lies in a high relief terrain with moderate altitudes in excess of 1,500 m amsl. Although snowfall is low and restricted to the peak winters, the annual rainfall in the area is high, about 2,257 mm. In general, springs in the project area are uncommon but are seen in the rainy season. Likewise, during monsoons, waterfalls also appear along seasonal drainages.

Based on the results of drilling and water percolation tests, the depth of water table has been found to vary between 0.65 m (in the drill holes along the river bed) to 92 m. Similarly, the permeability values have been found to be between 4.558 and 44.131 Lugeon.

4. GROUND CHARACTERIZATION AND INFORMATION

4.1 Introduction

Ground characterization in the headworks area, powerhouse area and along the headrace tunnel alignment was performed based on the Rock Tunnel Quality Index or Q-System (NGI, 2015). In addition, the Geomechanics Classification or RMR System (Bieniawski, 1976) was undertaken to estimate the rock mass rating of the rock masses. For Information purpose, the rock mass class based on the Q-System was taken into consideration (Table 5).

Table 5: Rock classes based on Q-system (NGI, 2015)

Rock Mass Class	Q Classification	
	Q-value	Category
I	>40	Very to Exceptionally Good
II	10 – 40	Good
III	4 – 10	Fair
IV	1 – 4	Poor
V	0.1 – 1	Very Poor
VI	< 0.1	Extremely Poor

4.2 Headworks Area

With an average width of about 20 m, Ghunsa Khola flows from ENE to WSW through a narrow V-shaped, asymmetrical and deeply incised valley. While the left bank of the river at this location is covered with dense forests with the main track to the upper catchment reaches, the right bank is fairly rocky. The river drops as an approximately 20 m high waterfall immediately downstream of the diversion weir (Photo 8).

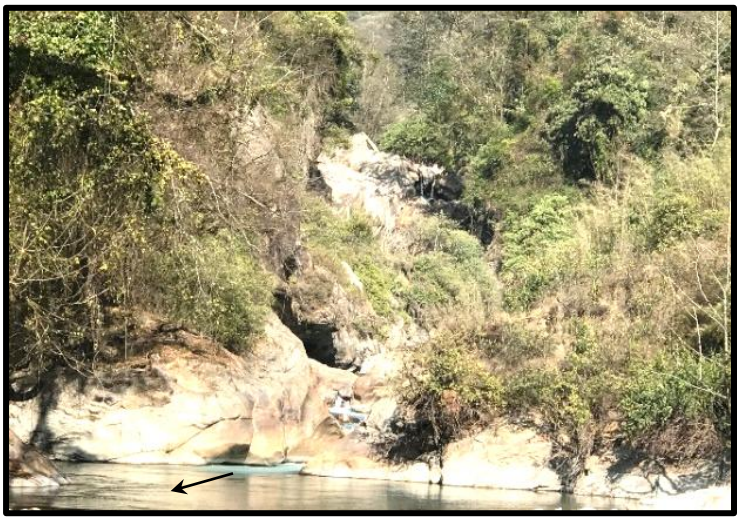


Photo 8: Waterfall downstream of weir site in dry season (looking upstream)

The headworks site is located in the wholly gneissic Lamatar Unit (refer geological map and section of the headworks area in *Volume V: Employers Concept Drawings (GK-GE-S-02)*. The rock mass is dissected by four sets of joints, viz., J1: N033/43 (foliation), J2: N246/73, J3: N154/76 and J4: N300/56. Data on the major discontinuity sets in for the headworks area, along with their engineering properties, are given in Table.6 and presented in Figure.14.

Table 6: Discontinuity sets in headworks area and their engineering properties

Set No.	Feature	Dip (Dir./ amt.)	Persistence (m)	Spacing (m)	Aperture (mm)	Surface[4]	Filling
J1	Foliation/ Joint	N033°/ 43°	5 to >10	0.1 to >3.0	Tight to 0.2	RU	Nil to Clay
J2	Joint (longitudinal)	N246°/ 73°	3 to >10	0.5 to >5.0	Tight to 0.1	RU to RP	Nil to Clay
J3	Joint (transverse)	N154°/ 76°	3 to >7	0.5 to >5.0	Tight to 0.2	RU to RP	Nil to Clay
J4	Joint	N300°/ 56°	5 to 7	1.0 to >5.0	Tight to 0.2	RU	Nil to Clay

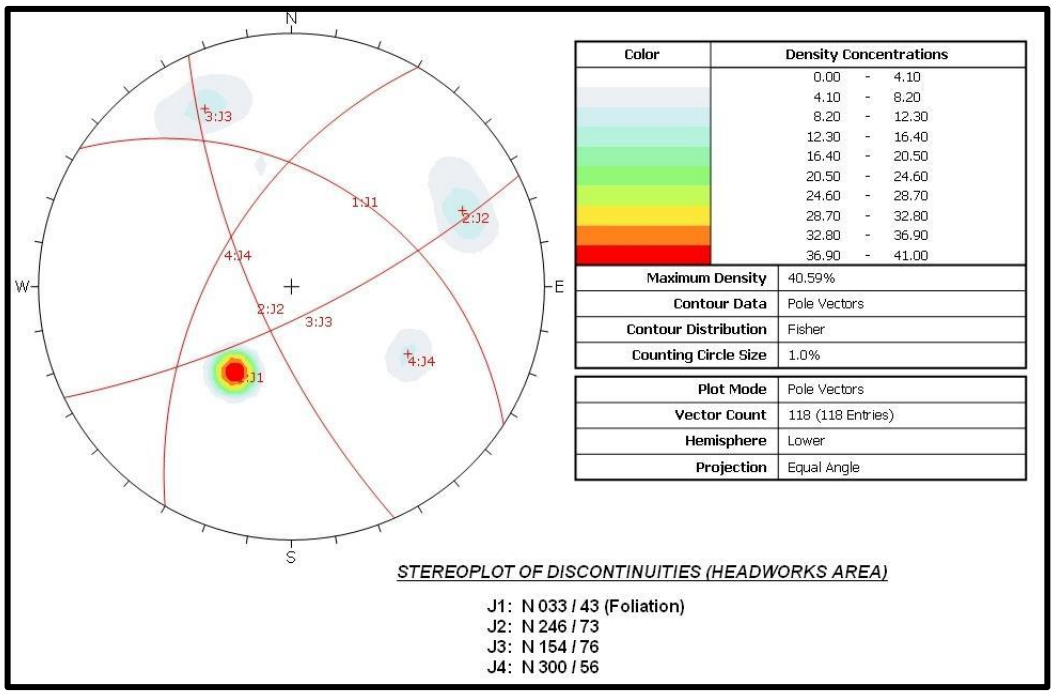


Figure 14: Stereoplot of discontinuities in headworks area (118 records)

4.2.1 Geotechnical Information of Weir

- For Information purposes, the deepest elevation of sound bedrock in the riverbed shall be taken as El. 1917.6 m (23.4 m depth). The weir is designed as a 14.5 m high structure on floating foundation.
- The permeability of the bedrock varies between 13.553 and 15.489 Lugeon. Likewise, the permeability of the overburden ranges from 1.06 X 10-3 to 9.79 X 10-4 cm/sec.

4.2.2 Geotechnical Information of Appurtenant Structures

- All the appurtenant structures at the headworks are underground and mainly include the diversion tunnel and the desilting chambers with the connecting and other ancillary tunnels.
- The portal faces of all underground components are located in rock.
- Joints are moderately to widely spaced with rough undulating surface and are generally tight. Orientation of the joints indicates that wedges will be formed and will have to be stabilized.
- Good to fair quality rock, with occasional poor to extremely poor quality rock, will be encountered.

[4] RP: Rough planar; RU: Rough undulating.

4.3 Headrace Tunnel

Located on the right bank of the Ghunsa Khola, the 4208.65 m long headrace tunnel is proposed to be excavated as a D-shaped tunnel through more or less homogeneous media represented mainly by garnetiferous banded gneisses with local intercalations of augen gneisses, migmatites and pegmatites (refer geological map and longitudinal profile of headrace tunnel in *Volume V: Employers Concept Drawings (GK-GE-S-04)*. The tunnel is mostly aligned through exposed rock with occasional colluvium covered pockets that are not expected to be thick. The maximum overburden cover of the tunnel is 270 m while the minimum cover is 65 m. High inflows are expected below Dople Khola and Jongim valley.

The rock formations along the headrace tunnel are traversed by structural discontinuities including joints, foliation, shear zones, etc. The major discontinuity sets with their engineering properties are given in Table 7, and their stereoplot is presented in Figure.15.

Table 7: Discontinuity sets along headrace tunnel and their engineering properties

Set No.	Feature	Dip (Dir./ amt.)	Persistence (m)	Spacing (m)	Aperture (mm)	Surface[5]	Filling
J1	Foliation/ Joint	N037°/ 38°	5 to >10	0.1 to >3.0	Tight to 0.2	RU	Nil to Clay
J2	Joint (longitudinal)	N246°/ 73°	3 to >10	0.5 to >5.0	Tight to 0.1	RU to RP	Nil to Clay
J3	Joint (transverse)	N155°/ 75°	3 to >7	0.5 to >5.0	Tight to 0.2	RU to RP	Nil to Clay
J4	Joint	N301°/ 55°	5 to 7	1.0 to >5.0	Tight to 0.2	RU	Nil to Clay

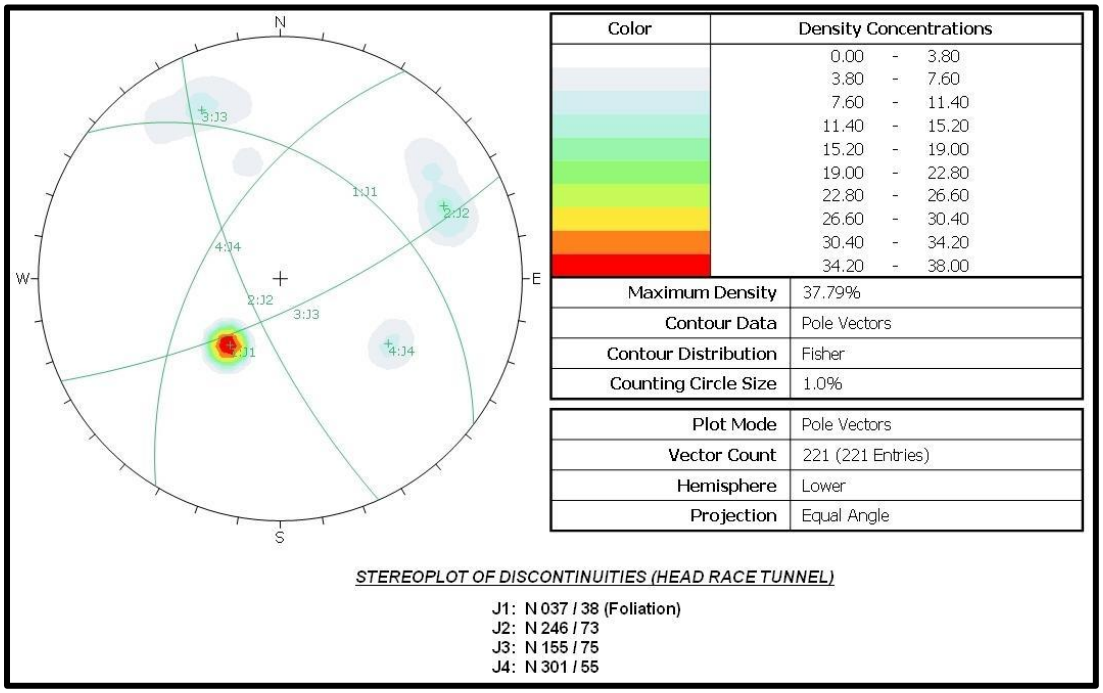


Figure 15: Stereoplot of discontinuities in the HRT area (221 records)

4.3.1 Engineering Geological Conditions

The relationship of the headrace tunnel alignment with discontinuities is presented in Figure 16. In general, the tunnel alignment cuts across the ruling set of discontinuities, viz. rock foliation, and is considered favorably oriented. With respect to the other joint sets also, the tunnel alignment is reasonably oriented barring its downstream stretch that makes a small angle with sets J2 and J3 and is, hence, unfavorably disposed.

[5] RP: Rough planar; RU: Rough undulating.

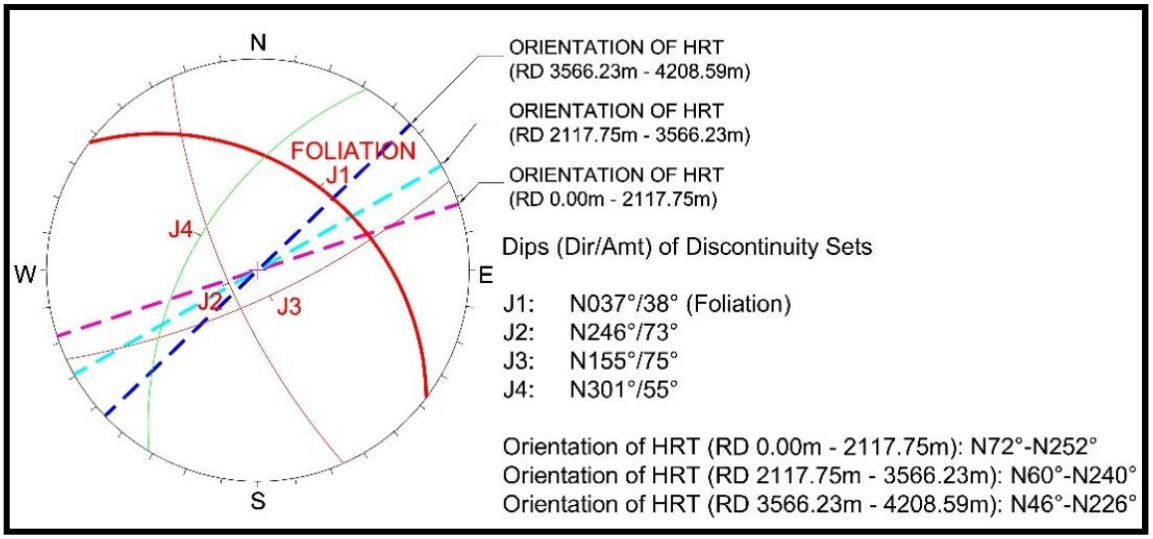


Figure 16: Relationship of headrace tunnel alignment with discontinuity sets

4.3.2 Rock Mass Condition

The distribution of rock classes along the headrace tunnel based on projections from engineering geological mapping (Annex.1 of GDR) is given in Table.8. The bidder may independently assess the rock mass conditions along the headrace tunnel.

Table 8: Rock mass classification along headrace tunnel

Chainage (m)		Length (m)	Feature	Rock class
From	To			
0+000	0+230	230	Normal rock condition, good rock cover	II – III
0+230	0+480	250	Thotane Khola crossing, low ground cover	III – V
0+480	1+840	1360	Normal rock condition, good rock cover	II – III
1+840	2+040	200	Low ground cover	III – V
2+040	2+220	180	Normal rock condition, good rock cover	II – III
2+220	2+270	50	Low ground cover	IV – VI
2+270	4+000	1730	Normal rock condition, good rock cover	II – III
4+000	4+208	208	Low ground cover	III – V

Note: Rocks of Class-I and VI may be expected in nominal lengths.

4.3.3 Reference Status

The estimated lengths and percentages of rock mass classes (based on the Q-system) for different rock types that are likely to be encountered in the headrace tunnel are given in Table 9.

Table 9: Summary of distribution of rock classes along headrace tunnel

Rock class	Length (m)	Length (%)
II	1,170	28
III	2,050	49
IV	688	16
V – VI	300	7
Total	4,208	100

4.4 Surge Shaft – Powerhouse – Tailrace Complex

The proposed Surge Shaft- Powerhouse - Tailrace complex is located around Sekathum-Japantar villages on the right bank of the Ghunsa Khola, immediately upstream of its confluence with the Tamor Khola. The area lies in a terrain occupied by exposed rocks and colluvium and terrace deposits. The TRT alignment runs through back slope of a prominent high level river terrace located in Sekathum village area. The surge shaft and a major part of the pressure shaft on upstream side are located in Lamatar Unit represented by garnetiferous banded gneisses (GBG), remaining pressure shaft and the powerhouse – tail race complex are located in Japantar Unit represented by quartz biotite gneiss with interbanded quartzite and marble. Geological maps and sections of the powerhouse area, main access tunnel and the tailrace tunnel are given in Annex 1 of the GDR.

The strata are well foliated and jointed. The gneisses belonging to Lamatar Unit are dissected by four sets of joints, viz. Set J1: N037/38 (Foliation), J2: N246/73, J3: N155/75 and J4: N301/55. Gneisses with interbedded quartzite and marble belonging to Japantar Unit are dissected by three sets of joints including Set J1: N041/39 (Foliation), J2: N234/73, and J3: N172/57. Engineering properties of the joints are given in Table 10 and is presented in Figure.17.

Table 10: Discontinuity sets in surge shaft – powerhouse – tailrace complex

Set No.	Feature	Dip (Dir./ amt.)	Persistence (m)	Spacing (m)	Aperture (mm)	Surface[6]	Filling
J1	Foliation/ Joint	N041°/ 39°	5 to >10	0.1 to >3.0	Tight to 0.2	RU	Nil to Clay
J2	Joint (longitudinal)	N234°/ 73°	3 to >10	0.5 to >5.0	Tight to 0.1	RU to RP	Nil to Clay
J3	Joint (transverse)	N172°/ 57°	3 to >7	0.5 to >5.0	Tight to 0.2	RU to RP	Nil to Clay

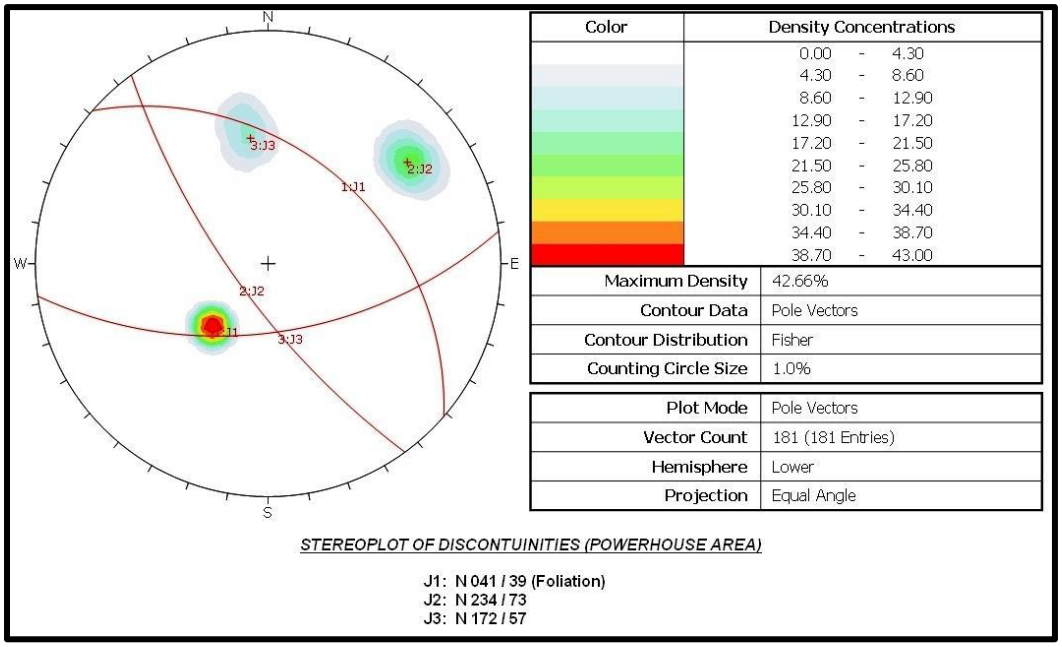


Figure 17: Stereoplot of discontinuities in the powerhouse area (181 records)

4.4.1 Engineering Geological Conditions

- Colluvium deposits predominate the valley slopes but may not necessarily be thick except across the pressure shaft alignment where drilling and ERT surveys have established a colluvium thickness of 30 to 40 m.
- The surge shaft is proposed in exposed gneisses belonging to the Lamatar Unit with parts of surrounding areas covered with superficial overburden of colluvium. ERT surveys in overburden dominated areas in the vicinity of this site, however, have indicated the presence of weak rock zones at depth.
- On the basis of the ERT results, the presence of carbonaceous material in overburden and results of drilling, a major weak zone is expected along the geological boundary between the Lamatar and Japantar Units (*Volume V: Employers Concept Drawings (GK-GE-S-05)*. ; Annex1 of GDR). The weak zone, therefore, is interpreted to be dipping N~040°/40° and could be represented by fractured and/ or sheared mass. It is estimated to be about 25 m thick and interpreted to intersect the middle horizontal segment of the pressure shaft.
- Barring the upper horizontal segment of the pressure shaft of insignificant length, the horizontal segments in the lower reaches make an angle of 58° with the foliation and are considered favorably oriented.
- The powerhouse and transformer caverns are oriented parallel to each other in the N086°-N264° direction and make a reasonably good angle of 45° with the foliation.

[6] RP: Rough planar; RU: Rough undulating.

- The site for the powerhouse cavern has been explored through a 105 m deep drill hole that intersected interbedded quartz biotite gneisses and marble with interbands of quartzite below a 5 m thick overburden of colluvium. However, a deeper hole down to the crown level of the cavern is considered advisable. This shall help assess the rock mass condition in the crown region of the cavern, along with lithological variation with particular reference to the presence of marble beds.
- The portal development for main access tunnel to the powerhouse and transformer caverns may require nominal slope stabilization measures through rock bolting and shotcreting. The tunnel makes a rather low angle of 38° with the foliation and may not be considered favorably oriented, and foliation shears, if intersected, may traverse this tunnel for longer lengths with consequent increase in the proportion of poor rock.
- The portal development for cable-cum-ventilation tunnel may require nominal slope stabilization measures through rock bolting and shotcreting. The tunnel makes a rather low angle of 22° with the foliation and may not be considered favorably oriented, and foliation shears, if intersected, may traverse the tunnel for longer lengths with consequent increase in the proportion of poor rock.
- The tailrace tunnel cuts across quartzite with marble bands and quartz biotite gneiss. The alignment has been explored through a 30 m deep drill hole in the backslope of the Sekathum river terrace and has intersected good bedrock of quartz biotite gneiss below a 13.5 m thick overburden and, hence, has established bedrock at tunnel grade.
- The outlet portal of tailrace tunnel is located close to the bridge across Tamor River, immediately downstream of the Sekathum terrace.Pportal development is expected to negotiate some overburden of colluvium, but, once in rock, it may require nominal slope stabilization measures through rock bolting and shotcreting.

4.4.2 Geotechnical Information of Surge Shaft

- The surge shaft is proposed in garnetiferous banded gneisses belonging to the Lamatar Unit dissected by four plus random sets of joints, viz. J1: N037/38 (Foliation), J2: N246/73, J3: N155/75 and J4: N301/55.
- The structure is likely to be excavated generally in Rock Class II and III (Good to Fair) with occasional presence of Rock Class IV, V and VI (Poor to Extremely Poor).

4.4.3 Geotechnical Information of Pressure Shaft

- The pressure shaft is proposed in garnetiferous banded gneisses belonging to Lamatar Unit and quartz biotite gneisses with interbands of quartzite and marble of Japantar Unit. The bedrock is dissected by three plus random sets of joints, viz. Set J1: N041/39 (Foliation), J2: N234/73, and J3: N172/57.
- The geological boundary of the Lamatar and Japantar Units is interpreted to be a weak zone.
- The shaft is expected to be excavated in Rock Class II and III (Good to Fair) with occasional poor rock zones (Rock Class IV, V and VI). The weak zone at the contact of the Lamatar and Japantar Units is also expected to be excavated in Rock Class IV to VI.

4.4.4 Geotechnical Information of Powerhouse Complex

- The powerhouse and transformer caverns are proposed in quartz biotite gneisses with interbands of quartzite and marble belonging to Japantar Unit dissected by three plus random sets of joints, viz. Set J1: N041/39 (Foliation), J2: N234/73, and J3: N172/57.
- The intersected discontinuities are favorably oriented with the orientation of caverns.
- The portals of the main access tunnel and the cable-cum-ventilation tunnel may require slope stabilization.
- The powerhouse complex with its ancillary structures is expected to be excavated in Rock Class II and III (Good to Fair) with occasional poor rock zones (Rock Class IV, V and VI).

4.4.5 Geotechnical Information of Tailrace Tunnel

- The tailrace tunnel will be excavated through quartz biotite gneiss with marble bands and quartzite belonging to Japantar Unit. The rocks is expected to be dissected by three plus random sets of joints, viz. Set J1: N041/39 (Foliation), J2: N234/73, and J3: N172/57.
- The outlet portal of the tunnel is located in overburden of colluvium followed by rocks of quartzite with marble bands and quartz biotite gneiss. The portal may require nominal slope stabilization.
- The tailrace tunnel is expected to be excavated in Rock Class II and III (Good to Fair) with occasional poor rock zones (Rock Class IV, V and VI).

5. DESIGN AND CONSTRUCTION CONSIDERATIONS

This section describes the design and construction considerations relevant to the Works covered under LOT 1.

5.1 Temporary Access Roads and Bridges

The Contractor's scope includes construction of temporary roads and bridges for execution of the Works. The layout of the temporary roads and bridges is shown on the Infrastructure Plan included in *Volume V Employer's Concept Drawings (9.1 to 10.16)*

The temporary roads may be located on bedrock, river borne material or talus slope materials. The Contractor shall be responsible for slope stabilization and rock fall protection measures necessary for safe construction and use of the temporary access roads under this lot.

5.2 Temporary River Diversion

The baseline value of design flood for temporary river diversion during construction of the weir and appurtenant structures shall be taken as 501 m^3/s. The dimensioning and construction schedule of the temporary river diversion arrangement shall be the responsibility of the Contractor.

5.3 Earthworks and Site Preparation

The earthworks and site preparation shall entirely be left to the Contractor. Preliminary designs and drawings for these activities presented in the Bid Documents are for information only.

5.4 Open Cut Excavation

Open cut excavation for the headworks structures will require excavation and stabilization of the overburden and benching of the exposed rock by drilling and blasting along with face stabilization by dowels and post-tensioned end-anchored bolts. Minor shears and locally fractured /weak strata will have to be treated during the construction of the headworks structures.

The hill slopes above the portals will be excavated by benching mechanically or by drill and blast and stabilized with support measures including pattern bolting, welded wire mesh and shotcrete, as shown in approved construction drawings. Safety protections are to be taken as per the Safety Manual.

5.4.1 Slopes

Adequately designed slopes have to be provided in case of open excavations. The maximum slope for the rock excavation shall not exceed 1H: 10V.

5.4.2 Drill and Blast

The Contractor shall be responsible for the design of perimeter and production holes, their spacing and amount of charge.

5.4.3 Secondary Blasting

At times, excavation of the first bench may produce large rock blocks at the surface that may require additional (secondary) blasting operations. The problem may arise due to decompression of joints or bad loading of the collar of the hole. The Contractor shall ensure that the excavated material is sized for effective clearing. All time and costs associated with secondary blasting of the excavated material shall be to Contractor's account.

5.4.4 Profile Trimming

The Contractor shall be responsible for the time and costs associated with trimming of the perimeter after blasting and scaling, as required, to meet the required excavation profile.

5.4.5 Stability of Cut Face

Steeply dipping, open and partially open discontinuities will dip towards the excavations in various open cut faces. The Contractor is required to design the blast hole spacings and charge sizes to minimize overbreak and slippage of blocks into the excavation.

5.4.6 Support and Reinforcement

The supports shown on the approved construction drawings shall be the supports required for long term stability. Methods and materials for rock support are specified in *Volume II Employers Requirement*.

5.5 Underground Excavation

5.5.1 Drill and Blast

The Contractor shall be responsible for the blast design, blasting operation, mucking and safety. The Contractor shall be required to perform these works to meet the requirements in *Volume II Employers Requirement*.

It is strongly recommended that the Contractor carries out probe drilling for the tunnels in an advancement of at least 50 m to assess the rock condition beforehand to plan his equipment. The Contractor shall be responsible for the design and execution of production holes and shall plan his methods such that the excavated material size is compatible with his muck handling and haulage methods. The Contractor shall be responsible for any secondary blasting of excavated material.

5.5.2 Support and Reinforcement

In case of underground excavations, support systems shall be installed based on Q-values. The support shown on the approved construction drawings is the support required for long term stability. Supports to contain wedges will be the responsibility of the Contractor. Methods and materials for rock support are specified in *Volume II Employers Requirement*.

The Contractor should anticipate needing to retroactively install additional support behind the face in response to ground conditions and wedge failure observed along the tunnel crowns and walls. Any additional support installed behind the face due to deformations exceeding the specified limits will be subject to rock mass reclassification. The Contractor shall carry out any design and construction work required by that reclassification under the existing contract and the Employer shall have no additional financial liability.

5.5.3 Overbreak

Discontinuities that are sub-parallel to the tunnel drive would be unfavorable for stability and can be expected to increase the potential for excavation overbreak. Overbreak can also occur as a result of stress concentrations or removal of confinement. It can also occur where the perimeter holes are too close together for the rock mass conditions or where charges in the perimeter holes are too large for the rock mass condition. Even in competent rock mass conditions, a certain amount of overbreak may occur from the perimeter hole "look-out" angles for each round.

The Contractor shall design his drill and blast pattern to minimize overbreaks. No allowance or additional compensation shall be made for overbreaks in the GIR or other Contract Documents.

5.5.4 Underbreak

Underbreak may occur where drilling and blasting control is poor. All underbreaks shall be removed mandatorily by the Contractor at his own expense. No allowance for additional compensation or removing underbreak shall be made in the GIR or other Contract Documents.

6. ADDITIONAL GEOLOGICAL AND GEOTECHNICAL INVESTIGATIONS

Although subsurface explorations have been performed at several locations in the project area, additional exploration drilling for addressing some major geological concerns arising out of the foregoing geological and geotechnical assessment is considered necessary during the pre-construction phase of the project. As such, additional investigations are to be carried out by the Contractor to ensure equal or better than the intent and performance of the Employer's concept design. These investigations shall include, but not be limited to, the following:

- Further assessment of the weak zone identified at the contact of the Lamatar and Japantar Units and interpreted to intersect the pressure shaft is essential for proper planning of the pressure shaft construction. An inclined borehole drilling is recommended to intersect the weak zone to determine its thickness and nature of materials.
- Multi Channel Analysis of Surface Waves (MASW) and Microtremor Array Measurements (MAM) investigations are recommended to assess the bearing capacity of the foundation materials at the weir site.
- Assessment of the rock mass conditions at the level of the underground powerhouse and in-situ stresses is the area is necessary for optimizing the powerhouse and transformer cavern designs. As the powerhouse area is located in the Japantar Unit consisting of banded gneiss, quartzite, schist and marble, excavation of an exploratory Adit/ drift at the main access tunnel location is recommended for better delineation of the rock mass conditions at the powerhouse cavern site. Furthermore, in-situ

stress measurements have to be carried out for the finalizing the orientation and design of the powerhouse and transformer caverns.

- Assessment of the physio-mechanical properties of the rocks of the Lamatar and Japantar Units, such as shear parameters, elasticity modulus, Poisson's ratio, specific gravity, bulk density, porosity, unconfined compressive strength (UCS), etc., from core samples from the proposed drill holes and lump samples collected from the Adit/ exploratory drifts is recommended.
- One borehole drilling is recommended in the settling basin/ desilting chamber area.
- Two plate load tests are recommended at the weir site for assessing the bearing capacity of the foundation material.

SECTION B: GEOTECHNICAL DATA REPORT

DISCLAIMER
The purpose of the Geotechnical Data Report (GDR) is to document and summarize the surface and subsurface data collected for the project both during the preparation of its Feasibility and Updated Feasibility Reports and subsequent tests and investigations. The GDR presents and organizes the data for use by the Owner and its project management team and the Bidder and its design team for the purpose of a better understanding of the project and preparation of their bid. However, under no circumstances can this document be used for the preparation/ raising of claims on account of variation in the geological/ geotechnical ground conditions as given in these documents and actually encountered during project construction and execution.

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Abbreviations & Acronyms

ERT	Electrical Resistivity Tomography
FI	Flakiness Index
GIR	Geotechnical Information Report
GDR	Geotechnical Data Report
GKHEP	Ghunsa Khola Hydroelectric Project
IS	Indian Standards
LAA	Los Angles Abrasion
LL	Liquid Limit
NDH	New Drill Hole
NTP	New Test Pit
NW	Northwest
PL	Plastic Limit
SG	Specific Gravity
SS	Sulphate Soundness
TP	Test Pit
WA	Water Absoption

Units and Symbols

%	Percent
km	kilometer
km^2	square kilometer
kV	kilo Volt
l	Liter
Lu	Lugeon
m	meter
m^2	square meter
m^3	cubic meter
min	minute
mm	millimeter
MPa	Mega Pascal
s	second

Preamble

The Geotechnical Data Report (GDR) is a compilation of all the geological and geotechnical data gathered during the course of preparation of the feasibility report and the updated feasibility reports of GKHEP involving studies and tests carried out over a period of time. All the data gathered/ obtained are attached as Annexes to this document as detailed below:

Annex 1: Geological Drawings

Annex 2: Rock Mass Classification

Annex 3: Geophysical Surveys

Annex 4: Drill Core Logs

Annex 5: Water Pressure and Permeability Tests

Annex 6: Laboratory Tests on Rock Core Samples

Annex 7: Construction Material Investigations and Laboratory Testing

Annex 8: Petrographic Studies

1. INTRODUCTION

This GDR presents the results of geological and geotechnical investigations completed by various agencies for GKHEP. The investigations were performed in accordance with industry guidelines for assessing geological and geotechnical conditions of the project.

GKHEP has been extensively investigated through a wide range of investigation techniques to gather the requisite geological and geotechnical data and information for various components of the project. These techniques included the following.

- Regional geological studies in and around the project area.
- Engineering geological mapping.
- Exploratory core drilling.
- Geophysical surveys.
- In-situ (field) tests.
- Laboratory tests.
- Construction material surveys.

2. GEOLOGICAL AND GEOTECHNICAL INVESTIGATIONS

Geological and geotechnical investigations on GKHEP were conducted for the following objectives:

- To evaluate and optimize the layout of the different project components based on geological considerations.
- To collect sufficient qualitative and quantitative geological and geotechnical information for techno-economic design of the project components so that basic parameters for major structures could be optimized.
- To collect sufficient data to plan suitable construction methodologies and a reasonable cost estimate, etc.

A summary of the various investigations and their results are presented in the following sections.

2.1 Engineering Geological Mapping

Detailed geological mapping of the project area was carried out at a 1:5,000 scale. A larger scale mapping at 1:1,000 was performed for the diversion structure and powerhouse sites.

Detailed discontinuity mapping was performed in the headworks and powerhouse areas, along the tunnel alignment and at sites of potential land instability. The recorded discontinuity dataset had 520 records. Of these, 118 records were from the headworks site, 221 from the headrace tunnel area and 181 from the powerhouse area.

Rock mass classification was performed using two rock mass classification systems, namely the Geomechanics Classification or RMR System (Bieniawski, 1976) and the Rock Mass Quality Index or "Q" System (NGI, 2015). Using both systems, the studied rock mass was classified into five major types, viz. good, fair, poor, very poor and extremely poor. RMR and Q-values were determined at ten different locations in the project area. Details of the ratings for the individual parameters involved in each classification system, the RMR values and Q-values thus obtained and the corresponding rock mass classes are appended as Annex 2.

2.1.1 Geology of Project Area

In the project area, the Ghunsa Khola flows through a narrow valley, mostly with precipitous valley faces for which high escarpments are common on both banks (Figure 1). The area has a high relief of over 2,500 m, with ground elevations varying between 1,500 m at the tailrace tunnel outfall near the confluence of the Ghunsa Khola and Tamor River to over 4,000 m in the higher reaches of the valley slopes. Ghunsa Khola descends from an elevation of 1,940 m at the weir site to 1,501 m at the tailrace outfall in a distance of about 5.77 km and thus has a steep gradient of 1 in 13 in this reach. An approximately 20 m high waterfall forms a prominent landscape along the river course immediately downstream of the headworks complex (Photo 1).

The project area is located in a moderately upstream dipping rock succession informally divided into two distinct sequences – Lamatar Unit towards the upstream and Japantar Unit towards the downstream (Figure 2) While the Lamatar Unit is dominated by thick beds of garnetiferous banded gneisses, the Japantar Unit is represented by a mixed lithology of quartz biotite gneiss, quartzite and marble.

Figure 1: Narrow, V-shaped, NE-SW flowing Ghunsa Khola (Courtesy: Google Earth)

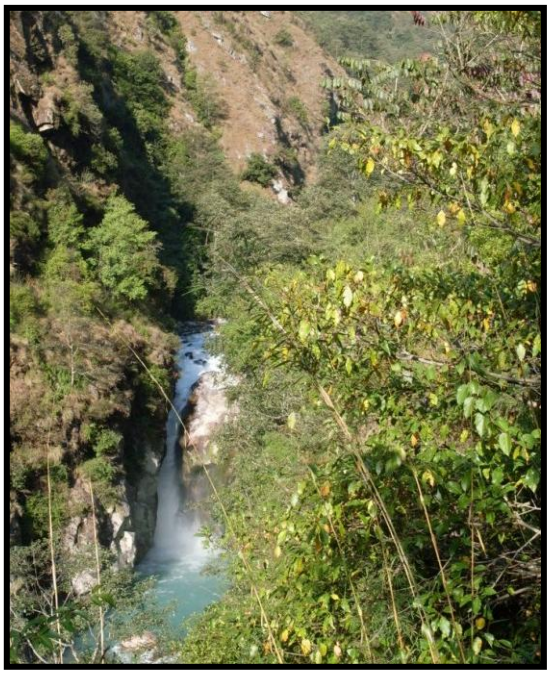

Photo 1: Prominent waterfall immediately downstream of weir site

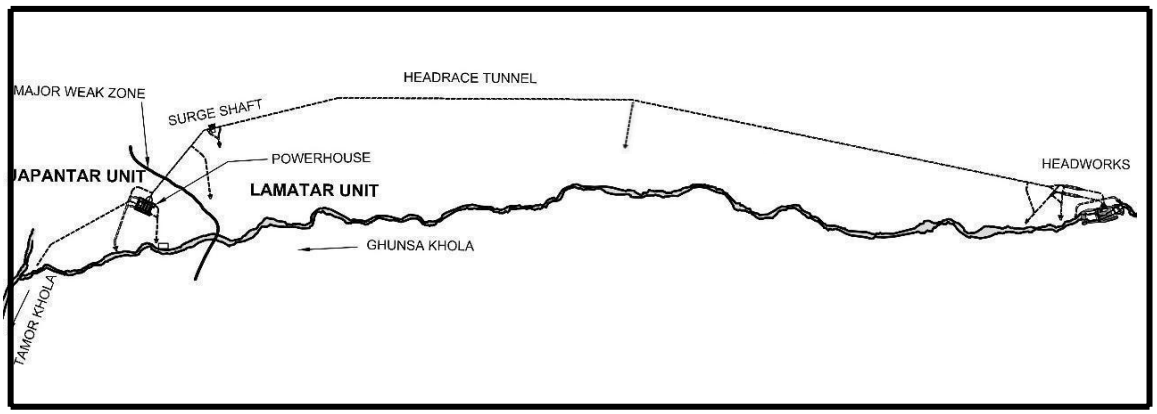

Figure 2: Disposition of Lamatar and Japantar units

The systematic discontinuity mapping and data analysis show the predominance of foliation joints designated as Joint Set J1 with dips of N037/40 (Figure 3). The rock is found dissected by three other major sets of joints designated as J2: N235/74, J3: N166/69 and J4: N300/56.

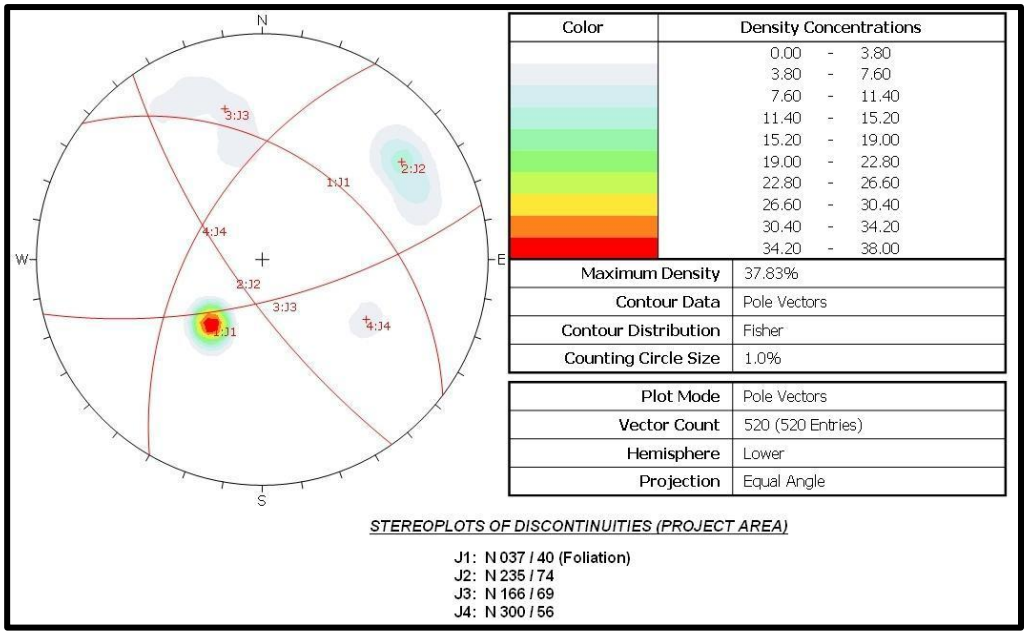

Figure 3: Stereoplot of discontinuities in project area (520 records)

The observed shear zones in the entire project area are interpreted to have preferential orientation along joint set J1 (N037/40), i.e., the foliation of the strata. Shear zones along joint set J2 (N235/74) are also found. The geological boundary between the Lamatar and Japantar Units is suspected to be a weak zone.

The geological maps and sections developed during the course of geotechnical investigations are included in the main tender drawing and listed in Table 1 for ready reference.

Table 1: List of geological drawings

SN	Description	Scale	Contour interval (m)	Drawing No.
1	Geological map of project area	1:7500	5 m	GK-GE-L-01
2	Geological profile along headrace tunnel	1:10000	NA	GK-GE-S-04
3	Geological map of headworks area	1:1500	5 m	GK-GE-S-02
4	Geological section along weir axis	1:750	NA	GK-GE-L-03
5	Geological map of powerhouse area	1:3000	5 m	GK-GE-L-05
6	Geological section along main access tunnel	1:2000	NA	GK-GE-S-06
7	Geological section along tailrace tunnel	1:4000	NA	GK-GE-S-07

2.2 Geophysical Surveys

Geophysical surveys were conducted at the proposed sites for the headworks, adit tunnels, surge shaft, underground powerhouse and tailrace tunnel. In all, 16 profile lines aggregating 4,350 m of two dimensional (2D) Electrical Resistivity Tomography (ERT) surveys were performed (Table 2). A detailed report on the geophysical surveys is appended with the GDR as Annex 3.

2.3 Exploratory Core Drilling

Exploratory core drilling was carried out through eight holes aggregating a total depth of 340 m. Of these holes, one hole of 40 m depth was drilled at the weir site, three holes aggregating 180 m depth in the powerhouse complex area, one hole of 30 m depth along the tailrace tunnel alignment and three holes of 30 m each at abutments of permanent bridges.

Detailed geological logging of the drill holes was carried out using standard procedures. The core logs, along with core box photographs, are appended in Annex 4. A summary of the drill holes is presented in Table 3.

Table 2: Details of 2D ERT survey coverage

Profile	Location	Length (m)
ERT-1	Left bank, across weir axis	250
ERT-2	Right bank, across weir axis	200
ERT-3	Right bank, across weir axis	200
ERT-4	Across river at headworks	250
ERT-5	Near Adit-1, across slope	200
ERT-6	Near Adit-2, along slope	250
ERT-7	Surge shaft (Option-3)	300
ERT-8	Powerhouse area	300
ERT-9	Powerhouse area	300
ERT-10	Surge shaft (Option-3)	300
ERT-11	Powerhouse	300
ERT-12	Powerhouse	300
ERT-13	Tailrace (Option-2)	300
ERT-14	Tailrace (Option-3)	300
ERT-15	Tailrace (Option-3)	300
ERT-16	Near Adit-2, along slope	300
	Total	**4,350**

Table 3: Details of drill holes

Drill hole	Location	Inclination	Total depth, m	Depth of bedrock, m	Remarks
NDH-1	Weir site, right bank	Vertical	40.0	23.4	Banded gneisses
NDH-2	Surge shaft	Vertical	40.0	-	Colluviums with fine and coarse rock fragments
NDH-3	Powerhouse site	Vertical	105.0	5.0	White marbles, calcareous gneisses
NDH-4	Flat area near powerhouse	Vertical	35.0	30.0	Gneisses, quartzites and marbles. Black carbonaceous material encountered at 19.5-20.5 m
NDH-5	Tailrace alignment	Vertical	30.0	13.5	Gneisses
DH-A	Tamor bridge, right abutment	Vertical	30	-	Alluvium with gravels and boulders of gneisses
DH-B	Ghunsa bridge, right abutment	Vertical	30	-	Gneisses and quartzites
DH-C	Ghunsa bridge, left abutment	Vertical	30	2.0	Banded gneisses
Total length			**340**		

2.4 Permeability Tests

Water pressure (Lugeon) tests were performed in the drill holes to estimate the permeability and tightness of the bedrock. A total of 35 water pressure tests were conducted in four drill holes, viz. NDH-1, NDH-3, NDH-5 and DH-C, where bedrock was encountered. The results of the tests are summarized in Table 4. Details are provided in Annex 5.

Likewise, permeability tests using the falling head method were performed in drill holes to estimate the permeability of alluvial and colluvial deposits. A total of 10 permeability tests were carried out in drill holes NDH-1, NDH-4 and DH-A. The results of the tests are summarized in Table 5. Details are provided in Annex 5.

Table 4: Summary of water pressure tests

SN	Drill hole	Location	Test depth, m	No. of tests	Lu, (representative)

1	NDH-1	Headworks	25.0 – 28.0	5	13.553
2			28.0 – 31.0	5	14.843
3			31.0 – 34.0	5	15.489
4			35.0 – 38.0	5	15.489
5	NDH-3	Powerhouse	6.0 – 9.0	5	34.632
6			9.0 – 12.0	5	8.032
7			13.0 – 16.0	5	44.131
8			16.0 – 19.0	5	27.491
9			19.0 – 22.0	5	22.653
10			22.0 – 25.0	5	22.018
11			25.0 – 28.0	5	13.763
12			28.0 – 31.0	5	16.563
13			31.0 – 34.0	5	14.371
14			34.0 – 37.0	5	15.028
15			37.0 – 40.0	5	14.897
16			40.5 – 43.5	5	15.053
17			44.0 – 47.0	5	14.853
18			47.0 – 50.0	5	15.745
19			51.0 – 54.0	5	15.459
20			54.0 – 57.0	5	14.554
21			57.0 – 60.0	5	14.916
22			60.0 – 63.0	5	13.630
23			63.0 – 66.0	5	12.987
24			67.0 – 70.0	5	12.971
25	NDH-5	Tailrace alignment	18.0 – 21.0	5	5.387
26			21.0 – 24.0	5	5.397
27			24.0 – 27.0	5	4.805
28			27.0 – 30.0	5	4.558
29	DH-C	Ghunsa bridge, left abutment	4.0 – 7.0	5	3.760
30			7.0 – 10.0	5	3.760
31			10.0 – 13.0	5	7.410
32			14.0 – 17.0	5	10.180
33			19.0 – 22.0	5	12.030
34			22.0 – 25.0	5	20.370
35			25.0 – 28.0	5	12.960

Table 5: Summary of permeability test results

SN	Drill hole	Location	Depth, m	Permeability (cm/s)
1	NDH-1	Headworks	2.0 – 3.0	1.41×10^{-3}
2			5.5 – 6.0	8.76×10^{-4}
3			8.0 – 9.0	1.06×10^{-3}
4			11.0 – 12.0	5.59×10^{-4}
5			15.0 – 16.0	9.79×10^{-4}
6			19.0 – 20.0	8.23×10^{-4}
7	NDH-4	Flat area near powerhouse	1.0 – 2.0	1.09×10^{-4}
8			4.0 – 5.0	4.13×10^{-5}
9			7.0 – 8.0	2.21×10^{-5}
10	DH-A	Tamor bridge, right abutment	10.0-12.0	2.15×10^{-5}

2.5 Dynamic Cone Penetration Test

Dynamic Cone Penetration Test (DCPT) was carried out at different depths in overburden material using a cone penetrator with a tip angle of 60° and a base area 10 cm^2. Cone driving was accomplished by a trip hammer weighing 63.5 kg falling freely through a distance of 750 mm onto the drive head fitted at the top of drill rods.

In all, seven DCPTs were conducted. Of these, three tests were conducted in drill hole NDH-1, one in NDH-2 and three in NDH-5. A summary of results of DCPTs is given in Table 6, and details are presented in the core logs in Annex 4.

Table 6: Summary of DCPT tests in overburden material

S.N.	Drill hole	Depth (m)		N value
		From	To	
1	NDH-1	0.0	1.0	70
2		6.0	7.0	Refusal
3		15.0	16.0	42
4	NDH-2	14.0	16.0	Refusal
5	NDH-5	1.5	2.5	Refusal
6		5.0	6.0	57

2.6 Laboratory Tests on Core Specimen

Laboratory tests on core samples comprised point load tests, uniaxial compressive strength tests and petrographic tests. These tests were conducted in accordance with standard methods and procedures.

Point load tests were performed on 15 samples from five drill holes. The results of the tests are summarized in Table 7. Details are provided in Annex 6.

Table 7: Summary of point load tests on core samples

SN	Drill hole	Depth, m	Test method	Is (50), MPa
1	NDH-1	26 – 28	Axial	5.96
2		38 – 40	Axial	8.74
3	NDH-3	8 – 9.15	Axial	6.35
4		10.3 – 11.5	Axial	6.30
5		34 – 36	Axial	12.38
6	DH-A	18 – 19	Axial	3.66
7			Diametral	3.13
8	DH-B	24 – 27	Axial	5.57
9			Diametral	1.98
10		27 – 30	Axial	3.86
11			Diametral	4.83
12	DH-C	7 – 8	Axial	4.73
13			Diametral	4.27
14		22 – 25	Axial	3.26
15			Diametral	4.15

Uniaxial compressive strength tests were conducted on three samples from NDH-1, six from NDH-3 and two each from DH-B and DH-C. Detailed tests results are presented in Annex 6 and are summarized in Table 8.

Table 8: Summary of uniaxial compressive strength tests on core samples

SN	Drill hole	Depth, m	Result, MPa
1	NDH-1	12.00 – 14.00	35.45
2		26.88 – 28.00	25.69
3		38.00 – 40.00	31.69
4	NDH-3	8.00 – 9.15	23.39
5		10.30 – 11.50	50.07
6		22.00 – 23.75	37.36
7		28.00 – 29.50	33.55
8		30.80 – 32.50	56.69
9		34.00 – 36.00	32.04
10	DH-B	24.00 - 27.00	48.66
11		27.00 – 30.00	83.69
12	DH-C	7.00 – 8.00	33.09
13		22.00 – 25.00	99.26

Petrographic studies were conducted on seven samples, two each from NDH-1 and NDH-4 and three from NDH-3. The results of these studies are summarized in Table 9. Details are presented in Annex 8.

Table 9: Summary of petrographic studies

SN	Drill hole	Depth, m	Mineral assemblage %	Rock type

1	NDH-1	12.0 – 14.0	Felspars 30, mica 30, quartz 20, garnet 10, others 10	Garnet-Mica Rich Augen Gneiss
2	NDH-1	26.0 – 28.0	Felspars 50, quartz 15, mica 15, garnet 15, others 5	Garnet rich Para-gneiss
3	NDH-3	8.0 – 9.15	Quartz 40, mica 35, Calcite 15, others 10	Calcareous – Quartz - Gneiss
4	NDH-3	10.3 – 11.5	Felspars 35, calcite 30, quartz 25, Mica 5, others 5	Calcareous – Quartz - Gneiss
5	NDH-3	42.0 – 43.5	Quartz 90, felspars 5, calcite 3, others 2	Green Quartzite
6	NDH-4	19.5 – 20.5	Mica 37, quartz 30, felspars 14, ferruginous material 10, others 9	Fault gauge
7	NDH 4	30.8 – 32.5	Quartz 45, felspars 15, mica 15, others including metallic minerals 25	Metasandstone with mica partings and metallic minerals

2.7 Construction Material Investigations

Prospecting for construction materials for the project, viz. fine aggregates, coarse aggregates and cohesive materials, was carried out at nine test pits and three borrow pits at different locations within the project area (Table 10). Laboratory tests were conducted on representative samples from the tests and borrow pits to assess their engineering properties. The results of the tests are summarized in Table 11. Details are presented in Annex 7.

Table 10: Details of test and borrow pits

Pit	Location	Material	Depth(m)	Usage
TP-1	Opposite cable-cum-ventilation tunnel portal, Japantar, left bank	Alluvium	3	Aggregates
TP-2	Opposite pothead yard, Japantar, left bank	Alluvium	3	Aggregates
TP-3	Near pothead yard, Japantar, left bank	Alluvium	3	Aggregates
TP-4	Near pothead yard, Japantar, left bank	Alluvium	3	Aggregates
TP-5	Near Adit 2 portal, Lamatar, left bank	Alluvium	3	Aggregates
TP-6	Near weir axis, Lamatar, left bank	Colluvium	3	Cohesive material
TP-7	Headworks site, Lamatar, left bank	Colluvium	3	Cohesive material
TP-8	Headworks site, Lamatar, left bank	Colluvium	3	Cohesive material
NTP-1	Sekathum, right bank	Alluvium	1.5	Aggregate
NTP-2	Sekathum, right bank	Alluvium	1.5	Aggregate
NTP-3	Weir site, Lamatar, left bank	Alluvial fan	3	Aggregates
NTP-4	Weir site, Lamatar, left bank	Alluvial fan	3	Aggregates

In addition to laboratory tests, petrography studies on five samples of coarse aggregates and four samples of fine aggregates from the test pits were carried out. The results of the studies are summarized in Table 12, and details are presented in Annex 8.

Table 11: Results of laboratory tests on test and borrow pit samples

Source		Unit weight (kg/m^3)		Gradation (%)			SG	WA (%)	LAA (%)	SS (%)	Alkali reactivity test (mmole/l)		ACV (%)	AIV (%)	FI (%)	EI (%)	Atterberg Limit			Modified Proctor Compaction Test	
		Loose	Compacted	Gravel	Sand	Silt/ clay					Reduction in alkalinity	Reactive Silica					LL (%)	PL (%)	PI (%)	OMC (%)	$\gamma_{d,max}$ (gm/cc)
TP-1		1656	1898	55.11	44.50	0.39	2.61	0.94	26.20	1.26	62.90	18.2	-	-	-	-	-	-	-	-	-
TP-2		1602	1833	45.45	45.4	9.15	2.59	0.67	34.00	1.10	55.30	14.5	-	-	-	-	-	-	-	-	-
TP-3		1551	1820	34.54	65.31	0.15	2.55	0.75	29.80	2.87	60.40	14.3	-	-	-	-	-	-	-	-	-
TP-4		1785	2023	58.65	37.11	4.24	2.63	0.93	32.00	1.37	47.80	16.5	-	-	-	-	-	-	-	-	-
TP-5		1146	1358	2.18	97.66	0.16	2.55	1.00	-	2.35	45.30	19.9	-	-	-	-	-	-	-	-	-
TP-6		-	-	64.69	28.59	6.72	-	-	-	-	-	-	-	-	-	-	30.8	0	30.8	9.52	2.03
TP-7		-	-	64.14	28.54	7.32	-	-	-	-	-	-	-	-	-	-	26.0	0	26.0	9.77	2.11
TP-8		-	-	59.75	30.20	10.05	-	-	-	-	-	-	-	-	-	-	27.5	0	27.5	8.50	2.12
NTP-1	C	-	2003	54.25	45.65	0.10	2.60	1.60	42.78	-	34	7.6	29.23	13.75	17.82	54.65	-	-	-	-	-
	F						2.60				49	4.9									
NTP-2	C	-	2055	49.60	49.58	0.82	2.65	1.30	28.84	-	49	5.8	28.24	20.73	15.12	47.16	-	-	-	-	-
	F						2.61				54	7.2									
NTP-3	C	1669	1828	47.56	51.99	0.45	2.61	1.40	42.15	-	70	4.7	-	18.65	13.05	14.06	-	-	-	-	-
	F						2.68				187	7.1									
NTP-4	C	1586	-	6.70	92.64	0.66	2.62	1.00	-	-	-	-	-	-	-	-	-	-	-	-	-
	F										194	6.8									
Limit A[7]								*<2*	*≤ 30*	*≤10*		-	*≤ 30*	*≤ 30*	*FI+EI ≤ 40*		-	-	-	-	-
Limit B[8]									*≤ 50*			-	*≤ 45*	*≤ 45*			-	-	-	-	-

SG: Specific Gravity
WA: Water Absorption
LAA: Los Angeles Abrasion
SS Sulphate Soundness
ACV: Aggregate Crushing Value
AIV Aggregate Impact Value
FI: Flakiness Index
EI: Elongation Index
LL: Liquid Limit
PL: Plastic Limit
PI: Plasticity Index
OMC: Optimum moisture content
γ_{dmax} Maximum dry density
C Coarse aggregates
F Fine aggregates

[7] For wearing course (tunnel lining, spillway, etc.) as per IS 383: 2016 – Coarse and Fine Aggregates for Concrete – Specifications (Third Revision)
[8] For concrete other than in wearing course, per IS 383: 2016 – Coarse and Fine Aggregates for Concrete – Specifications (Third Revision)

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Table 12: Summary of petrographic studies on test pit samples

SN	Location	Mineral assemblage (%)	Rock type
1	NTP-1	Quartz 91, garnet 4, mica 3, others 2	Quartzite
2	NTP-1	Felspars: 50, quartz 20, mica 15, others 15	Ortho-Gneiss
3	NTP-1	Felspars 60, quartz 20, mica 15, others 5	Felspathic gneiss
4	NTP-2	Felspars 50, quartz 20, mica 15, others 15	Ortho-Gneiss
5	NTP-2	Felspars 60, quartz 20, mica 15, others 5	Felspathic gneiss
6	NTP-1	Quartz 40, mica 24, felspars 13, rock fragments 21, others 2	Sand
7	NTP-2	Quartz 44, mica 21, felspars 16, rock fragments 17, others 2	Sand
8	NTP-3	Quartz 49, mica 30, felspars 16, rock fragments 4, others 1	Sand
9	NTP-4	Felspars 46, quartz 30, mica 9, rock fragments 3, others 12	Sand

The results of the laboratory tests in Table 11 and Table 12 show the following:

a. The loose unit weights of the tested samples vary between 1,146 to 1,785 kg/m^3 while the compacted unit weights vary between 1,358 to 2,055 kg/m^3.
b. The aggregate samples from TP-1 to TP-4 and NTP-1 to NTP-3 are well-graded sandy gravels while those from TP-5 and NTP-4 are poorly graded sandy. The cohesive materials from the borrow pits (TP-6 to TP-8) consists of clays mixed with sands and gravels.
c. The specific gravity of the aggregate samples varies between 2.55 and 2.68, which is within the desirable range of 2.4 to 2.9 for concrete aggregates.
d. The water absorption of all aggregate samples is less than the desirable limit of 2%.
e. The Los Angeles Abrasion value of the samples ranges from 26.2 to 42.78%. These values are within the permissible upper limit of 30% for wearing courses (spillway surface, tunnel lining, etc.) and 50% for concrete to be used elsewhere.
f. The sulphate soundness values of the samples range between 0.112% and 2.87%, which is significantly lower than the permissible limit of 10%.
g. The reduction in alkalinity and the quantity of reactive silica obtained from the alkali reactivity tests shows that the aggregates are innocuous, i.e., they are not deleterious.
h. The crushing values of coarse aggregates are around 29%, less than the permissible limit of 30% for wearing courses and 45% for concrete to be used elsewhere.
i. The impact values of coarse aggregates are between 13.75% and 20.73%, less than the permissible limit of 30% for concrete in wearing courses and 45% for other concrete.
j. The sum of the Flakiness Index and Elongation Index of the tested coarse aggregate samples is less than the allowable limit of 40%.

2.7.1 Reserve Estimate

The quantities of coarse and fine aggregates available in the headworks and powerhouse areas were estimated assuming a depth of excavation of 3 m only. Considering the results of the sieve analysis conducted on the test pit specimen, a ratio of 60:40 was assumed for the coarse and fine aggregates available from these deposits. The volume of available aggregates was estimated by dividing the deposits into different blocks. The resulting reserve estimate is presented in Table 13.

The total estimated quantity of construction material available from the different identified quarry sites is about 307,800 m^3. Of this, the quantities of coarse and fine aggregates are estimated to be 184,680 m^3 (60% of the total) and 123,120 m^3 (40% of the total), respectively.

Coarse aggregates can also be obtained from the estimated 316,475 m^3 of muck excavated from underground excavations. This may help reduce excavation of the riverbed alluvium as aggregate. However, prior to its use, the excavated muck will have to be tested for its suitability for use as concrete aggregate. Prima facie, quartzite and quartz biotite gneisses with acceptable limits of deleterious mica content could be suitable.

Table 13: Estimated aggregate reserves in identified quarry sites

Site	Ave. length (m)	Ave. width (m)	Area (m^2)	Depth (m)	Total volume, m^3	Coarse aggregate, m^3 (60%)	Fine aggregate, m^3 (40%)
TP-1	200	50	10,000	3	30,000	18,000	12,000
TP-2	150	60	9,000	3	27,000	16,200	10,800
TP-3	150	30	4,500	3	13,500	8,100	5,400

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TP-4	170	70	11,900	3	35,700	21,420	14,280
TP-5	100	50	5,000	3	15,000	9,000	6,000
TP-6	180	70	12,600	3	37,800	22,680	15,120
TP-7	150	50	7,500	3	22,500	13,500	9,000
TP-8	160	50	8,000	3	24,000	14,400	9,600
NTP-1	40	40	1,600	3	4,800	2,880	1,920
NTP-2	220	40	8,800	3	26,400	15,840	10,560
NTP-3	140	80	11,200	3	33,600	20,160	13,440
NTP-4	250	50	12,500	3	37,500	22,500	15,000
				Total	**307,800**	**184,680**	**123,120**

The estimated quantities of coarse and fine aggregates required for construction of the project are about 111,300 m^3 and 60,135 m^3, respectively. Therefore, the total construction materials available from the river deposits and excavated muck would be more than sufficient.

Annex 1: Geological Drawings

	STEREOPLOT OF DISCONTINUITIES			
AREA	PROJECT AREA	HEADWORKS AREA	HEADRACE TUNNEL	POWERHOUSE AREA
MAJOR DISCONTINUITIES SETS	J1: N037°/40° (FOLIATION) J2: N235°/74° J3: N169°/69° J4: N300°/58°	J1: N033°/43° (FOLIATION) J2: N245°/73° J3: N154°/75° J4: N300°/56°	J1: N037°/38° (FOLIATION) J2: N246°/73° J3: N155°/73° J4: N301°/55°	J1: N041°/39° (FOLIATION) J2: N234°/73° J3: N172°/57°

NOTES:

1. GEOLOGICAL MAP IS BASED ON DETAILED GEOLOGICAL TRAVERSES ALONG THE KANCHENJUNGA BASE CAMP (KBC) MULE TRACK AND LOCAL FOOTPATHS.
2. MAJOR LITHOUNITS PLOTTED REPRESENT PREDOMINANT ROCK TYPES. INTERCALATIONS OF OTHER ROCK TYPES, E.G. PEGMATITE AND AUGEN GNEISS IN GNEISSES (GBG) AND QUARTZITE AND MARBLE IN GNEISSES (GQM), MAY BE EXPECTED.
3. THE ONLY MAJOR GEOLOGICAL CONTACT BETWEEN LAMATAR UNIT AND JAPANTAR UNIT HAS BEEN INTERPRETED AS POSSIBLE TECTONIC BASED ON EXPOSED CARBONACEOUS MATERIAL ON LEFT BANK HILL SLOPE OF GHUNSA KHOLA & IN DRILL HOLE NDH-4 ALONG WITH GEOMORPHIC BREAK IN SLOPE AND HAS BEEN SHOWN AS CONCEALED ON RIGHT BANK SLOPES.
4. VALLEY SLOPES ARE VEGETATED AND INACCESSIBLE, AND ONLY MAJOR COLLUVIUM AND ROCK OUTCROPS ARE SHOWN. AREAS MARKED WITH COLLUVIUM MAY HAVE ISOLATED ROCK OUTCROPS, SIMILARLY, AREAS SHOWN AS ROCK MAY HAVE ISOLATED POCKETS OF COLLUVIUM.
5. INDIVIDUAL ROCK SYMBOLS HAVE BEEN USED ONLY FOR ACCESSIBLE ROCK OUTCROPS THAT HAVE BEEN PHYSICALLY VERIFIED. LITHOLOGICAL SYMBOLS FOR SMALL OUTCROPS ARE SHOWN IN BOXES.
6. WHEREVER POSSIBLE WITH A HIGH DEGREE OF CONFIDENCE, ROCK STRUCTURE DATA IN INACCESSIBLE REACHES HAVE BEEN MEASURED FROM A DISTANCE AND USED AS REGULAR DATA.

NORTH

GEOLOGICAL MAP OF PROJECT AREA

CONTOUR INTERVAL 5 M

SCALE: 1:2,000

GHUNSA KHOLA HEP TAPLEJUNG, NEPAL	
REMIT HYDRO LIMITED KATHMANDU, NEPAL	
GEOLOGY GEOLOGICAL MAP OF PROJECT AREA	
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	In of Indo Canadian Consultancy Services Ltd. & Shah Consult International (P.) Ltd. THAPATHALI, KATHMANDU, NEPAL PHONE: 01-5106166, FAX:01-5011886
DRAWN BY I. A. ANSARI	APPROVED BY S. BHUSHAN
DATE: 2025	DWG. NO GK-GE-L-01
SCALE: NTS	

DWG. NO. DESCRIPTION:
REFERENCE DRAWINGS

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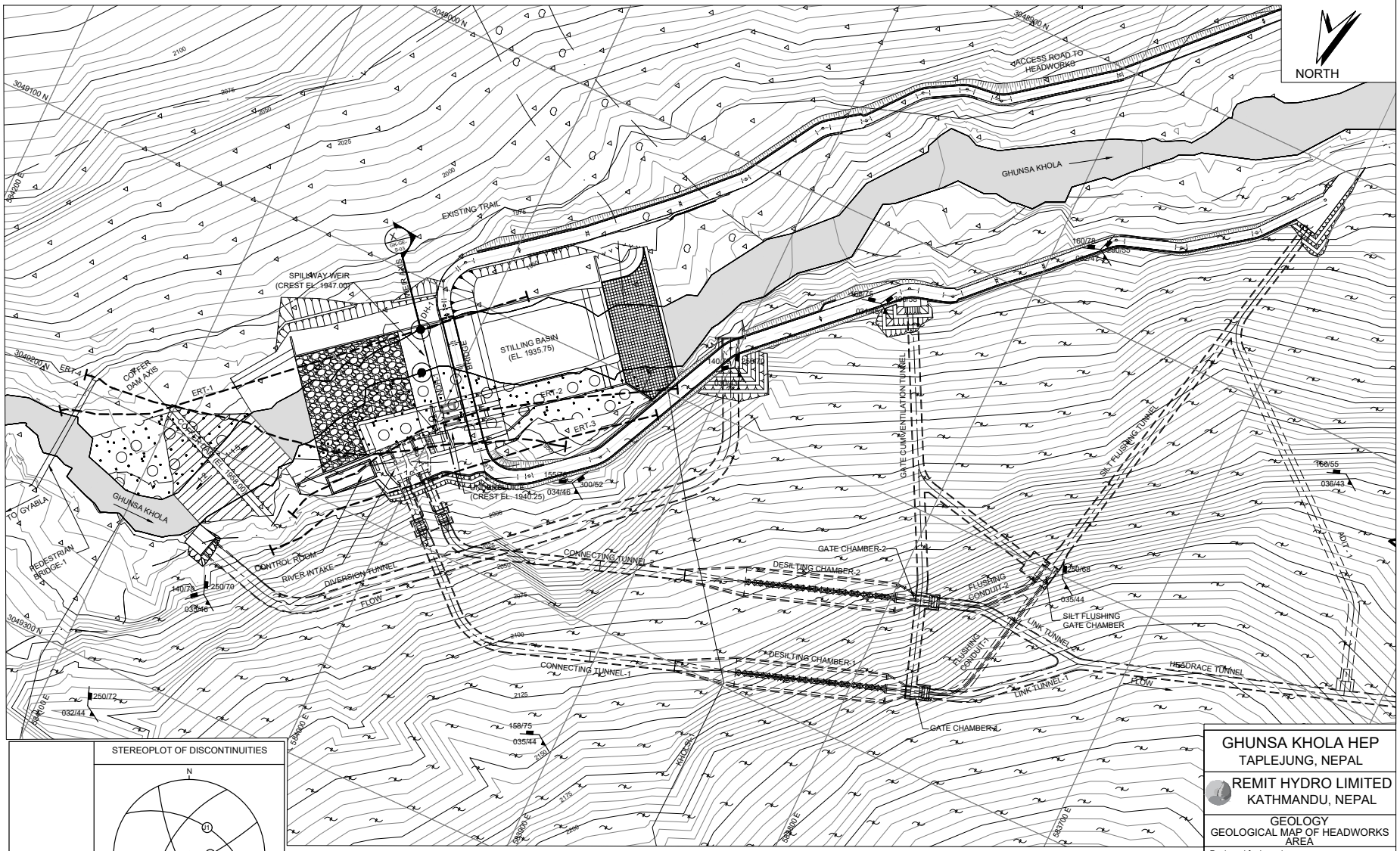

GEOLOGICAL MAP OF HEADWORKS AREA

NOTES:

1. MAJOR PLOTTED LITHOUNITS REPRESENT PREDOMINANT ROCK TYPES. INTERCALATIONS OF OTHER ROCK TYPES, e.g. PEGMATITES AND AUGEN GNEISSES IN GARNETIFEROUS BANDED GNEISSES (GBG), MAY BE EXPECTED.
2. WHEREVER POSSIBLE WITH A HIGH DEGREE OF CONFIDENCE, ROCK STRUCTURE DATA IN INACCESSIBLE REACHES HAVE BEEN MEASURED FROM A DISTANCE AND USED AS REGULAR DATA. VERY WELL EXPOSED BEDROCK, PARTICULARLY GNEISSES, IN VALLEY SLOPES/ ESCARPMENTS HAS FACILITATED SUCH ROCK STRUCTURE MEASUREMENTS.

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	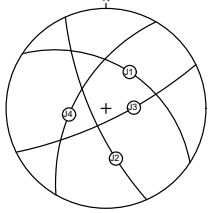
MAJOR DISCONTINUITIES SETS	J1: N033°/43° (FOLIATION) J2: N246°/73° J3: N154°/76° J4: N300°/56°

CONTOUR INTERVAL 5 M

15 0 15 30 45 60 75M

SCALE : 1:1,500

GK-GE-S-03	GEOLOGICALSECTION ALONG WEIR AXIS
DWG. NO.	DESCRIPTION:
	REFERENCE DRAWINGS

GHUNSA KHOLA HEP
TAPLEJUNG, NEPAL

REMIT HYDRO LIMITED
KATHMANDU, NEPAL

GEOLOGY
GEOLOGICAL MAP OF HEADWORKS AREA

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SCALE: NTS	

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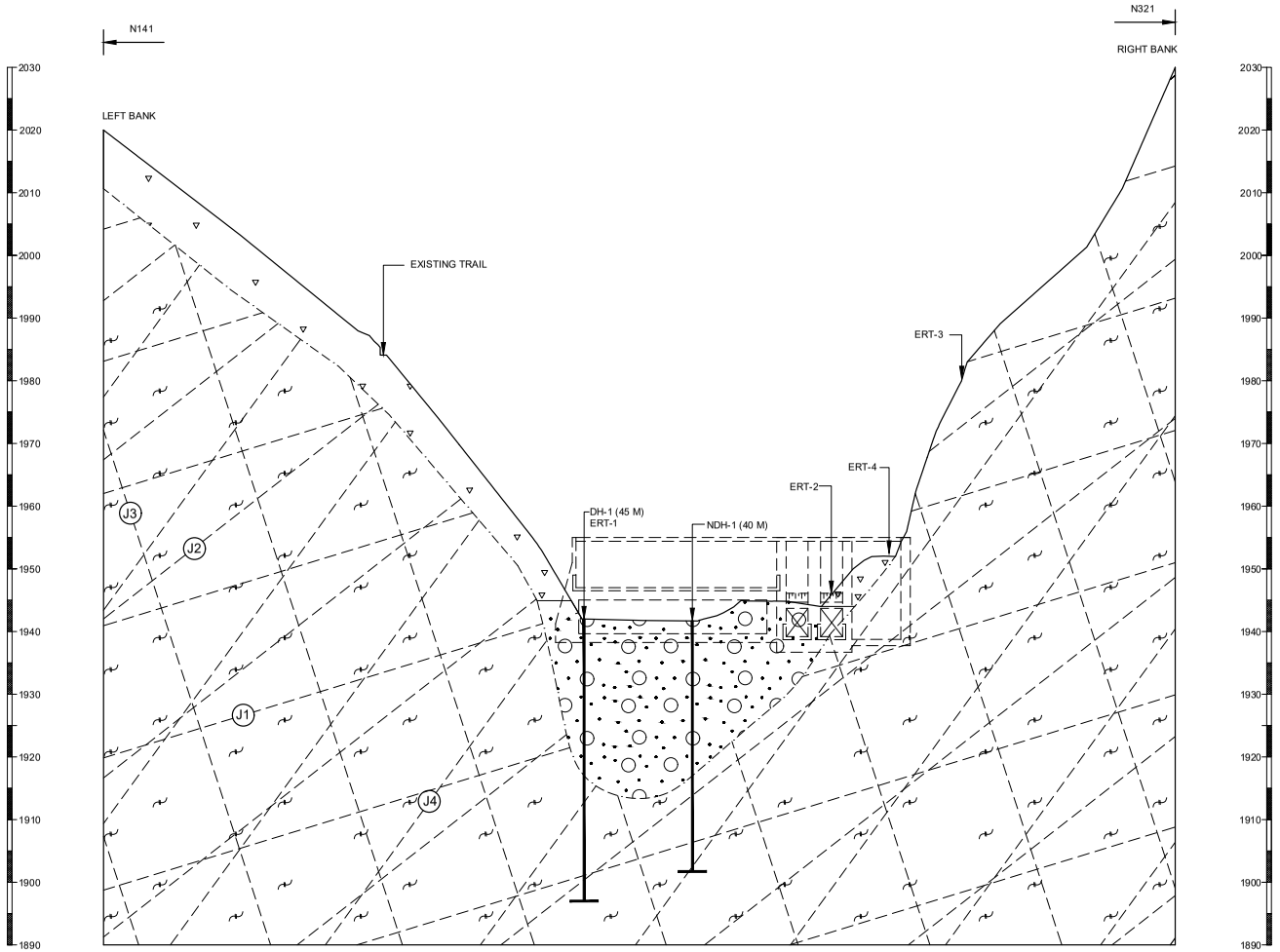
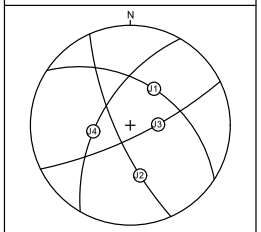


GEOLOGICAL SECTION ALONG WEIR AXIS

SECTION X (GK-GE-L-02)

STEREOPLOT OF DISCONTINUITIES	
	
MAJOR DISCONTINUITIES SETS	J1: N033°/43° (FOLIATION) J2: N246°/73° J3: N154°/76° J4: N300°/56°

NOTES:

1. GEOLOGICAL SECTION IS BASED ON GEOLOGICAL MAP OF HEADWORKS AREA.
2. PLOTTED JOINTS ARE SCHEMATIC AND NOT AS PER THEIR ACTUAL SPACING.

7.5 0 7.5 15 22.5 30 37.5M
SCALE: 1:750

GK-GE-L-02	GEOLOGICAL MAP OF HEADWORKS AREA
DWG. NO.	DESCRIPTION:
REFERENCE DRAWINGS	

GHUNSA KHOLA HEP
TAPLEJUNG, NEPAL

REMIT HYDRO LIMITED
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GEOLOGICAL SECTION ALONG WEIR AXIS

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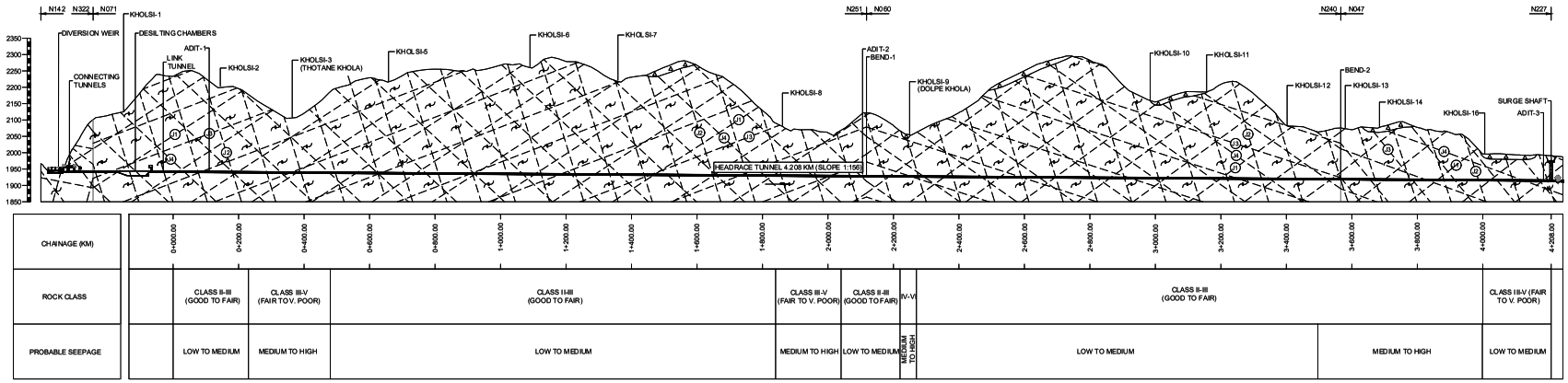
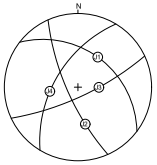
SCALE: NTS	



GEOLOGICAL PROFILE ALONG HEADRACE TUNNEL

	STEREOPLOT OF DISCONTINUITIES
	
MAJOR DISCONTINUITIES SETS	J1: N037°/08° (FOLIATION) J2: N246°/73° J3: N155°/75° J4: N301°/55°

ROCK CLASS

CLASS	CATEGORY	'Q'-VALUE
I	VERY GOOD	40-100
II	GOOD	10-40
III	FAIR	4-10
IV	POOR	1-4
V	VERY GOOD	0.1-1
VI	EXTREMELY POOR	0.01-0.1

SEEPAGE

LOW (L):	<2.5 L/MIN.
MEDIUM (M):	<2.5-125 L/MIN.
HIGH (H):	>125 L/MIN.

NOTES:

1. GEOLOGICAL SECTION IS BASED ON GEOLOGICAL MAP OF PROJECT AREA.
2. GEOLOGICAL SECTION HAS BEEN SIMPLIFIED AND ONLY MAJOR LITHOUNIT GARNETIFERROUS BANDED GNEISS (GBG) HAS BEEN SHOWN.
3. THICKNESS OF COLLUVIUM ALONG HEADRACE TUNNEL ALIGNMENT IS TENTATIVE.
4. ROCK CLASS SHOWN ARE BASED ON SURFACE STUDIES AND HENCE ARE ONLY REPRESENTATIVE. POCKETS OF INFERIOR ROCK CLASSES (V & VI) ARE EXPECTED IN ROCK CLASS OF POOR TO GOOD QUALITY.
5. PLOTTED JOINTS ARE SCHEMATIC AND NOT AS PER THEIR ACTUAL SPACING.

100 0 100 200 300 400 500M
SCALE : 1:10,000

DWG. NO.	DESCRIPTION:
	REFERENCE DRAWINGS

REMIT HYDRO रेमिट हाइड्रो लिमिटेड

GHUNSA KHOLA HEP
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REMIT HYDRO LIMITED
KATHMANDU, NEPAL

GEOLOGY
GEOLOGICAL PROFILE ALONG HEADRACE TUNNEL

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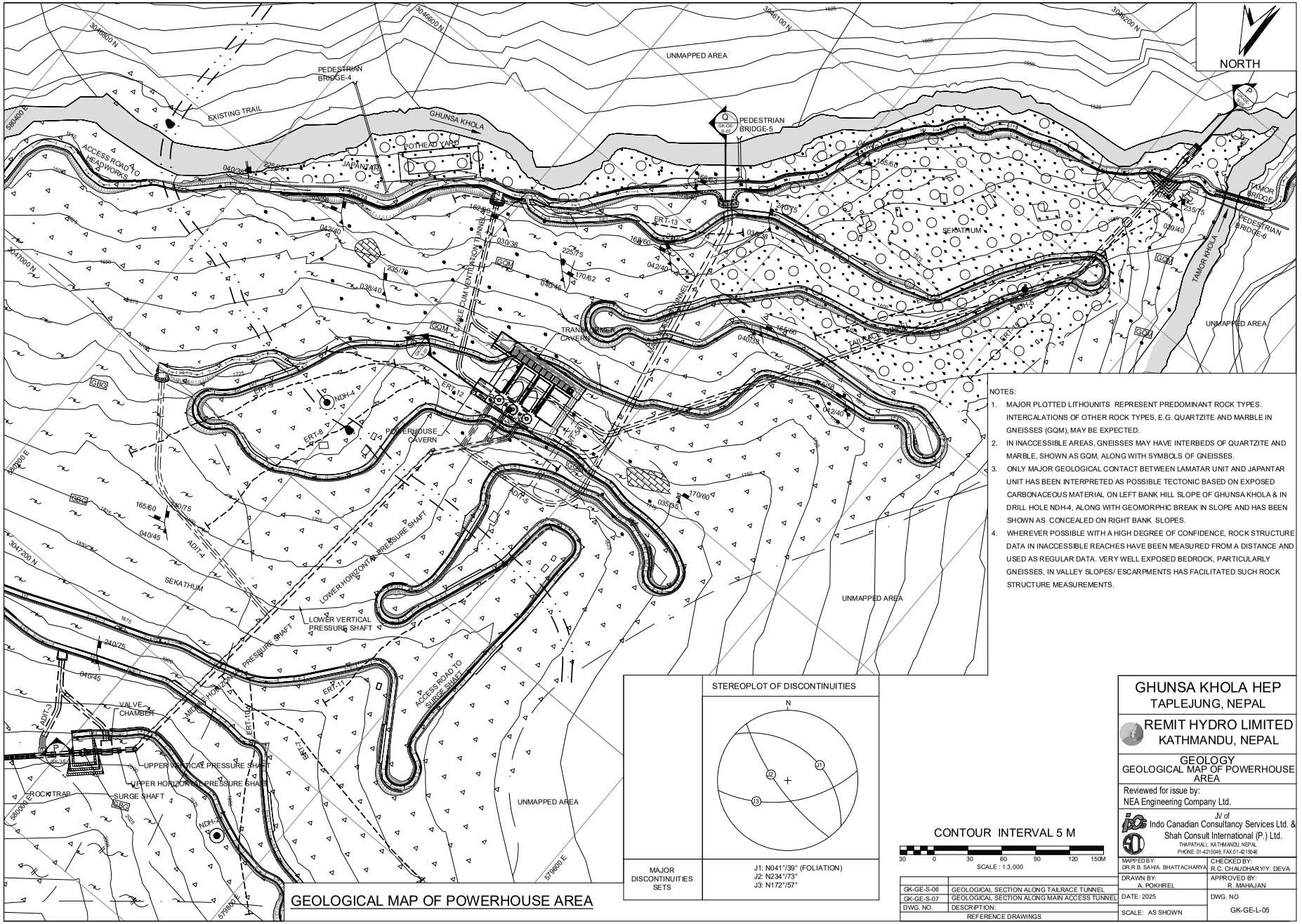
SCALE: NTS	


NORTH
UNMAPPED AREA
PEDESTRIAN BRIDGE-4
PEDESTRIAN BRIDGE-5
PEDESTRIAN BRIDGE-6
EXISTING TRAIL
GHUNSA KHOLA
TAMOR KHOLA
TAMOR BRIDGE
ACCESS ROAD TO HEADWORKS
POTHEAD YARD
JAPANTAR
SEKATHUM
TRANSFORMER CAVERN
POWERHOUSE CAVERN
TAILRACE TUNNEL
TAILRACE OUTLET
MAIN ACCESS TUNNEL
ADIT-1
ADIT-2
ADIT-3
ADIT-4
ADIT-5
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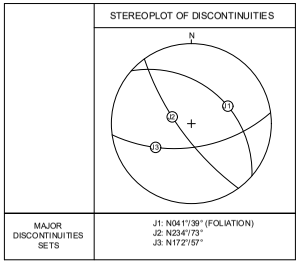

NOTES:

1. GEOLOGICAL SECTION IS BASED ON GEOLOGICAL MAP OF POWERHOUSE AREA.
2. PLOTTED JOINTS ARE SCHEMATIC AND NOT AS PER THEIR ACTUAL SPACING.

CHAINAGE (KM)	0+000.00	0+200.00	0+400.00	0+600.00	0+800.00	1+000.00	1+200.00	1+400.00	1+600.00	1+800.00	2+000.00	2+200.00	2+400.00	2+600.00	2+800.00	3+000.00	3+100.00
ROCK CLASS	CLASS II-III (GOOD TO FAIR)																
PROBABLE SEEPAGE	LOW																

GEOLOGICAL PROFILE ALONG MAIN ACCESS TUNNEL

SECTION Q (GK-GE-L-05)

SCALE: 1:2,000

GK-GE-L-05	GEOLOGICAL MAP OF POWERHOUSE AREA
DWG. NO.	DESCRIPTION:
REFERENCE DRAWINGS	

GHUNSA KHOLA HEP
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REMIT HYDRO LIMITED
KATHMANDU, NEPAL

GEOLOGY
GEOLOGICAL SECTION ALONG MAIN ACCESS TUNNEL

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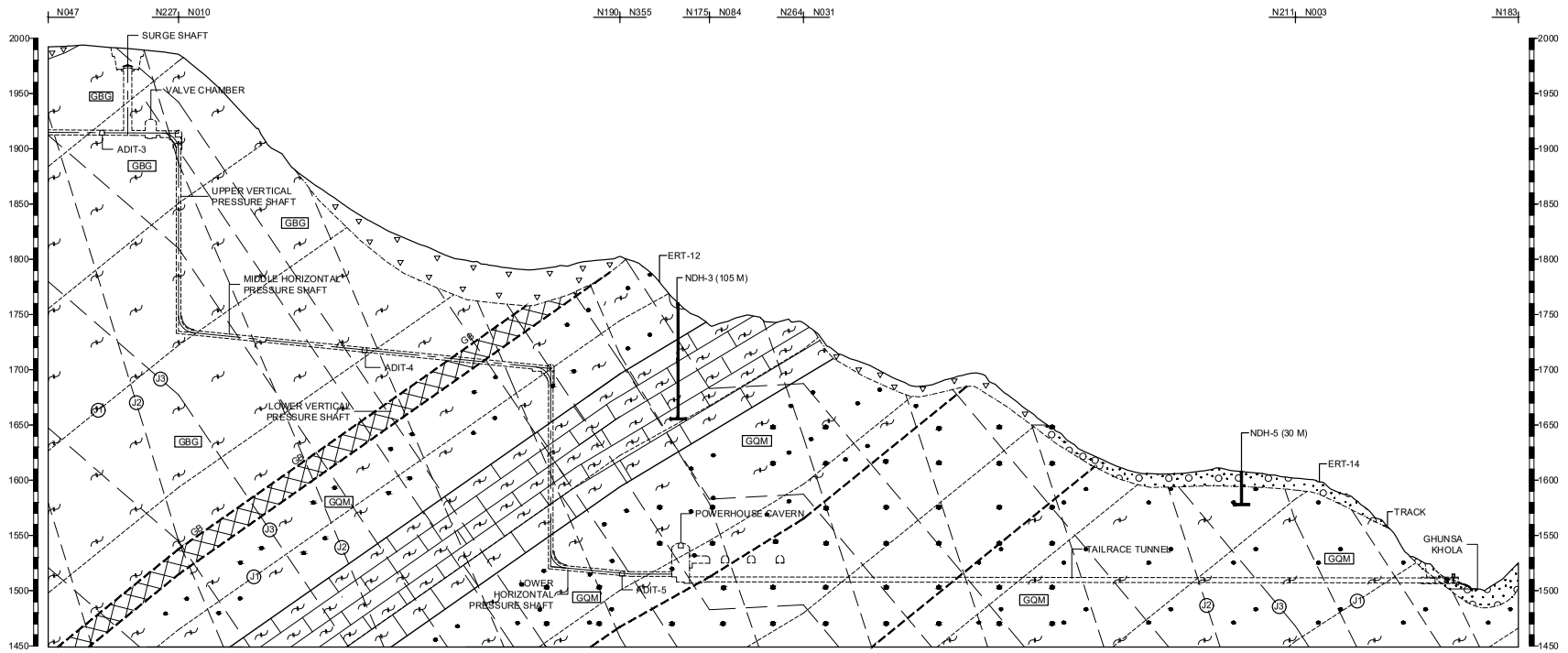
DATE: 2025	DWG. NO GK-GE-S-06
SCALE: NTS	



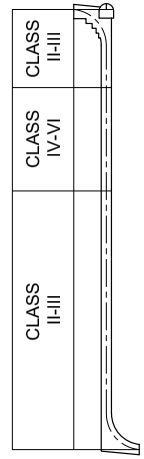
GEOLOGICAL PROFILE ALONG TAILRACE TUNNEL

SECTION Q (GK-GE-L-05)

SCALE A

UPPER VERTICAL PRESSURE SHAFT

SCALE B



LOWER VERTICAL PRESSURE SHAFT

SCALE B

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	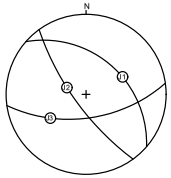
MAJOR DISCONTINUITIES SETS	J1: N041°/39° (FOLIATION) J2: N234°/73° J3: N172°/57°

NOTES:

1. SECTION IS BASED ON THE GEOLOGICAL MAP OF POWERHOUSE AREA.
2. OVERBURDEN THICKNESS IS BASED ON RESULTS OF DRILL HOLES.
3. WEAK ZONE IS MARKED ON BASIS OF ERT RESULTS IN THE NEARBY AREA & PRESENCE OF BLACK CARBONACEOUS MATERIAL ON HILL SLOPE, LEFT BANK OF GHUNSA KHOLA & DRILL HOLE NDH-4
4. PLOTTED JOINTS ARE SCHEMATIC AND NOT AS PER THEIR ACTUAL SPACING.

SCALE-A: 1:4,000

SCALE-A: 1:2,000

GK-GE-L-05	GEOLOGICAL MAP OF POWERHOUSE AREA
DWG. NO.	DESCRIPTION: REFERENCE DRAWINGS

GHUNSA KHOLA HEP

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GEOLOGY
GEOLOGICAL SECTION ALONG TAILRACE TUNNEL

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DRAWN BY: A. POKHREL	APPROVED BY: R. MAHAJAN
DATE: 2025	DWG. NO GK-GE-S-07
SCALE: NTS	

Annex 2: Rock Mass Classification

1. ROCK MASS CLASSIFICATION

1.1 BACKGROUND

Rock mass classification was performed using two rock mass classification systems, namely the Geo-mechanics Classification or RMR System (Bieniawski, 1976) and the Rock Tunnel Quality Index or Q System (NGI, 2015). Using both systems, the studied rock mass was classified into five major types, viz. good, fair, poor, very poor and extremely poor, as shown in (Table III- 1-1).

Table III- 1-1: Summary of rock mass classification

S.N.	Rock class	Q-range	Description
1	I (Q_1)	40-10	Good
2	II (Q_2)	10-4	Fair
3	III (Q_3)	4-1	Poor
4	IV (Q_4)	1-0.1	Very poor
5	V (Q_5)	0.1-0.001	Extremely poor

1.2 ROCK MASS CLASSIFICATION

RMR and Q-values were determined at ten different locations in the project area. Details of the ratings for the individual parameters involved in each classification system, the RMR values and Q-values thus obtained and the corresponding rock mass classes are summarized in Table III- 1-2 and Table III- 1-3. The calculation of Q-values for rock mass classification using Q-System is depicted in Table III- 1-4.

Table III- 1-2: Rock mass classification using Geo-mechanics Classification (RMR) System

Location	Values of rating parameters							
	Strength of intact rock	Drill core quality, RQD	Spacing of discontinuities	Condition of discontinuities	Groundwater	Adjustment for discontinuity	RMR	Rock mass class
Tailrace portal, GK-1	12	17	16	13	7	0	65	Good
Powerhouse site, GK-2	7	13	10	14	15	-2	57	Fair
Powerhouse site, GK-3	7	13	10	13	15	-2	56	Fair
Surge shaft site, GK-4	7	8	16	17	15	-10	53	Fair
Headrace Tunnel, GK-5	2	3	5	5	7	-2	20	Very Poor
Headrace Tunnel, GK-6	7	13	16	21	10	-5	62	Good
Headrace Tunnel, GK-7	7	17	10	25	15	-10	64	Good
Headworks, GK-8	7	8	16	13	10	-5	49	Fair
Headworks, GK-9	7	13	16	12	10	-2	56	Fair
Headworks, GK-10	12	13	16	18	7	-2	64	Good

Investigators: Dr. R. B. Sah and S. Singh

Table III- 1-3: Rock mass classification using Rock Tunnel Quality Index or Q system

Locations	Rated Parameters (Values)							
	Drill Core Quality (% RQD)	No. of Joint sets, J_n	Joint Roughness, J_r	Joint Alteration, J_a	Joint Water Reduction Factor, Jw	Stress Reduction Factor, SRF	Q-value (Range Value)	Rock class
Tailrace portal, GK-1	80	9	3.0	2	1	2.5	5.33	Fair
Powerhouse site, GK-2	62	9	3.0	3	1	2.5	2.76	Poor
Powerhouse site, GK-3	60	9	1.5	1	1	2.5	4.00	Poor
Surge shaft site, GK-4	35	12	3.0	3	1	2.5	1.17	Poor
Headrace Tunnel, GK-5	10	12	1.0	6	1	2.5	0.06	Extremely poor
Headrace Tunnel, GK-6	80	12	2.0	1	1	1.0	13.33	Good
Headrace Tunnel, GK-7	85	12	3.0	2	1	2.5	4.25	Fair
Headworks, GK-8 (inlet DT)	80	12	2.0	1	1	2.5	5.33	Fair
Headworks, GK-9 (outlet FT)	70	12	1.5	1	1	2.5	3.50	Poor
Headworks, GK-10 (outlet DT)	60	12	1.5	1	1	2.5	3.00	Poor

Investigators: Dr. R. B. Sah and S. Singh

Table III- 1-4: Calculation of Q-values for rock mass classification

Description	Rated Parameters (Values)						
	Drill Core Quality (% RQD)	No. of Joint sets, J_n	Joint Roughness, Jr	Joint Alteration, J_a	Joint Water Reduction Factor, Jw	Stress Reduction Factor, SRF	Q-value (Range Value)
Tailrace portal, GK-1							
Q_{max}	83	9	3	1	1	1	27.67
Q_{min}	12	9	1	2	0.5	2.5	0.13
$Q_{measured}$	80	9	3	2	1	2.5	5.33
Powerhouse site, GK-2							
Q_{max}	94	9	3	1	1	1	31.33
Q_{min}	8	12	1	3	0.5	2.5	0.04
$Q_{measured}$	62	9	3	3	1	2.5	2.76
Powerhouse site, GK-3							
Q_{max}	94	9	3	1	1	1	31.33
Q_{min}	8	12	1	3	0.5	2.5	0.04
$Q_{measured}$	51	12	1.5	1.5	0.66	2.5	1.12
Surge shaft site, GK-4							
Q_{max}	91	12	3	1	1	1	22.75
Q_{min}	24	15	1	3	0.5	2.5	0.11
$Q_{measured}$	69.5	15	2	2	0.66	2.5	1.22
Headrace Tunnel, GK-5							
Q_{max}	10	12	1.5	5	1	2.5	0.10
Q_{min}	10	15	0.5	6	0.5	5	0.006
$Q_{measured}$	10	12	1	6	1	2.5	0.06
Headrace Tunnel, GK-6							
Q_{max}	92	12	3	1	1	1	23.00
Q_{min}	10	15	1	2	0.5	2.5	0.07
$Q_{measured}$	80	15	1.5	1	1	1	8.00
Headrace Tunnel, GK-7							
Q_{max}	92	12	3	1	1	1	23.00
Q_{min}	10	15	1	2	0.5	2.5	0.07

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Description	Rated Parameters (Values)						
	Drill Core Quality (% RQD)	No. of Joint sets, J_n	Joint Roughness, Jr	Joint Alteration, J_a	Joint Water Reduction Factor, Jw	Stress Reduction Factor, SRF	Q-value (Range Value)
$Q_{measured}$	85	12	3	2	1	2.5	4.25
Headworks, GK-8 (inlet DT)							
Q_{max}	92	12	3	1	1	1	23.00
Q_{min}	10	15	1	2	0.5	2.5	0.07
$Q_{measured}$	80	12	2	1	1	2.5	5.33
Headworks, GK-9 (outlet FT)							
Q_{max}	92	12	3	1	1	1	23.00
Q_{min}	10	15	1	2	0.5	2.5	0.07
$Q_{measured}$	70	12	1.5	1	1	2.5	3.50
Headworks, GK-10 (outlet DT)							
Q_{max}	92	12	3	1	1	1	23.00
Q_{min}	10	15	1	2	0.5	2.5	0.07
$Q_{measured}$	60	12	1.5	1	1	2.5	3.00

Annex 3: Geophysical Surveys

1. GEOPHYSICAL SURVEYS

1.1 INTRODUCTION

Geophysical studies were conducted at the proposed sites for the headworks, adit tunnels, underground powerhouse, surge shaft and tailrace tunnel. These studies were aimed at mapping the vertical and horizontal variations in the electrical resistivity of the substrata at these sites and thus detect their subsurface lithology. The mapping was performed using two dimensional (2D) Electrical Resistivity Tomography (ERT) survey procedures to attain the following objectives:

a. Establish 2D electrical resistivity sections of the ground profile showing different layers of lithological strata.
b. Estimate the thickness of overburden colluvials and alluvials.
c. Estimate the depths of rock head and rock mass conditions up to the depth of investigation.
d. Detect weak zones, fault zones and water bearing zones in the survey areas.
e. Detect the planes of weakness, deformed zones in bedrock, slopes and slides, sheared zones, etc.
f. Prepare analytical results in standard graphical format.
g. Interpret findings of the study and make suitable recommendationss on the subsurface geological conditions.

1.2 METHODOLOGY

1.2.1 Proposal for Geophysical Surveys

At the outset, the Consultant prepared a geophysical survey scheme based on the following:

- Results of the geological mapping, geophysical surveys and geotechnical investigations at various project components conducted during the 2012 feasibility report.
- Project layout proposed in the Consultant's Inception Report submitted to RHL on December 12, 2018, and approved by RHL on December 30, 2018.

The proposed geophysical survey scheme is presented in Table 1-1 and Figure 1-1.

Table 1-1: Proposed 2D ERT survey scheme

Profile No.	Location/ Component	Length (m)	Start		End	
			Northing	Easting	Northing	Easting
S1-S1'	Left bank of weir	270	584117.56	3049200.9	583917.29	3049019.8
S2-S2'	Along river	270	584098.29	3049218.7	583898.02	3049037.6
S3-S3'	Right bank of weir	270	584088.52	3049260.4	583888.25	3049079.3
S4-S4'	Desanding basin	270	583944.74	3049259.1	583744.47	3049077.9
S5-S5'	Adit-1	270	583156.44	3048697.3	582907.60	3048592.4
S6-S6'	Adit-2	270	581639.08	3048283.5	581443.24	3048097.6
S7-S7'	Surge shaft (Option 2 & 2a)	270	580176.06	3047418.5	579955.00	3047263.5
S8-S8'	Flat terrain (U/S of powerhouse-3)	270	580133.15	3047076.3	579951.04	3046876.9
S9-S9'	Powerhouse (Option 2 & 2a)	270	580211.19.	3047088.5	580024.82	3046893.1
S10-S10'	Surge shaft (Option 3)	270	579959.62	3047285.9	579689.67	3047291.9
S11-S11'	Pressure shaft (Option 3)	270	579934.01	3047155.1	579783.77	3046930.7
S12-S12'	Powerhouse (Option 3)	270	580019.93	3046857.9	579758.21	3046791.4
S13-S13'	MAT/ CTVT (Option 3)	270	580037.73	3046706.9	579816.77	3046551.7
S14-S14'	Tailrace tunnel (Option 3)	270	579698.10	3046609.9	579562.53	3046376.4
S15-S15'	Tail race tunnel (Option 3)	270	579586.56	3046544.5	579569.90	3046275.0
	Total	**4,050**				

The Consultant submitted the survey scheme to RHL on January 9, 2019, and the latter approved this scheme on January 11, 2019.

1.2.2 Survey Team

The Consultant carried out the geophysical surveys through its approved Sub-consultant M/S Explorer Geophysical Consultants Pvt. Ltd. (EGC). EGC's team for the geophysical survey comprised a geophysicist, an engineering geologist, two geophysical survey assistants, a senior surveyor and two survey assistants. Local laborers also were hired by the team as needed.

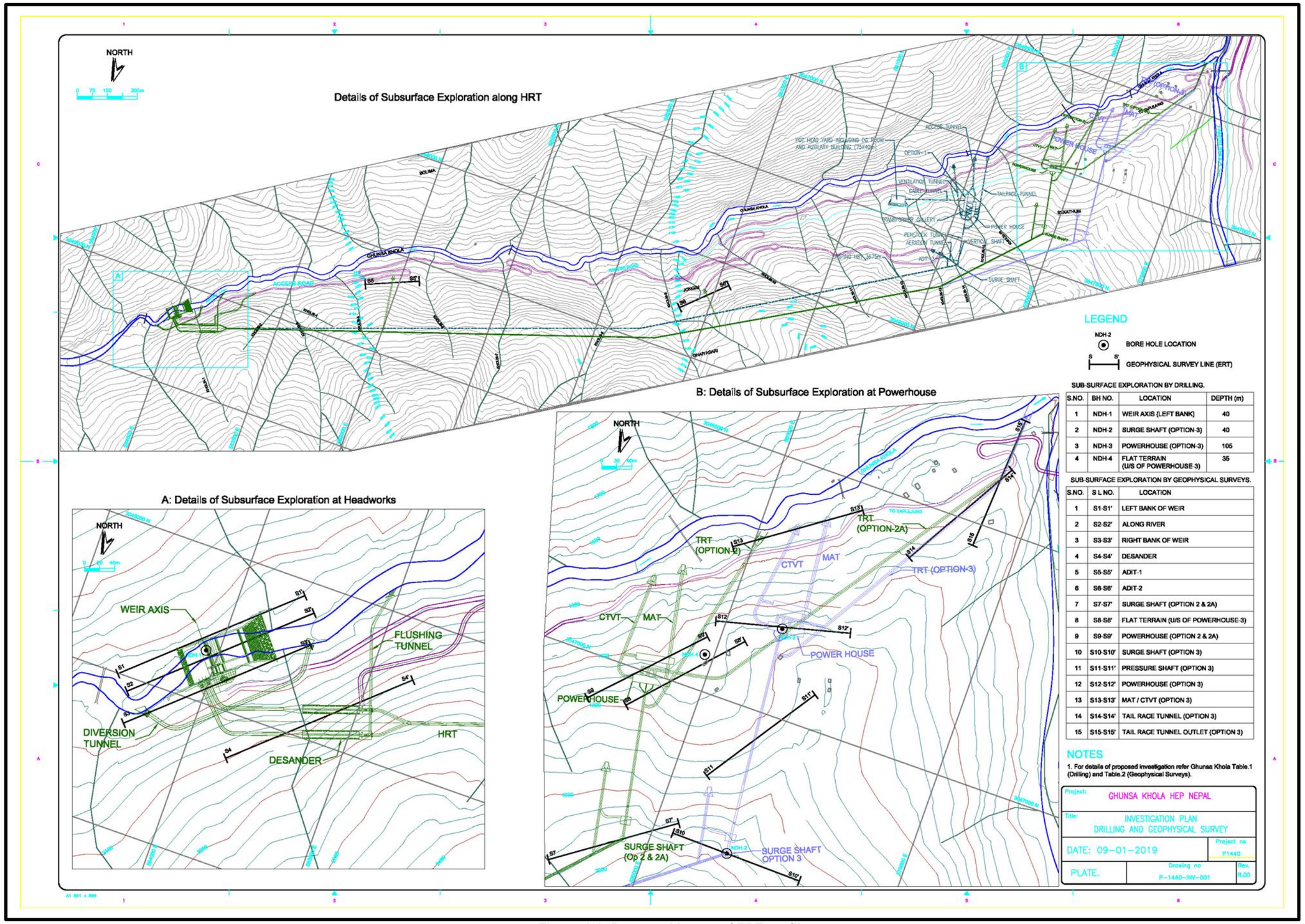
1.2.3 Survey Equipment

The geophysical surveys were conducted using the following equipment:

1. WDJD-4 Resistivity/IP equipment, with multi-function digital DC electrical resistivity and induced polarization meter, multi-core cable and stainless steel electrodes.
2. Total station.
3. Brunton Compass.
4. Digital camera.
5. Personal computer.

1.2.4 Field Mobilization

The geophysical survey team mobilized to site on January 22, 2019, and completed the 2D ERT surveys at the headworks, surge shaft, powerhouse and tailrace sites by February 7, 2019. However, the surveys at the two adits coud not be completed due to malfunctioning of the survey equipment. As such, a second survey team was mobilized to site on March 6, 2019, after the equipment was repaired. This team completed the outstanding surveys on March 13, 2019.



SUB-SURFACE EXPLORATION BY DRILLING.

S.NO.	BH NO.	LOCATION	DEPTH (m)
1	NDH-1	WEIR AXIS (LEFT BANK)	40
2	NDH-2	SURGE SHAFT (OPTION-3)	40
3	NDH-3	POWERHOUSE (OPTION-3)	105
4	NDH-4	FLAT TERRAIN (U/S OF POWERHOUSE-3)	35

SUB-SURFACE EXPLORATION BY GEOPHYSICAL SURVEYS.

S.NO.	S L NO.	LOCATION
1	S1-S1'	LEFT BANK OF WEIR
2	S2-S2'	ALONG RIVER
3	S3-S3'	RIGHT BANK OF WEIR
4	S4-S4'	DESANDER
5	S5-S5'	ADIT-1
6	S6-S6'	ADIT-2
7	S7-S7'	SURGE SHAFT (OPTION 2 & 2A)
8	S8-S8'	FLAT TERRAIN (U/S OF POWERHOUSE-3)
9	S9-S9'	POWERHOUSE (OPTION 2 & 2A)
10	S10-S10'	SURGE SHAFT (OPTION 3)
11	S11-S11'	PRESSURE SHAFT (OPTION 3)
12	S12-S12'	POWERHOUSE (OPTION 3)
13	S13-S13'	MAT / CTVT (OPTION 3)
14	S14-S14'	TAIL RACE TUNNEL (OPTION 3)
15	S15-S15'	TAIL RACE TUNNEL OUTLET (OPTION 3)

NOTES

1. For details of proposed investigation refer Ghunsa Khola Table.1 (Drilling) and Table.2 (Geophysical Surveys).

Project: GHUNSA KHOLA HEP NEPAL
Title: INVESTIGATION PLAN DRILLING AND GEOPHYSICAL SURVEY
DATE: 09-01-2019
Project no P1440
PLATE.
Drawing no P-1440-INV-001
Rev. R.00

Figure 1-1: Proposed layout of ERT profiles

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1.2.5 Field Surveys

1.2.5.1 Basic Procedure

The resistivity surveys were carried out with the WDJD-4 Resistivity/IP equipment and its accessories. The surveys were performed by injecting DC current into the ground through two current electrodes and measuring the resulting voltage differences at two potential electrodes. Field data were gathered to obtain a continuous coverage of the subsurface along the lines of investigation.

The Wenner electrode array configuration was employed for resistivity measurements. This configuration was selected as it has the advantages of high signal-to-noise ratio, symmetrical array and hence symmetrical anomalies, good lateral resolution, good vertical resolution, etc. as compared to other configurations. However, the Wenner Schlumberger and Dipole – Dipole configurations were also employed for crosschecking the acquired data.

The profile lengths and electrode spacing were chosen to permit acquisition of ground information up to a sufficient depth. Profile lengths of 90 m was used for information of shallow depths. To collect information up to depths of 40 m or more, a full profile length of more than 250 m was used. A minimum electrode spacing of 5 m was used in the survey procedure.

1.2.5.2 Field Procedures

At each of the proposed profiles, the following procedure was followed to carry out the 2D ERT survey:

a. The ERT profile was set out at the site with reference to the project area coordinates using a total station.
b. The multicore cables were laid along the set-out profile.
c. The positions of the electrodes were set out using a total station.
d. The apparent resistivity data along the profile was acquired using the WDJD-4 Resistivity/ IP equipment.
e. The collected ERT data was downloaded to a personal computer in the requisite format and cross-checked for its correctness and consistency.
f. Photographs were taken to characterize the topographical and morphological characteristics and bedrock exposures of the area.
g. Other important features at and around the prfofile, such as bedrock exposure, river/ stream channels, etc. were also surveyed with a total station.
h. The general geological features around the survey profiles and the project area were studied and recorded for reference and use in the interpretation of the geophysical data.

1.2.5.3 Surveyed Profiles

During the field surveys, attempts were made to follow the proposed profiles as closely as possible. However, some modifications to the proposed ERT survey scheme had to be made, both in terms of alignment and orientation, because of the prevailing conditions at a few of the proposed locations. Some of these modifications, and the reasons behind them, are listed below:

- The proposed ERT-2 line in the headworks area crossed the river channel, and it was not possible to carry out ERT survey in water. As such, this line was shifted slightly and placed adjacent to the right bank of the river.
- The proposed ERT-4 line in the desanding basin area could not be surveyed because of the very steep slope and extremely risky access conditions in this area. This line was shifted to the area just upstream of the headworks area and run across Ghunsa Khola.
- The ERT-7 line in the surge shaft area could not be surveyed at the proposed location because of very steep slopes and extremely risky access conditions in this area. As such, this line was modified suitably at the site ensuring that useful data could be derived from the modified line.
- Orientation of the proposed ERT-8 and ERT-9 lines in the powerhouse area were modified slightly because of the steep slope in the area and the large boulder deposits present near the terminal points of these lines.

- Orientation of the proposed ERT-11 line in the powerhouse area was slightly changed because of the presence of large fractured rock fragments and boulders towards the end of the profile.
- Orientation of ERT-10 was varied in order to layout the cable on colluvial deposit for better depth information around the surge shaft site.
- ERT surveys at Adit-1 and Adit-2 were carried out at the nearest possible area to the proposed areas as the site conditions of the proposed locations were difficult to access. Two ERT lines were located near Adit-2.

During the surveys, poor conductor materials, such as bedrock, large boulders and fractured rock fragment deposits were, in general, avoided to obtain appropriate results. Despite the modifications mentioned above, all of the proposed survey locations were appropriately and adequately covered.

The actual surveyed profile lines at the various project components are illustrated in Photo 1-1 to Photo 1-5 and Figure 1-2. Details of the profiles lines are presented in Table 1-2.

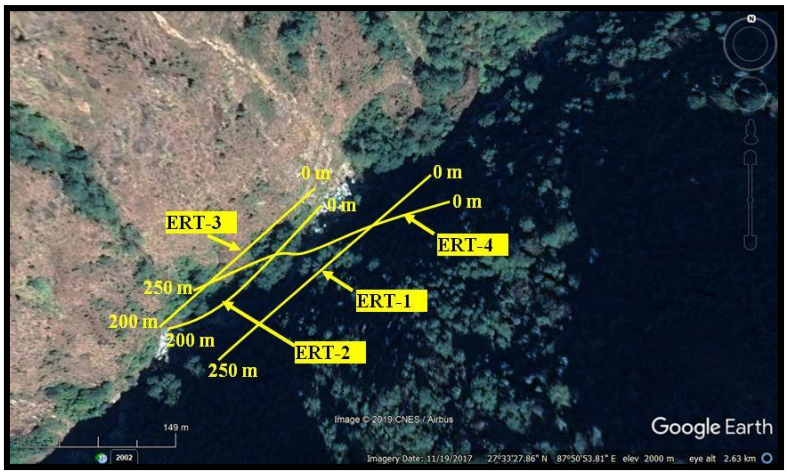


Photo 1-1: ERT layouts in headworks area

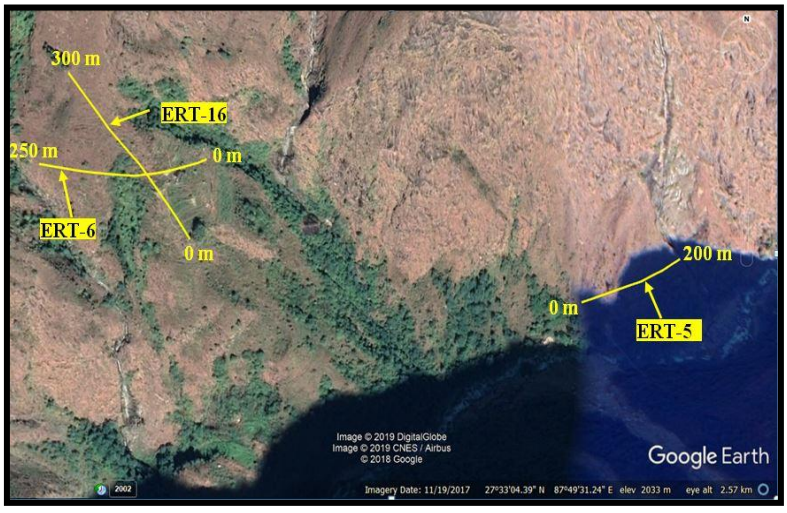


Photo 1-2: ERT profiles near Adit 1 and Adit 2 areas

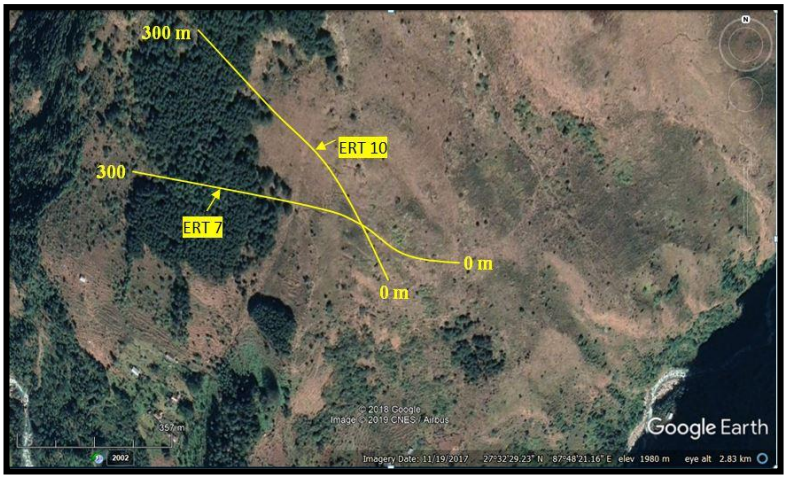


Photo 1-3: ERT profiles in surge shaft area

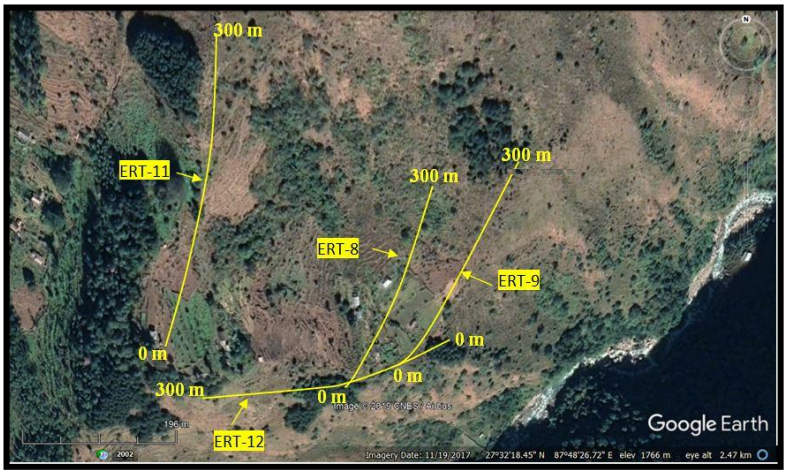


Photo 1-4: ERT profiles in powerhouse area

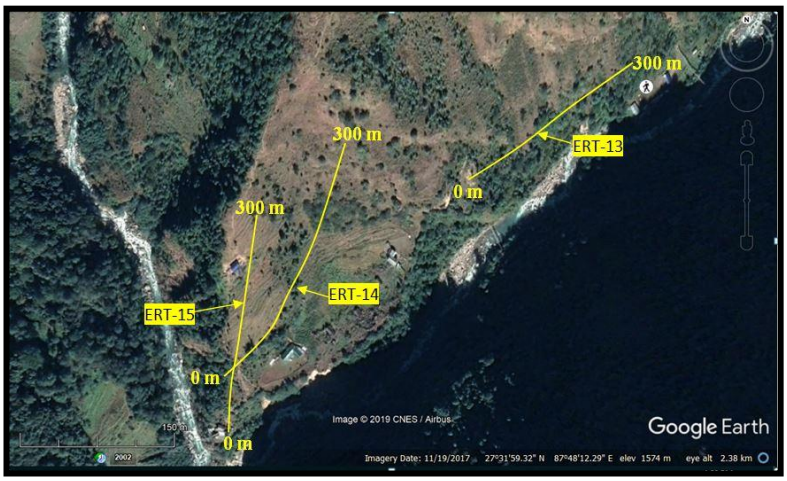


Photo 1-5: ERT profiles in tailrace area

1.2.6 Data Processing

The apparent resistivity data collected at the site was filtered, processed and treated with the software RES2DINV. This software inverts the field data, calculates an appropriate model in terms of resistivity and provides output in the form of resistivity contours. This inversion data was used to draw up the lithological and geological information along the ERT profiles.

Table 1-2: Details of 2D ERT survey coverage

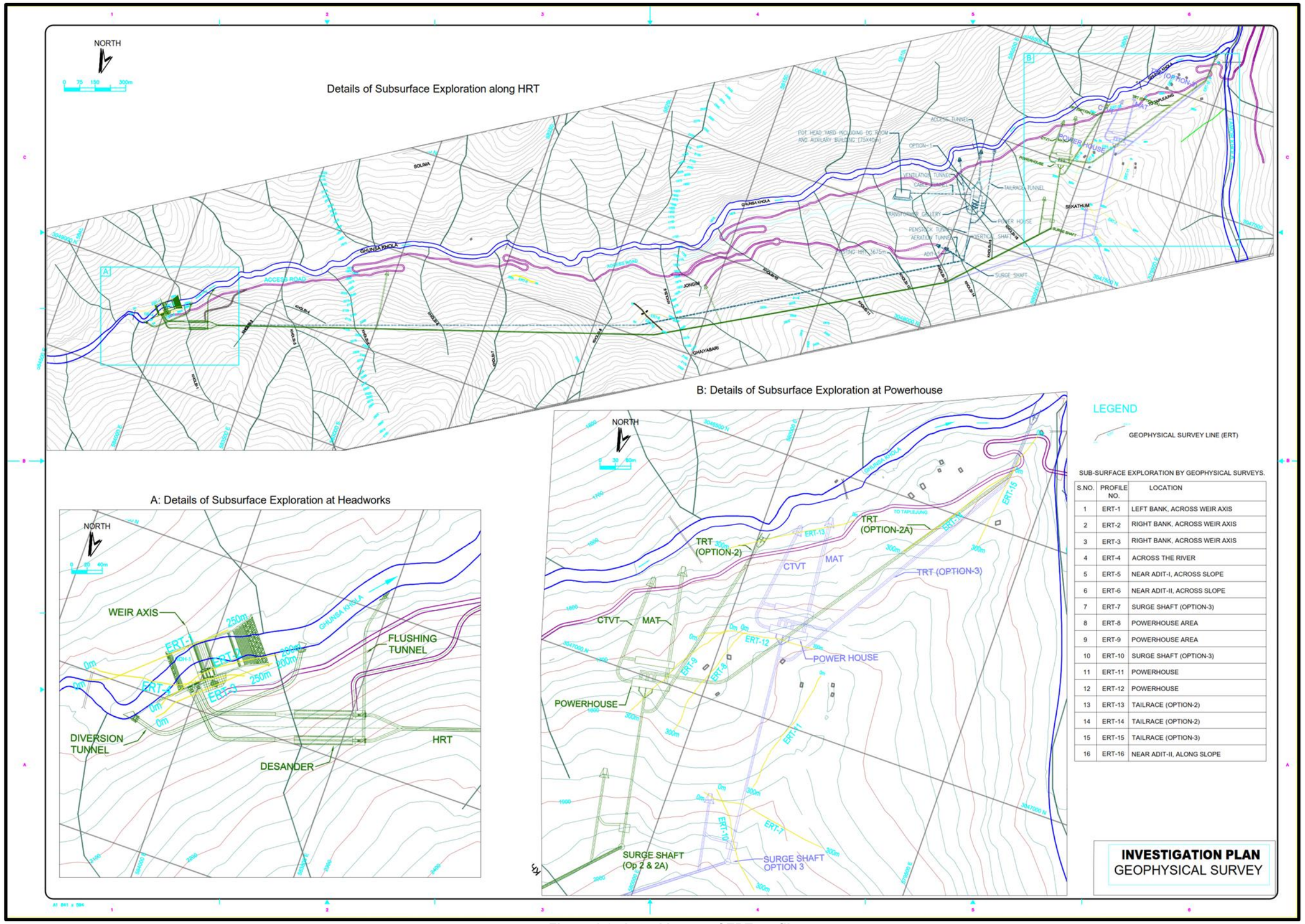
Profile No.	Location	Length (m)	Start		End	
			Northing	Easting	Northing	Easting
ERT-1	Left bank, across weir axis	250	3049207.3	584150.01	3049065.6	583984.9
ERT-2	Right bank, across weir axis	200	3049200.5	584043.4	3049091.1	583921.51
ERT-3	Right bank, across weir axis	200	3049218.8	584046.7	3049109.0	583926.364
ERT-4	Across river	250	3049185.7	584146.4	3049131.0	583948.6
ERT-5	Near Adit-I, across Slope	200	3048419.9	582340.2	3048429.6	582458.9
ERT-6	Near Adit-II, along slope	250	3048435.8	581809.3	3048365.3	581665.9
ERT-7	Surge shaft (Option-3)	300	3047176.0	579933.5	3047223.6	579631.2
ERT-8	Powerhouse area	300	3046842.7	579948.0	3047081.6	580035.0
ERT-9	Powerhouse area	300	3046862.3	579994.6	3047076.0	580123.4
ERT-10	Surge shaft (Option-3)	300	3047161.6	579913.0	3047345.3	579732.8
ERT-11	Powerhouse	300	3046885.0	579793.2	3047129.4	579844.9
ERT-12	Powerhouse	300	3046892.6	580051.7	3046845.2	579807.8
ERT-13	Tailrace (Option-2)	300	3046574.0	579829.7	3046713.3	580047.8
ERT-14	Tailrace (Option-3)	300	3046371.4	579563.5	3046599.8	579723.8
ERT-15	Tailrace (Option-3)	300	3046296.6	579557.4	3046535.7	579591.5
ERT-16	Near Adit-II, along slope	300	3048355.0	581849.9	3048432.1	581678.0
	Total (m)	**4350**				

1.2.7 Data Analysis and Interpretation

The data from the resistivity surveys were interpreted by correlating the surveyed resistivity values with standard resistivity values of rocks, soils and other materials related to the geological formations of the project area. For the correlation, the resistivity value ranges listed in Table 1-3 were assumed to be representative of the different geological materials present in the project area.

Table 1-3: General correlation of electrical resistivity and lithology

Electrical resistivity, Ω-m	Resistivity class	Expected lithology	
		Overburden	Bedrock
Up to 500	Very low	Saturated alluvial/colluvial deposits of fine sediments/ residual soil	Highly weathered /fractured saturated rock of gneiss
500 – 1000	Low	Saturated alluvial deposit of sand and gravel with pebble, cobble and boulders/partly saturated colluvial deposit of fine sediments/colluvial deposit of fine sediments	Weathered/ saturated to partly saturated fractured rock of gneiss
1000 – 2000	Medium	Colluvial deposit of residual soil/fine sediments, saturated to partly saturated alluvial deposit of sand, gravel, pebble, coble , medium to large boulders and fractured rock fragments	Weathered/fractured rock of gneiss.
≥ 2000	High	Extremely dry overburden of alluvial/colluvial deposit sand and gravel with Medium to large boulders and fractured rock fragments with dry residual soil.	Competent/intact bedrock of Gneiss



S.NO.	PROFILE NO.	LOCATION
1	ERT-1	LEFT BANK, ACROSS WEIR AXIS
2	ERT-2	RIGHT BANK, ACROSS WEIR AXIS
3	ERT-3	RIGHT BANK, ACROSS WEIR AXIS
4	ERT-4	ACROSS THE RIVER
5	ERT-5	NEAR ADIT-I, ACROSS SLOPE
6	ERT-6	NEAR ADIT-II, ACROSS SLOPE
7	ERT-7	SURGE SHAFT (OPTION-3)
8	ERT-8	POWERHOUSE AREA
9	ERT-9	POWERHOUSE AREA
10	ERT-10	SURGE SHAFT (OPTION-3)
11	ERT-11	POWERHOUSE
12	ERT-12	POWERHOUSE
13	ERT-13	TAILRACE (OPTION-2)
14	ERT-14	TAILRACE (OPTION-2)
15	ERT-15	TAILRACE (OPTION-3)
16	ERT-16	NEAR ADIT-II, ALONG SLOPE

Figure 1-2: Actual layout of ERT profiles

The inversion results showing the resistivity tomogram and interpretative lithological cross-sections along the ERT profiles were prepared as per the actual locations of the electrodes along the surveyed profile slope (not the horizontal or plan distance). Hence, although they are apparently long in section, the electrode distances are actually short in plan.

1.3 INTERPRETATION OF SURVEY RESULTS

1.3.1 General

Representative resistivity tomograms for the profiles and their interpretations were carried out based on the methodology described in Chapter 1.2. Their interpretation is discussed in the following sections.

1.3.2 Headworks Area

1.3.2.1 ERT-1

ERT-1 runs along the left bank of Ghunsa Khola, across the weir axis (Photo 1-1, Figure 1-2. The total length of the profile was 250 m. The representative resistivity tomogram of this profile is presented in Figure 1-3, and the lithological interpretation of this tomogram is presented in Figure 1-4.

The lithological section along ERT-1 can be interpreted as a three-layered model. The top layer with high resistivity values of around 3,000 Ω-m from the initial to the terminal points of the profile represent dry colluvial deposits containing boulders of fractured rock fragments that are covered by residual soil deposits. A very thin layer of the deposit (about 1 to 2 m) is present between the initial point of the profile up to Ch. 30 m. From Ch. 30 m to the end of the profile, the thickness varies from 10 to 15 m. The thickness of the top layer at the weir axis is expected to be about 11 m.

The second layer with low resistivity values below 500 Ω-m from Ch. 35 to 70 m and Ch. 165 to 230 m indicates saturated to partially saturated colluvial deposits or colluvial deposits consisting of fine sediments.

The high resistivity values in the third layer indicate competent/intact bedrock of gneiss. The bedrock is present at shallow depth of less than 1 m towards the initial point of the profile up to Ch. 35 m, and from thereon to the end of the profile, at varying depths of 10 to 17 m. At the weir axis, bedrock can be expected at a depth of about 15 m from the surface.

1.3.2.2 ERT-2

ERT-2 runs along right bank of Ghunsa Khola, across the weir axis (Photo 1-1, Figure 1-2). The total length of the profile was 200 m. The representative resistivity tomogram is presented in Figure 1-5, and its lithological interpretation is presented in Figure 1-6.

The lithological section along ERT-2 can be interpreted as a three layered model. The top layer with high resistivity values between the intial and terminal points of the profile show the presence of unsaturated alluvial deposits of sand and gravel with pebbles, cobbles and medium to large boulders. The thickness of the layer varies from 3 to 8 m from Ch. 20 m to the end of the profile.

The second layer with low to medium resistivity values indicates saturated alluvial deposit of sand and gravel with pebbles, cobbles and medium to large boulders. The layer is expected at varying depths of 3 to 8 m.

The high resistivity values of above 2,000 Ω-m in the third layer indicate competent/intact bedrock of gneiss. The bedrock is found at shallow depths of less than 1 m from the initial point of the profile. From Ch. 20 m to the end of the profile, the bedrock can be expected at varying depths of 10 to 18 m. At the weir axis, the competent bedrock is expected at a depth of 18 m from the surface.

Low resistivity values of around 1000 Ω-m from Ch. 140 to 170 m indicate saturated, weathered or fractured rock of gneiss at an expected depth of 6 to 8 m.

1.3.2.3 ERT-3

ERT-3 runs along the right bank of Ghunsa Khola, across the weir axis and just near the steep slope Photo 1-1, Figure 1-2). Its total length was 200 m. The representative resistivity tomogram of this profile is presented in Figure 1-7, and its lithological interpretation is presented in Figure 1-8.

The lithological section along ERT-3 can be interpreted as a three layered model. The top layer with very high resistivity values along the profile indicates the presence of unsaturated alluvial deposits of sand and gravel with pebbles, cobbles and medium to large boulders.

The second layer with low to medium resistivity is present from Ch. 25 to 110 m and Ch. 165 to the end of the profile. Its thickness varies between 3 to 8 m. This layer indicates saturated alluvial deposits of sand and gravel with pebbles, cobbles and medium to large boulders.

The high resistivity value in the third layer shows competent/intact bedrock of gneiss. It is found at shallow depths of less than 1 m towards the beginning of the profile up to Ch. 20 m. From Ch. 20 m to 130 m and Ch. 155 to the end of the profile, the competent bedrock is found at varying depths of 5 to 15 m. From Ch. 130 to 155 m, saturated weathered/ fractured rock of gneiss is found at varying depths of 6 to 8 m.

1.3.2.4 ERT-4

ERT-4 extends from the left bank to the right bank of Ghunsa Khola, crossing the river at an acute angle to the flow direction (Photo 1-1, Figure 1-2). Its total length was 250 m. The representative resistivity tomogram for ERT-4 is presented in Figure 1-9, and the lithological interpretation of this tomogram is presented in Figure 1-10.

The lithological section along ERT-4 can be interpreted as a three layered model. The top layer with high resistivity values from the initial point of the profile to Ch. 110 m shows dry colluvial deposits of boulders and fractured rock fragments covered with residual soil with thickness varying from 1 to 12 m. The thickness is less than 1 m up to Ch. 20 m. High resistivity values in the first layer between Ch. 110 m and the end of the profile shows unsaturated alluvial deposits of sand and gravel with pebbles, cobbles and medium to large boulders with thickness varying between 1 10 m.

The second layer with very low resistivity values (<700 Ω-m) from Ch. 45 to 90 m and from Ch. 110 to 190 m indicates saturated to partly saturated colluvial deposit/colluvial deposit of fine sediments with average thickness of 3 to 8 m. The second layer with low resistivity values from Ch. 110 to 190 m indicates the presence of saturated alluvial deposits of sand and gravel with pebbles, cobbles and boulders.

The high resistivity values in the third layera indicate competent/intact bedrock of gneiss at shallow depths of less than 1 to 2 m from the initial point of the profile up to Ch. 20 m. From Ch. 20 m to the end of the profile, the competent bedrock is found at varying depths of 8 of 20 m. In the river section, the bedrock can be expected at depths of 18 to 20 m from the surface.

1.3.3 Adit-1

1.3.3.1 ERT-5

ERT-5 runs across the slope near around the location of Adit-1 (Photo 1-2, Figure 1-2). Its total length was 200 m. The representative resistivity tomogram of ERT-5 is presented in Figure 1-11, and its lithological interpretation is presented in Figure 1-12.

The lithological section along ERT-5 can be interpreted as a three layered model. The top layer with very high resistivity values along the profile shows highly unsaturated colluvial deposits of boulders and fractured rock fragments with residual soil. The thickness of this layer varies from 8 to 10 m up to Ch. 160 m. The layer becomes thinner towards the end of the profile.

The second layer with medium resistivity values (<2000 Ω-m) occurs from Ch. 40 to 140 m. This layer indicates a colluvial deposit of fine sediments.

The high resistivity values in the third layer indicate competent/intact bedrock of gneiss at varying depths of 12 to 15 m from the surface up to Ch. 160 m. It is found at shallow depths of 1 to 2 m towards the end of the profile.

1.3.4 Adit-2

1.3.4.1 ERT-6

ERT-6 runs across the slope around the Adit-2 location (Photo 1-2, Figure 1-2). Its total length was 250 m. The representative resistivity tomogram of ERT-6 is presented in Figure 1-13, and its lithological interpretation is presented in Figure 1-14

The lithological section along ERT-6 can be interpreted as a three layered model. The top layer with very high resistivity values along the profile shows highly unsaturated colluvial deposit of boulders and fractured rock fragments with residual soil. The thickness of this layer varies between 7 and 12 m.

The second layer with low resistivity values (<1000 Ω-m) runs from Ch. 45 to 200 m. This layer indicates colluvial deposits of fine sediments.

The high resistivity values in the third layer indicates competent/intact bedrock of gneiss. The bedrock is expected to occur at varying depth of 10 to 20 m from the surface.

1.3.4.2 ERT-16

ERT-16 runs along the slope near the Adit-2 site (Photo 1-2, Figure 1-2). Its total length was 300 m. The representative resistivity tomogram of ERT-16 is presented in Figure 1-15 and its lithological interpretation of resistivity tomogram is presented in Figure 1-16.

The lithological section along ERT-16 can be interpreted as a three layered model. The top layer with very high resistivity values along the profile shows highly unsaturated colluvial deposits of boulders and fractured rock fragments with residual soil. Its thickness is 5 to 7 m up to Ch. 80 m. The thickness varies from 10 to 15 m towards the end of the profile.

The second layer with low resistivity values (<1000 Ω-m) is present from Ch. 45 to 200 m. It indicates colluvial deposit of fine sediments.

The high resistivity values in the third layer indicate competent/intact bedrock of gneiss. Up to Ch. 80 m, the bedrock is expected to occur at varying depths of 5 to 8 m from the surface. It is found at depths between 15 to 25 m towards the end of the profile.

1.3.5 Surge Shaft Area

1.3.5.1 ERT-7

ERT-7 was carried out in the area proposed for Option-3 of the surge shaft (Photo 1-3, Figure 1-2). Its total length was 300 m. The representative resistivity tomogram of this profile is presented in Figure 1-17, and its lithological interpretation of resistivity tomogram is presented in Figure 1-18.

The lithological section along ERT-7 can be interpreted as a two layered model. The top layer with very high resistivity values (>3000 Ω-m) along the profile indicates highly unsaturated colluvial deposits of boulders and fractured rock fragments along with residual soil, with thickness varying from 5 to 16 m. Very low resistivity values (<500 Ω-m) from Ch. 150 to 190 m and from Ch. 230 to 255 m, at varying depths of 10 to 15 m and with varying thicknesses of 5 to 10 m, indicate colluvial deposits of fine sediments.

Competent/intact bedrock of gneiss is expected at depths of 10 to 18 m from Ch. 25 to 140 m and around Ch. 195 m. Low resistivity values around Ch. 160 m at a depth of 18 to 20 m indicate weathered bedrock of gneiss. The low resistivity zone, comprising weathered rock between two high resistivity zones of hard rock, indicates a fault zone.

1.3.5.2 ERT-10

ERT-10 was carried out at Option-2 of the surge shaft area (Photo 1-3, Figure 1-2).Its total length was 300 m. The representative resistivity tomogram along this profile is presented in Figure 1-19, and its lithological interpretation tomogram is presented in Figure 1-20.

The lithological section along ERT-10 can be interpreted as a three layered model. The top layer with very high resistivity values (>4000 Ω-m) along the profile indicates highly unsaturated colluvial deposit of fractured rock fragments with residual soil, with thickness varying between 10 to 20 m.

The second layer with low resistivity values (<1000 Ω-m) is present from Ch. 85 to 190 m and Ch. 220 to 270 m at varying depths of 8 of 16 m. This layer indicates colluvial deposit consisting of fine sediments.

Competent/intact bedrock of gneiss is expected to occur at varying depths of 10 to 23 m from Ch. 35 to 140 m and varying depth of 10 to 20 m from Ch. 200 to 275 m. Weathered bedrock of gneiss is expected to occur at a depth of 20 to 25 m from Ch. 140 to 200 m. Bedrock is found at an average depth of 10 m from the surface near the proposed surge shaft.

1.3.6 Powerhouse Area

1.3.6.1 ERT-8

ERT-8 was carried out around Option-1 of the powerhouse (Photo 1-4, Figure 1-2). Its total length was 300 m. The representative resistivity tomogram along this profile is presented in Figure 1-21, and its lithological interpretation is presented in Figure 1-22.

The lithological section along ERT-8 can be interpreted as a multi-layered model. The top layer with resistivity values above 1000 Ω-m along the profile indicates highly unsaturated colluvial deposit of boulders and fractured rock fragments with residual soil. The thickness of the layer varies from 7 to 12 m.

The second layer with medium resistivity values from Ch. 45 to 200 m and around Ch. 240 m indicates the colluvial deposit of fine sediments. The thickness of the layer varies from 5 to 12 m.

In the third layer, competent/intact bedrock of gneiss from Ch. 5 to 50 m, Ch. 125 to 190 m and Ch. 200 to 265 m is expected to occur at varying depths of 8 to 20 m. At the center part of the profile, this bedrock is expected to occur at a depth of 15 m.

Low resistivity values from Ch. 50 to 125 m and around Ch. 195 m of the third layer implies weathered bedrock of gneiss which is expected to occur at varying depths of 15 to 20 m. This weathered rock area from Ch. 70 m to 130 m indicates a fault zone.

1.3.6.2 ERT-9

This profile was carried out around Option-1 of the powerhouse (Photo 1-4, Figure 1-2Its total length was 300 m. The representative resistivity tomogram along this profile is presented in Figure 1-23, and its lithological interpretation is presented in Figure 1-24.

The lithological section along ERT-9 can be interpreted as a three layered model. The top layer with very high resistivity values (>4000 Ω-m) along the profile indicates highly unsaturated colluvial deposits of boulders and fractured rock fragments with residual soil. The thickness of the layer varies from 5 to 15 m. In the powerhouse area, the thickness of this layer is between 10 to 15 m.

The second layer with low resistivity values (<1000 Ω-m) from Ch. 15 m to 100 m and Ch. 155 to 270 m indicates colluvial deposit of fine sediments. The depth of this layer varies between 5 to 10 m.

In the third layer, high resistivity values from Ch. 140 to 255 m at varying depths of 15 to 18 m indicate competent/intact bedrock of gneiss. Low to medium resistivity values from Ch. 50 to 140 m indicate the presence of weathered bedrock of gneiss at varying depths of 12 to 17 m. This weathered rock area up to Ch. 140 m implies a fault zone.

1.3.6.3 ERT-11

ERT-11 was carried out near Option-2 of the powerhouse (Photo 1-4, Figure 1-2). Its total length was 300 m. Th representative resistivity tomogram along this profle is presented in Figure 1-25 and its lithological interpretation is presented in Figure 1-26.

The lithological section along ERT-11 can be interpreted as a three layered model. The top layer with resistivity values above 1200 Ω-m along the profile is present up to Ch. 165 m with varying thickness of 2 to 15 m and from thereon to the end of the profile wit thickness of 5 to 9 m. This layer indicates highly unsaturated colluvial deposit of boulders of fractured rock fragments with residual soil.

Th second layer with low resistivity values (<1000 Ω-m) from the initial point of the profile to Ch. 250 m indicates colluvial deposits consisting fine sediments. The thickness of this layer varies from 5 to 20 m.

Bedrock is expected at shallow depths of 5 to 8 m from Ch. 250 m to the end of the profile. Between Ch. 165 to 250 m, it is expected at varied depths of 10 of 18 m from the surface. Medium resistivity values of around 1400 to 2000 Ω-m in the third layer from the Ch. 105 to 165 m indicated weathered bedrock of gneiss at varying depths of 20 to 30 m. Competent/ intact bedrock of gneiss is expected from the Ch. 165 to 290 m.

1.3.6.4 ERT-12

ERT-12 was carried out in the area around Option-2 of the powerhouse (Photo 1-4, Figure 1-2). Its total length is 300 m. The representative resistivity tomogram along this profile is presented in Figure 1-27, and its lithological interpretation is presented in Figure 1-28

The lithological section along ERRT-12 can be interpreted as a multi-layered model. The top layer with resistivity values greater than 1000 Ω-m along the profile indicates highly unsaturated colluvial deposits of boulders of fractured rock fragments with residual soil, with varying thickness of 4 of 15 m. In the powerhouse area, the thickness of this layer varies between 8 and 10 m.

The second layer with low resistivity values (<1000 Ω-m) from Ch. 15 to 205 m and Ch. 230 to 265 m indicates colluvial deposit of fine sediments at the varying depth of 4 to 15 m. The thickness of this layer varies between 4 and 12 m.

Competent/intact bedrock of gneiss is found from Ch. 40 to 130 m at depths of 10 to 15 m and from Ch. 170 to 260 m at depths of 5 to 20 m. It is expected at shallow depths of 5 to 8 m between Ch. 205 and 230 m. In the powerhouse area, the bedrock is expected at varying depths of 8 to 15 m from the surface.

Highly weathered bedrock of gneiss is found around Ch. 130 to 170 m at depths 15 of 20 m. This area of rock mass between two hard rock masses implies a fault or a highly deformed zone.

1.3.7 Tailrace Area

1.3.7.1 ERT-13

ERT-13 was carried out along Option-2 of the tailrace, along the right bank of Ghunsa Khola (Photo 1-5, Figure 1-2). Its total length was 300 m. The representative resistivity tomogram along this profile is presented in Figure 1-29, and its lithological interpretation is presented in Figure 1-30.

The lithological section along ERT-13 can be interpreted as a multi layered model. Its top layer with resistivity values above 2000 Ω-m along the profile indicates highly unsaturated colluvial deposit of boulders of fractured rock fragments with residual soil. It thickness varies between 2 and 16 m.

The second layer with low resistivity values (<700 Ω-m) is present between Ch. 100 and 160 m with thickness of 5 to 13 m and between Ch. 170 and 230 m with thickness of 5 to 10 m. This layer indicates saturated to partly saturated colluvial deposit/ colluvial deposit of fine sediments.

In the third layer, low resistivity values (<800 Ω-m) from Ch. 100 to 160 m at depths of 13 to 22 m and from Ch. 205 to 250 m at depths of 10 to 15 m indicate the presence of weathered/ fractured, saturated to partly saturated rock of gneiss. Competent/ intact bedrock of gneiss is expected to occur at shallow depths of 3 to 5 m around Ch. 160 m. From Ch. 35 to 100 m and 160 to 205 m, competent/intact bedrock of gneiss is expected to occur at varying depths of 5 to 16 m. The bedrock is found at shallow depths of 1 to 5 m towards the end of the profile.

1.3.7.2 ERT-14

ERT-14 was carried out in the area around Option-3 of the tailrace (Photo 1-5, Figure 1-2). Its total length was 300 m. The representative resistivity tomogram of this profile is presented in Figure 1-31, and its lithological interpretation is presented in Figure 1-32

The lithological section along ERT-14 can be interpreted asa three-layered model. The top layer with high resistivity values (>3000 Ω-m) along the profile indicates highly unsaturated colluvial deposit of boulders of fractured rock fragments with residual soil. The thickness of this unsaturated colluvial deposit varies from 5 to 20 m along the profile.

The second layer with low resistivity values less than 1000 Ω-m from Ch. 85 to 240 m at depths of 5 to 20 m indicates colluvial deposit of fine sediments. The thickness of this layer varies from 5 to 25 m.

In the third layer, competent/ intact bedrock of gneiss is expected to occur from Ch. 40 to 85 m at depths of 15 to 17 m. It is also expected from Ch. 85 to 195 m at depths of 25 to 30 m.

1.3.7.3 ERT-15

ERT-15 was carried out along the hill slope housing Option-3 of the tailrace, near the confluence of Ghunsa Khola and Tamor River. Its total length was 300 m. Representative resistivity tomogram

along the profileis presented in Figure 1-33, and its lithological interpretation is presented in Figure 1-34.

The lithological section along ERT-15 can be interpreted as a multi-layered model. The top layer with very high resistivity values up to Ch. 80 m, with thickness varying from less than 2 m and to 10 m, shows highly unsaturated colluvial deposits of boulders of fractured rock fragments with residual soil. The same layer can be found from Ch. 95 m to the end of the profile with thickness varying between 5 and 13 m.

The second layer with low resistivity values from Ch. 45 to 75 m at depths of 8 to 10 m and thickness between 3 and 7 m indicates colluvial deposits of fine sediments. The same layer is also found from Ch. 155 to 270 m at depths of 8 to 15 m and thickness of around 5 to 18 m.

Resistivity values greater than 2000 Ω-m in the third layer indicate competent/intact bedrock of gneiss. It is found at a depth of 1 m from the surface between Ch. 5 and 15 m and around Ch. 80 m. The bedrock is expected to occur at depths of 10 to 15 m from Ch. 15 to 75 m and from Ch. 105 to 150 m, and it extends to greater depth of 15 to 25 m from the center of the profile to its end.

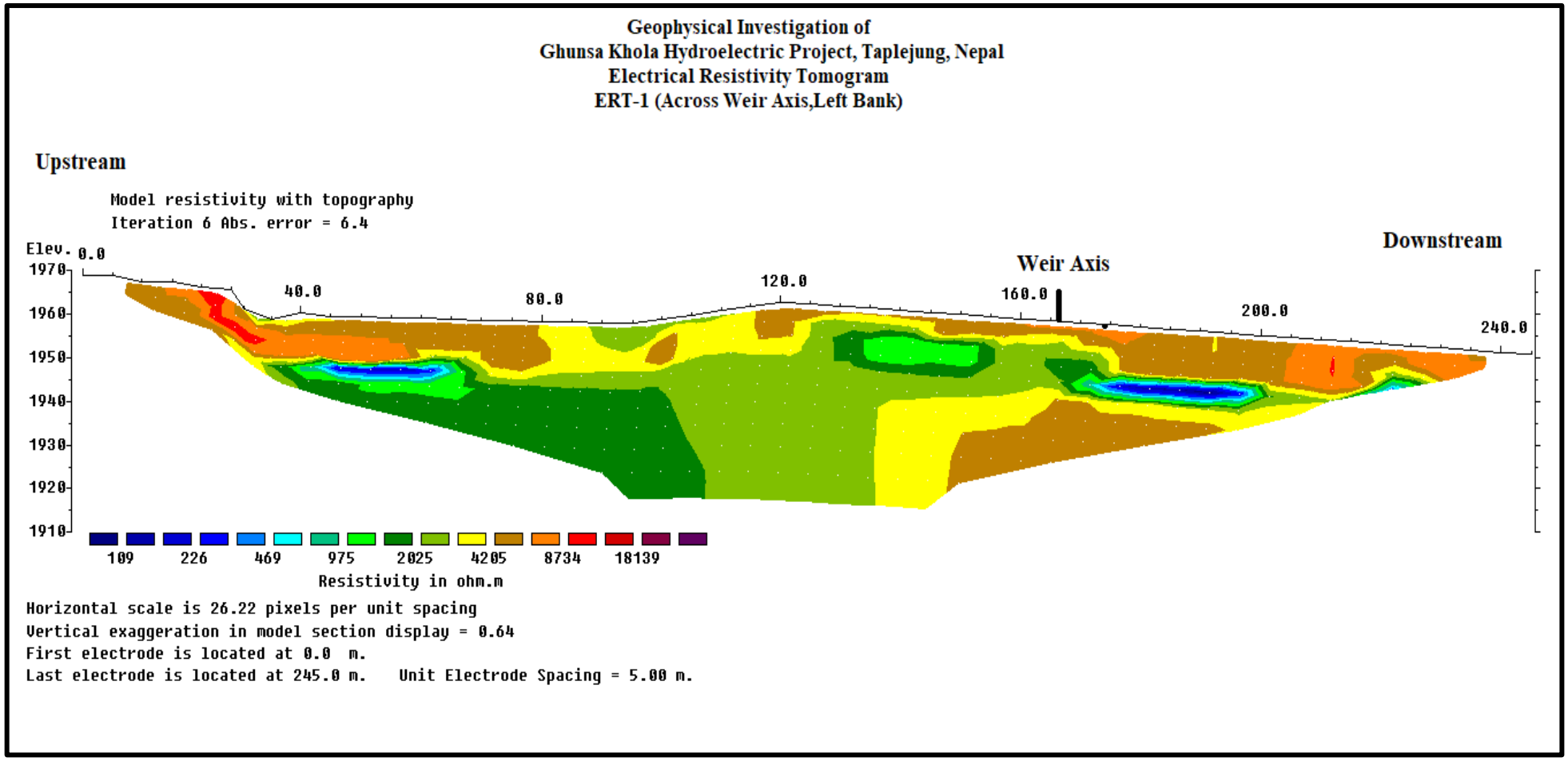


Figure 1-3: Electrical resistivity tomogram of ERT–1

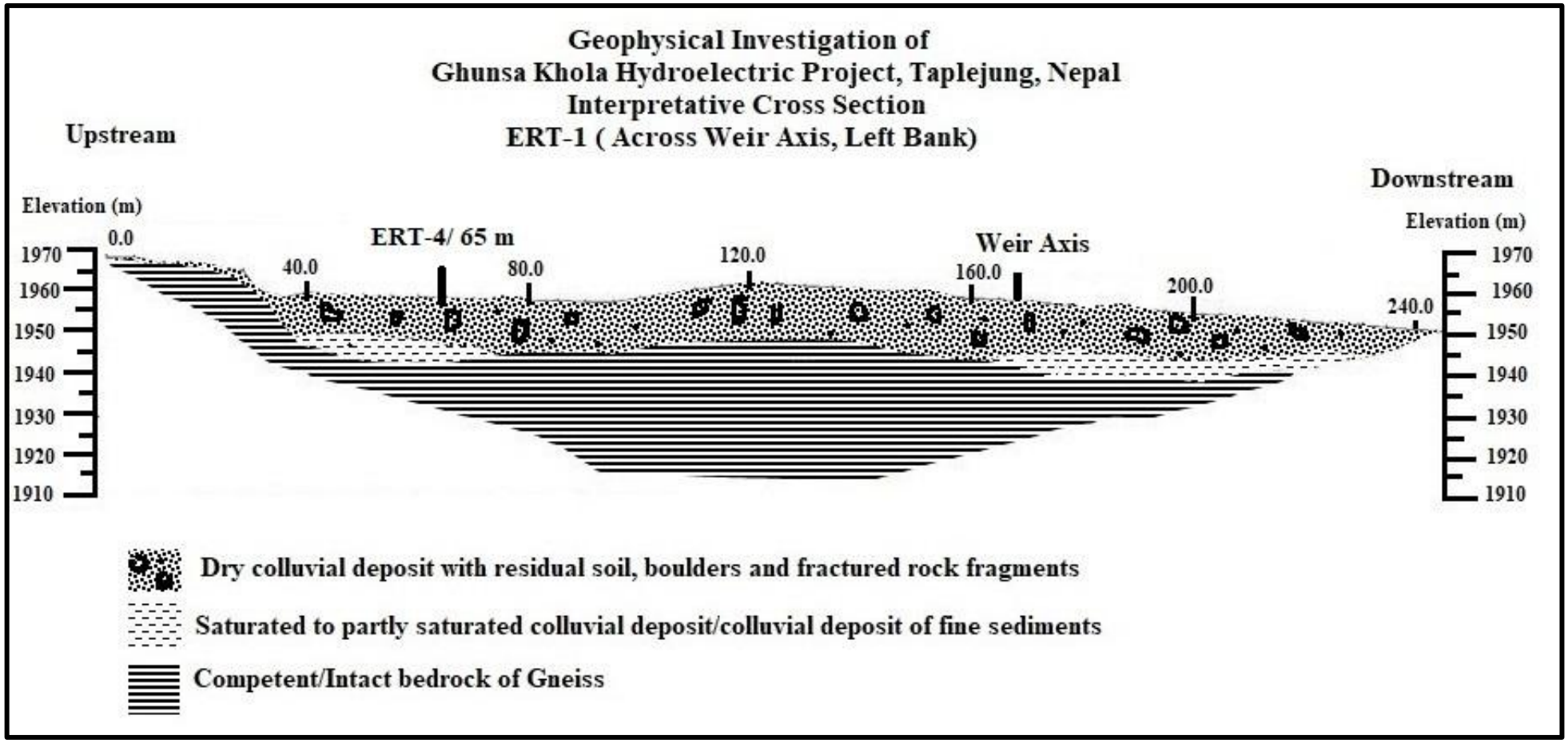


Figure 1-4: Interpretative cross-section of ERT–1

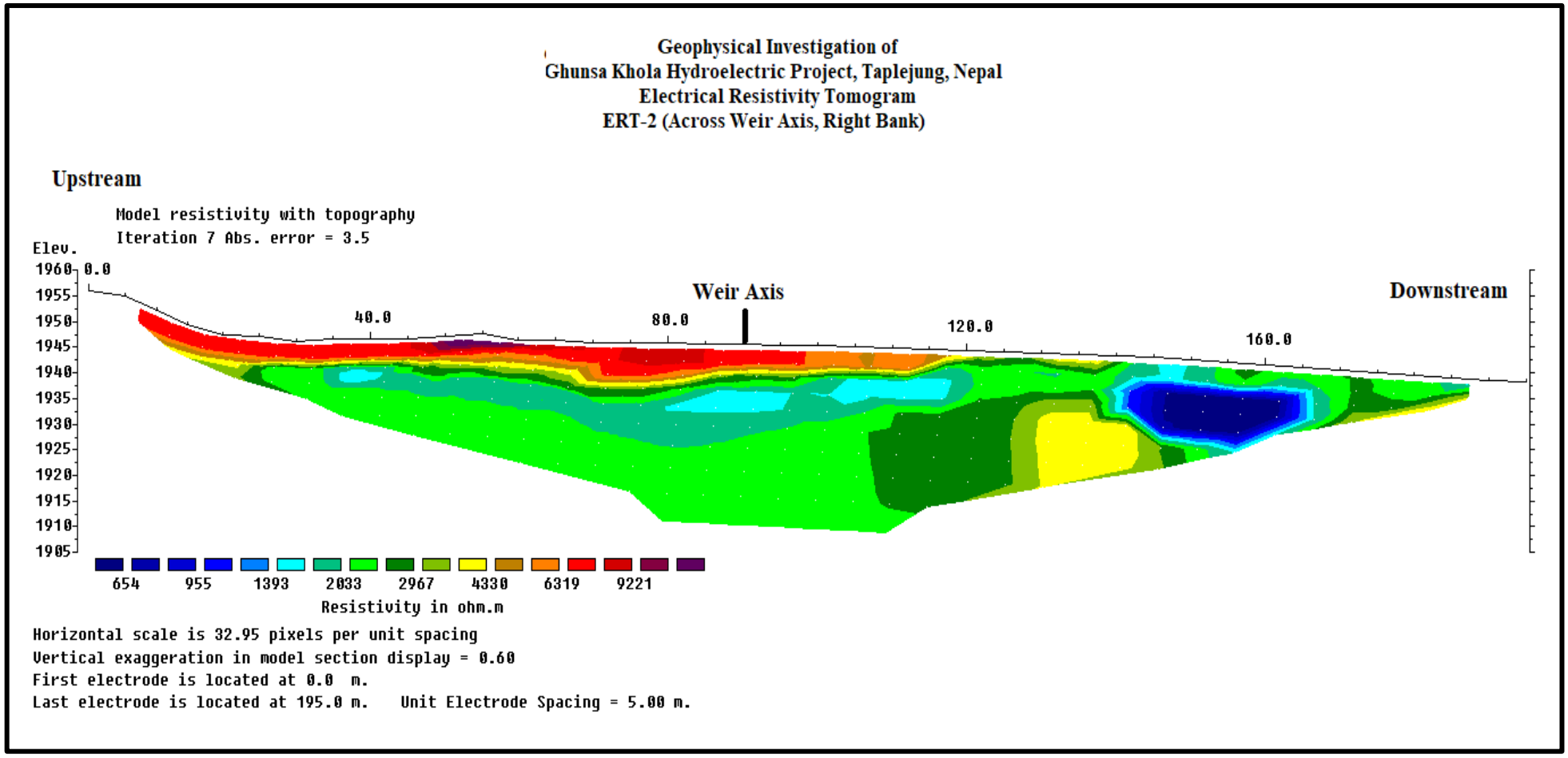


Figure 1-5: Electrical resistivity tomogram of ERT–2

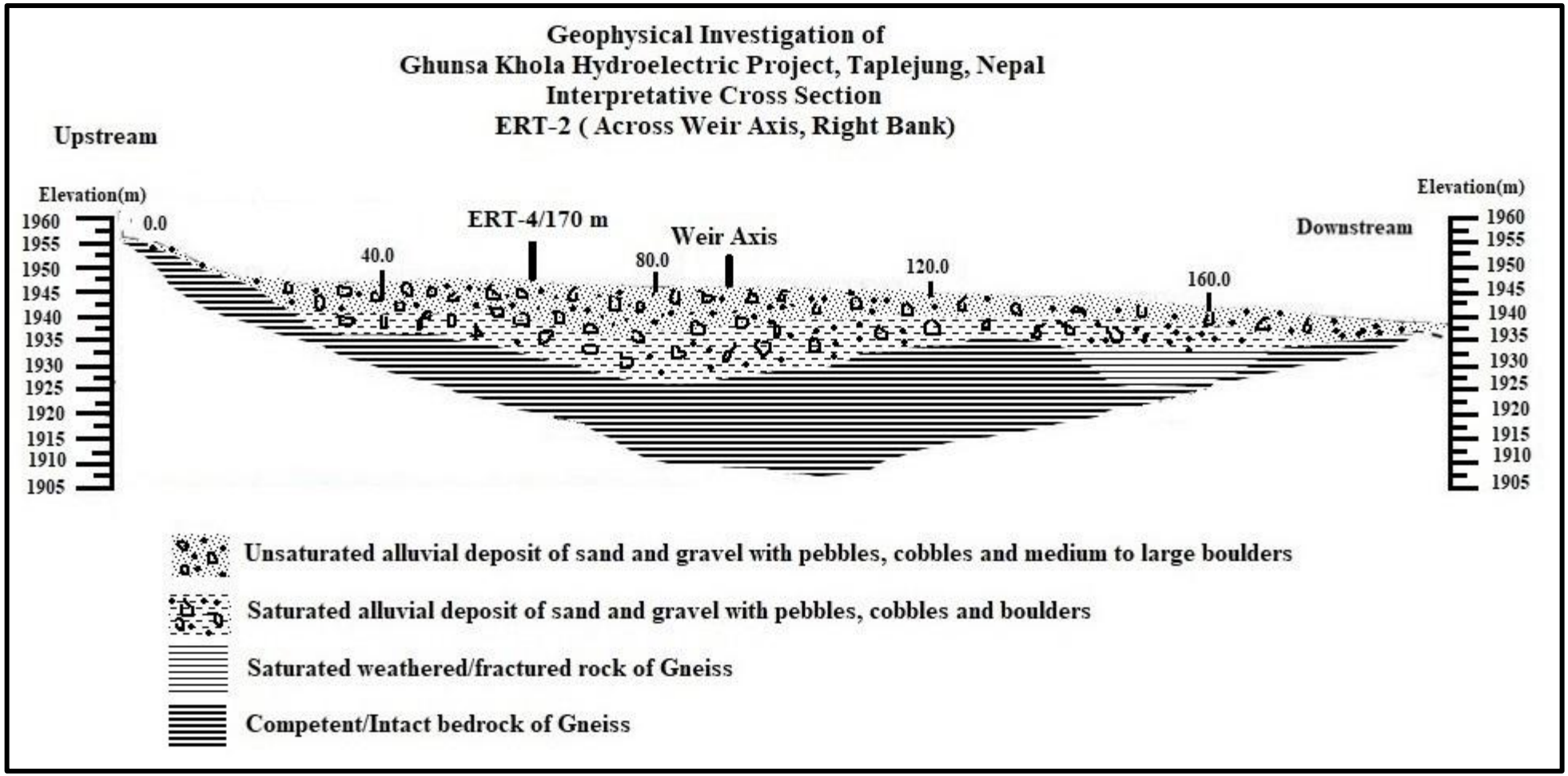


Figure 1-6: Interpretative cross-section of ERT–2

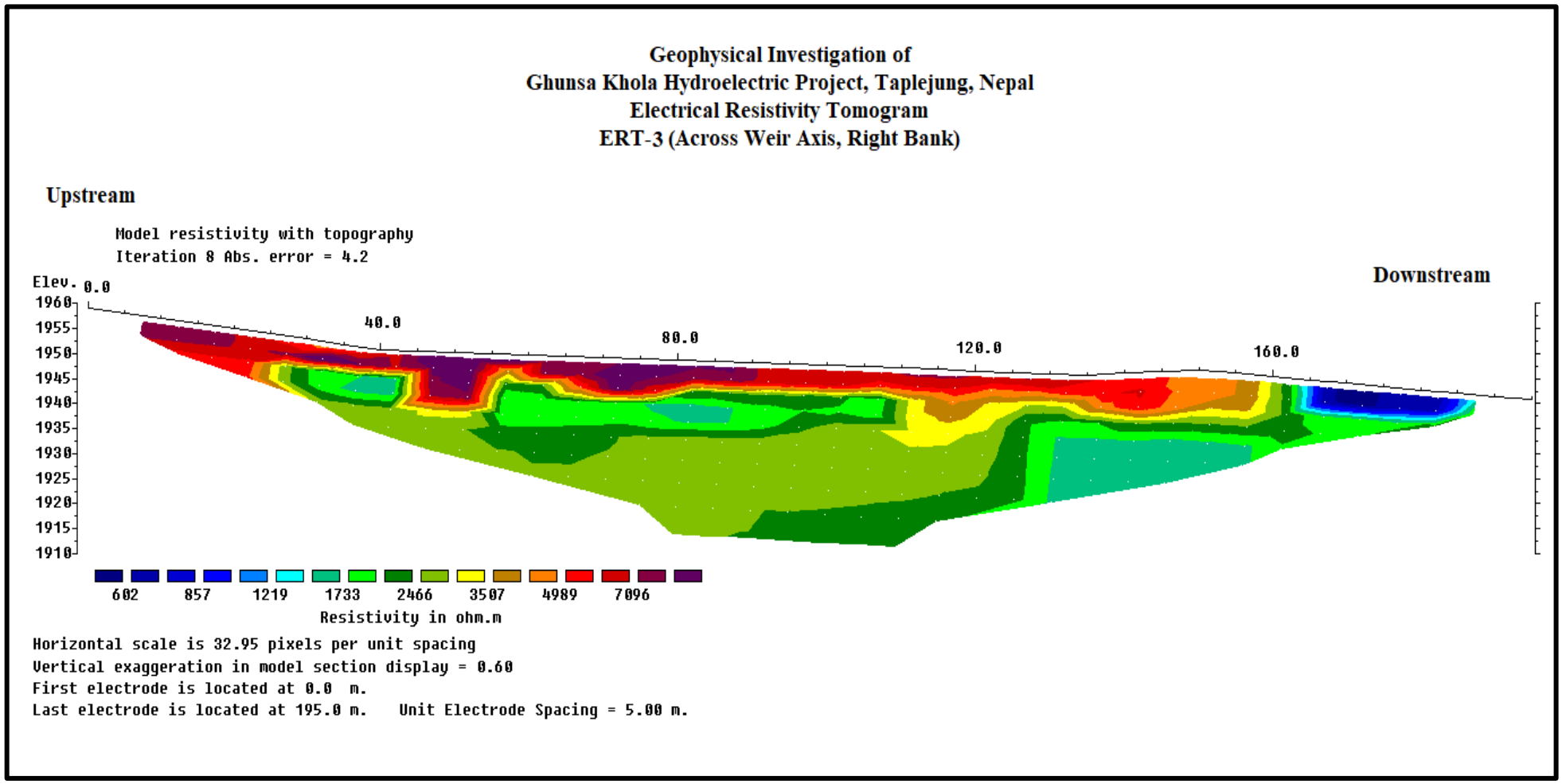


Figure 1-7: Electrical resistivity tomogram of ERT–3

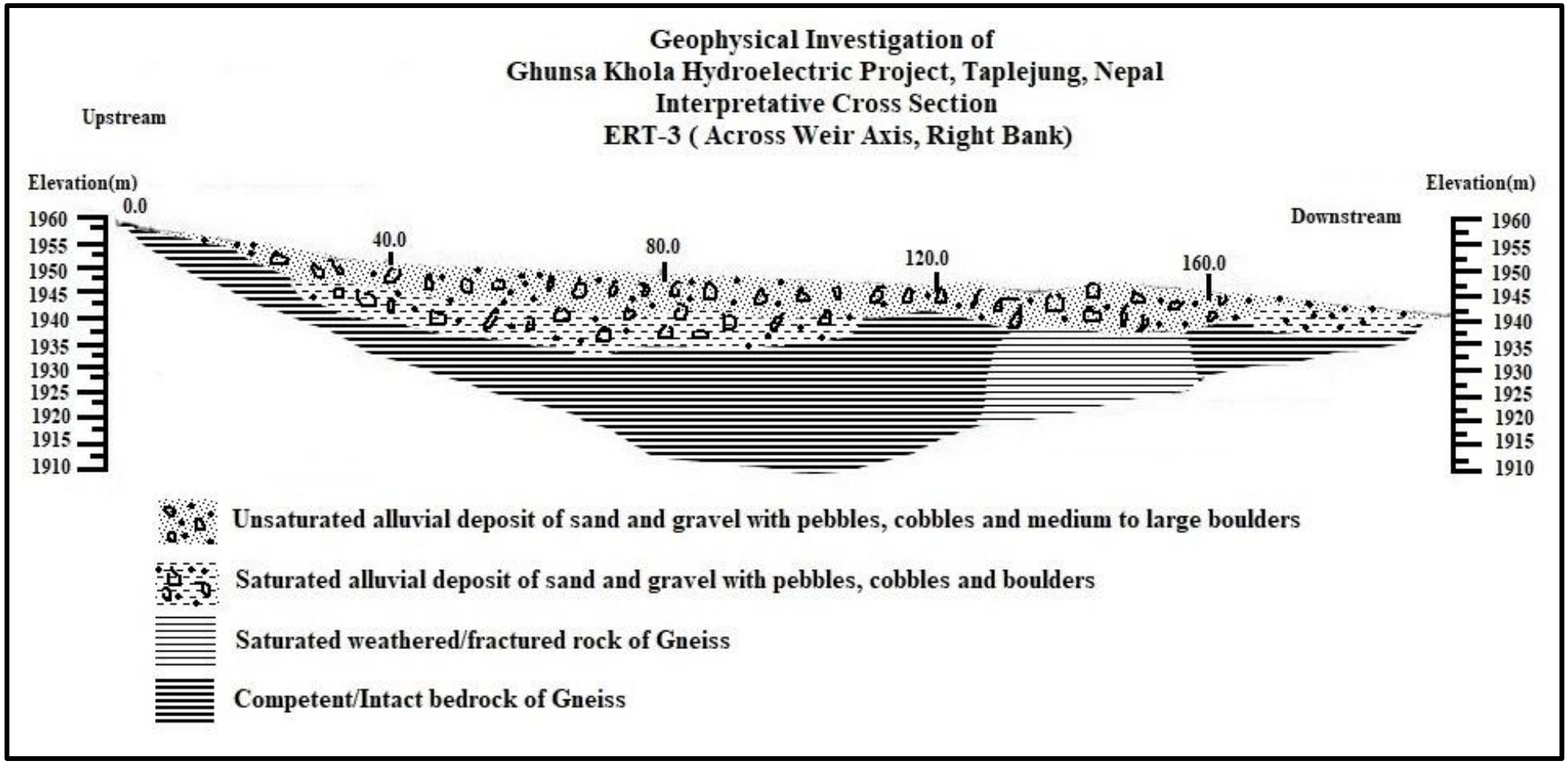


Figure 1-8: Interpretative cross-section of ERT–3

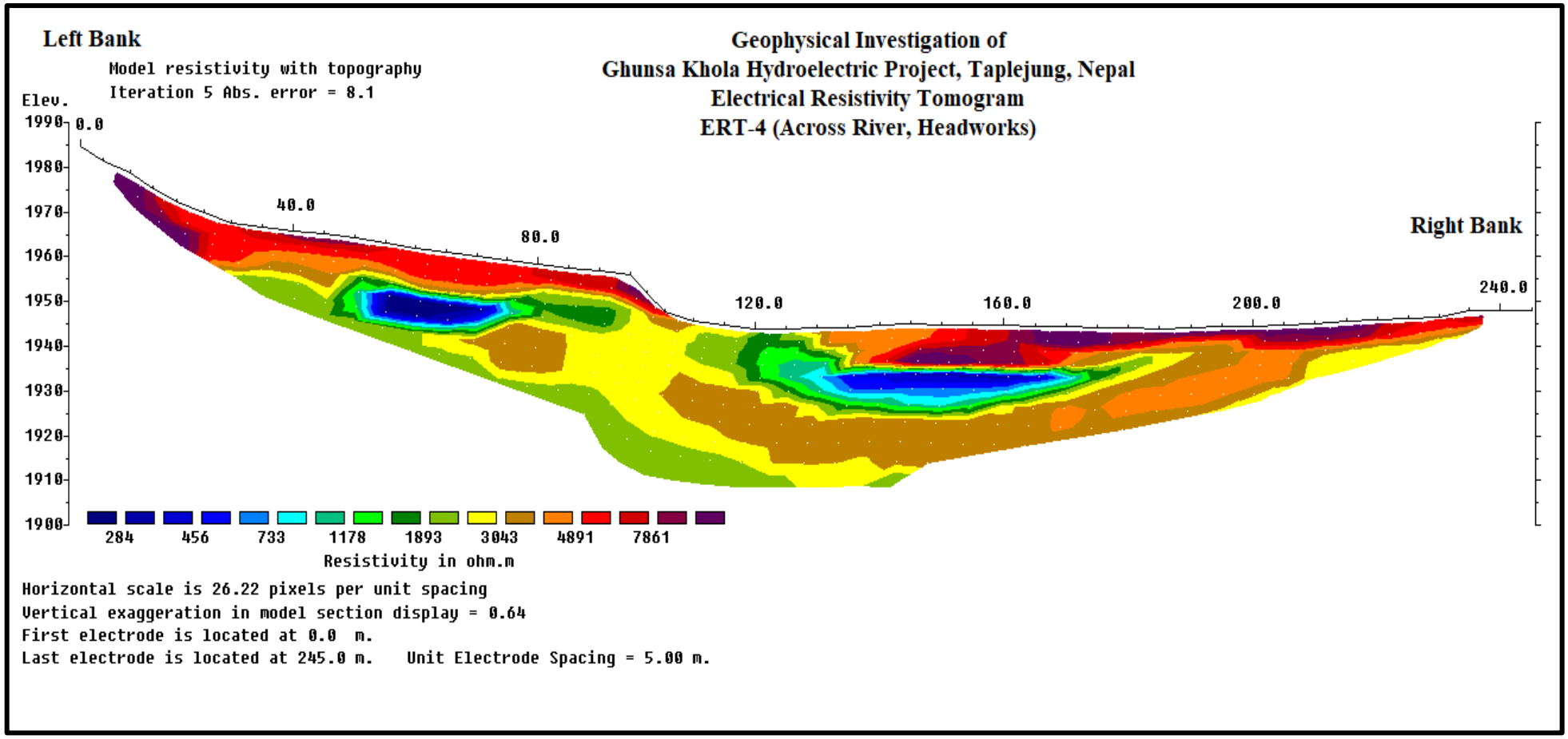


Figure 1-9: Electrical resistivity tomogram of ERT–4

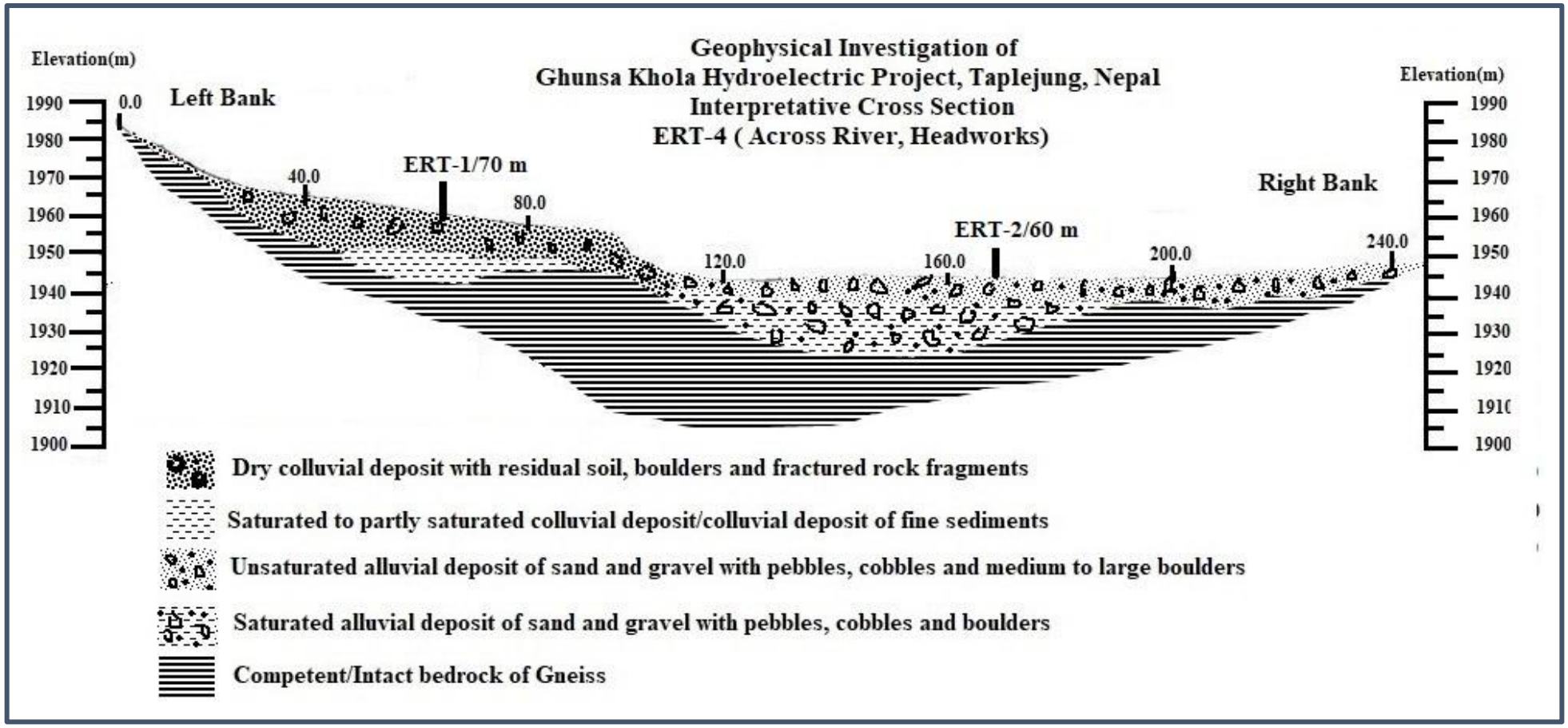


Figure 1-10: Interpretative cross-section of ERT–4

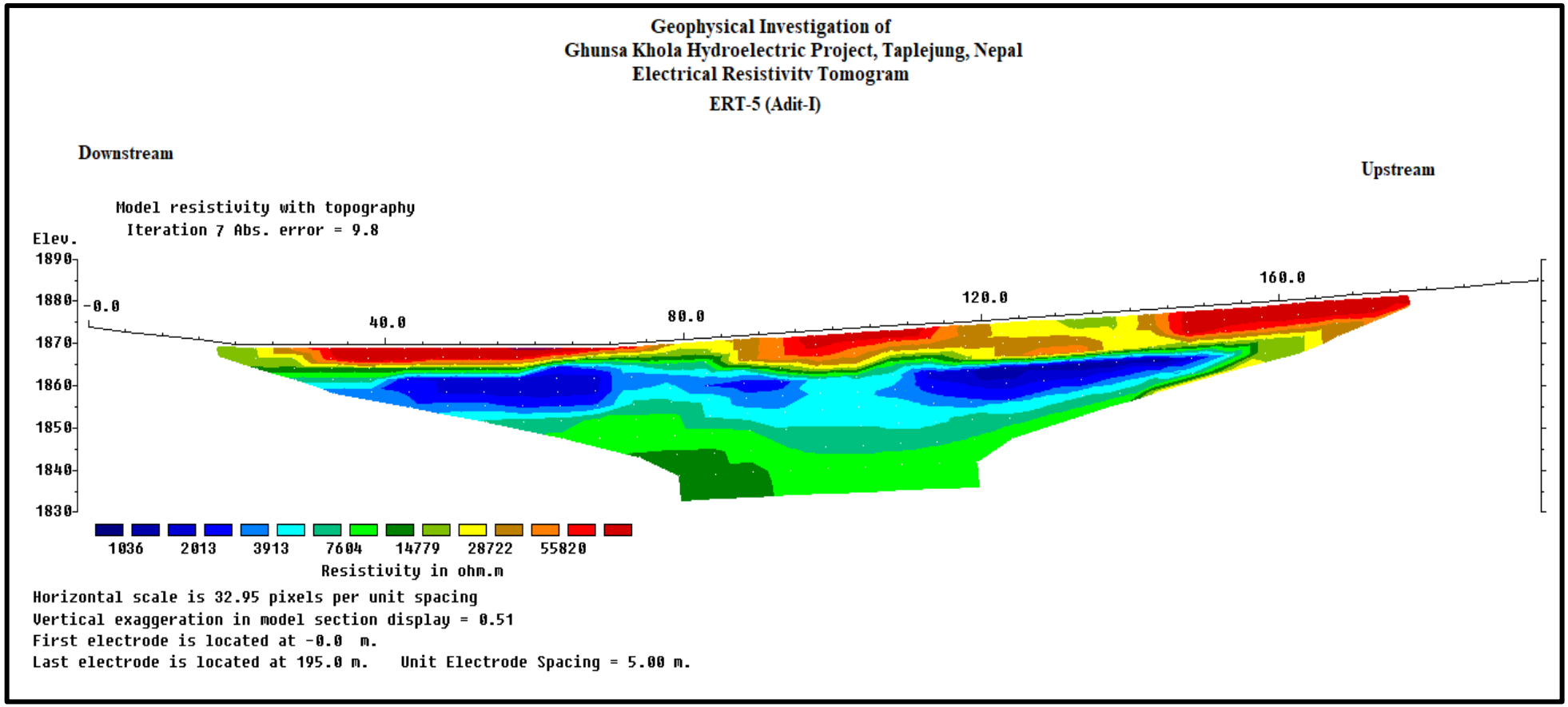


Figure 1-11: Electrical resistivity tomogram of ERT–5

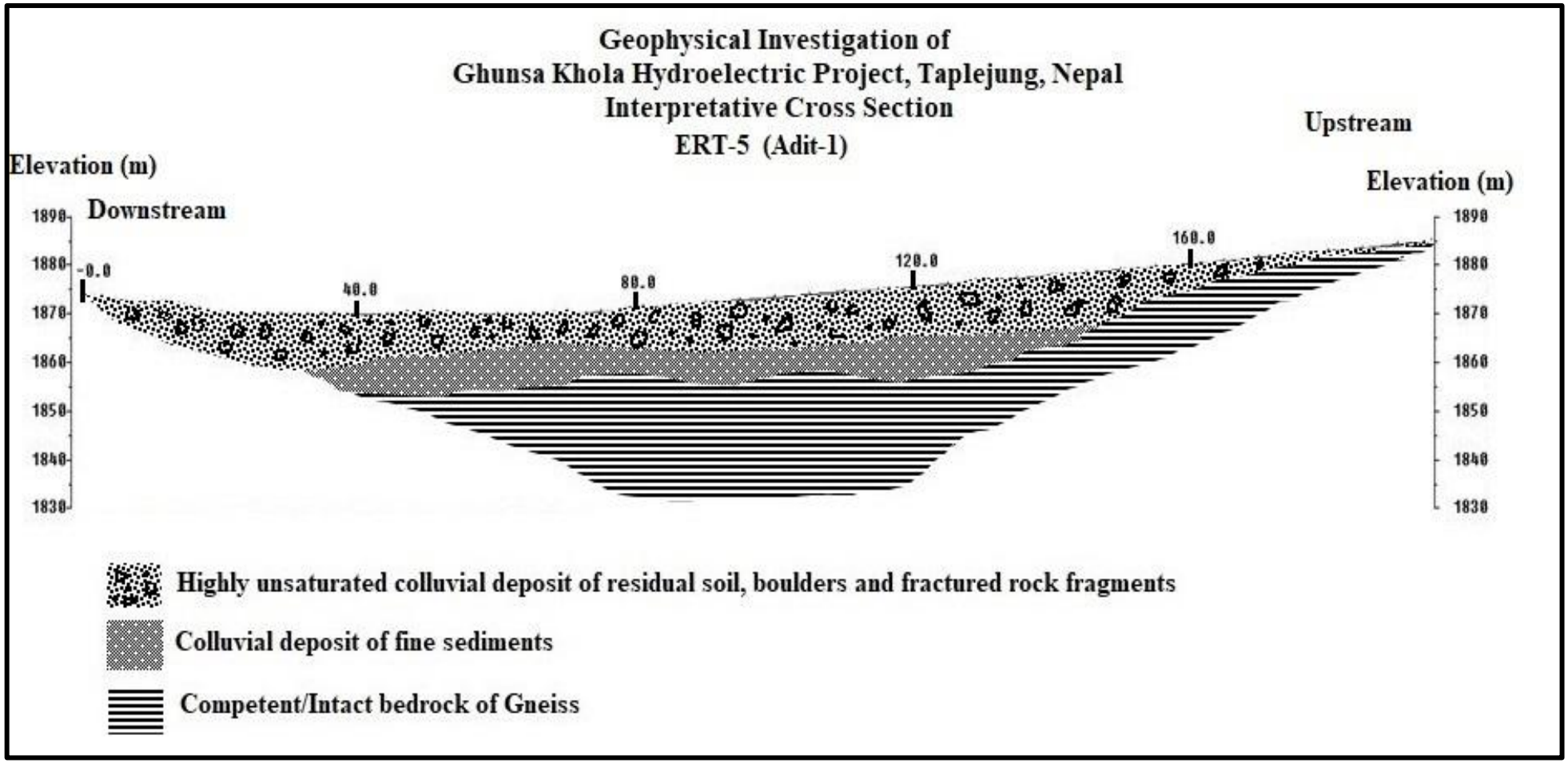


Figure 1-12: Interpretative cross-section of ERT–5

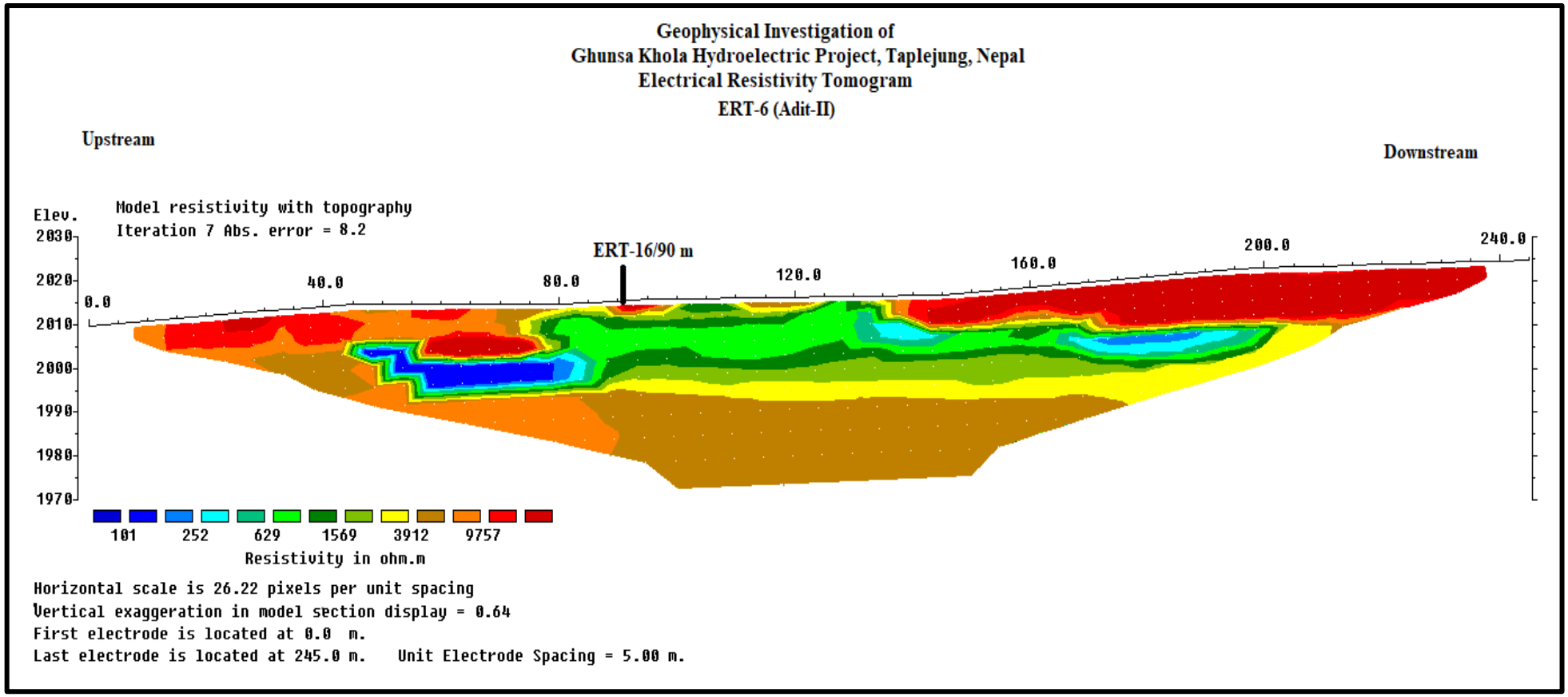


Figure 1-13: Electrical resistivity tomogram of ERT–6

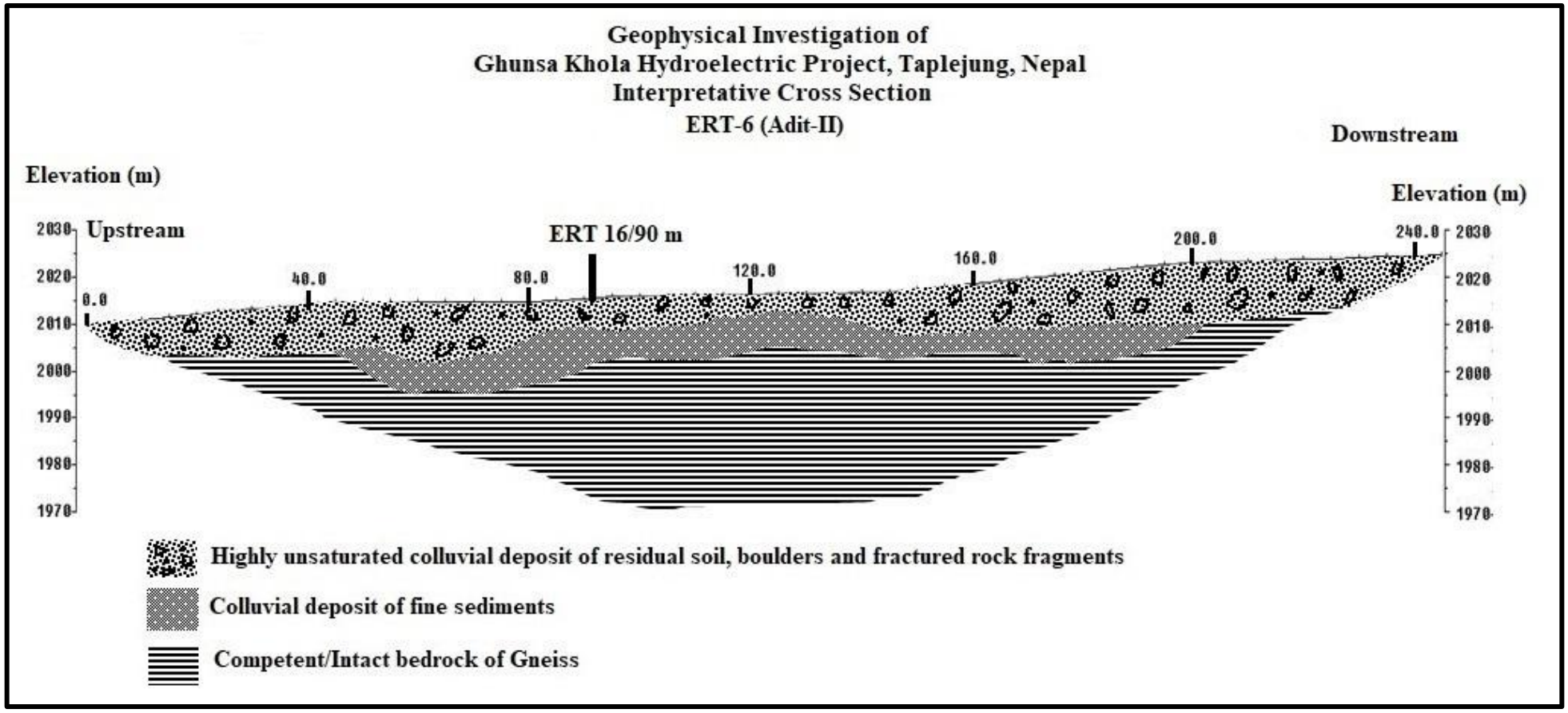


Figure 1-14: Interpretative cross-section of ERT–6

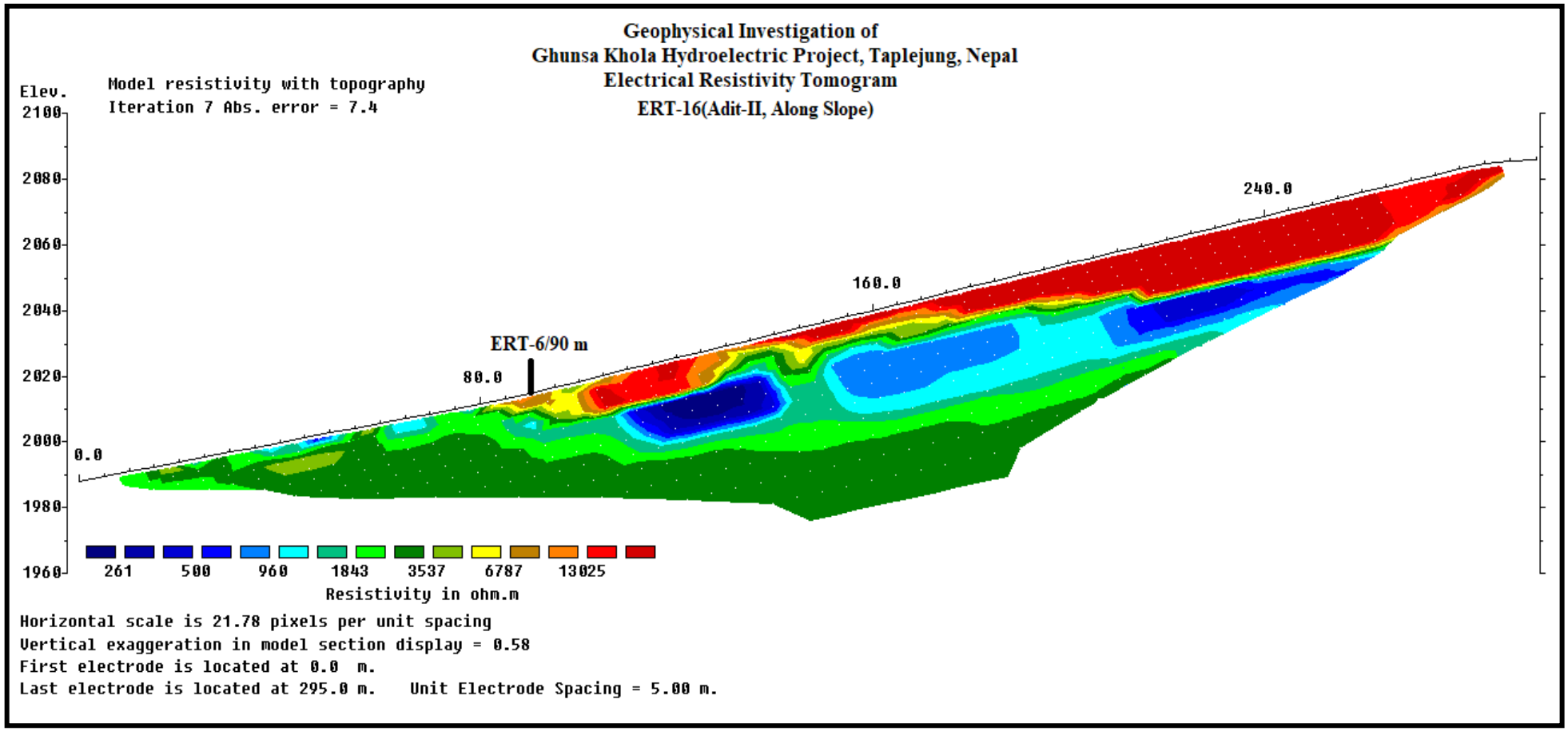


Figure 1-15: Electrical resistivity tomogram of ERT–16

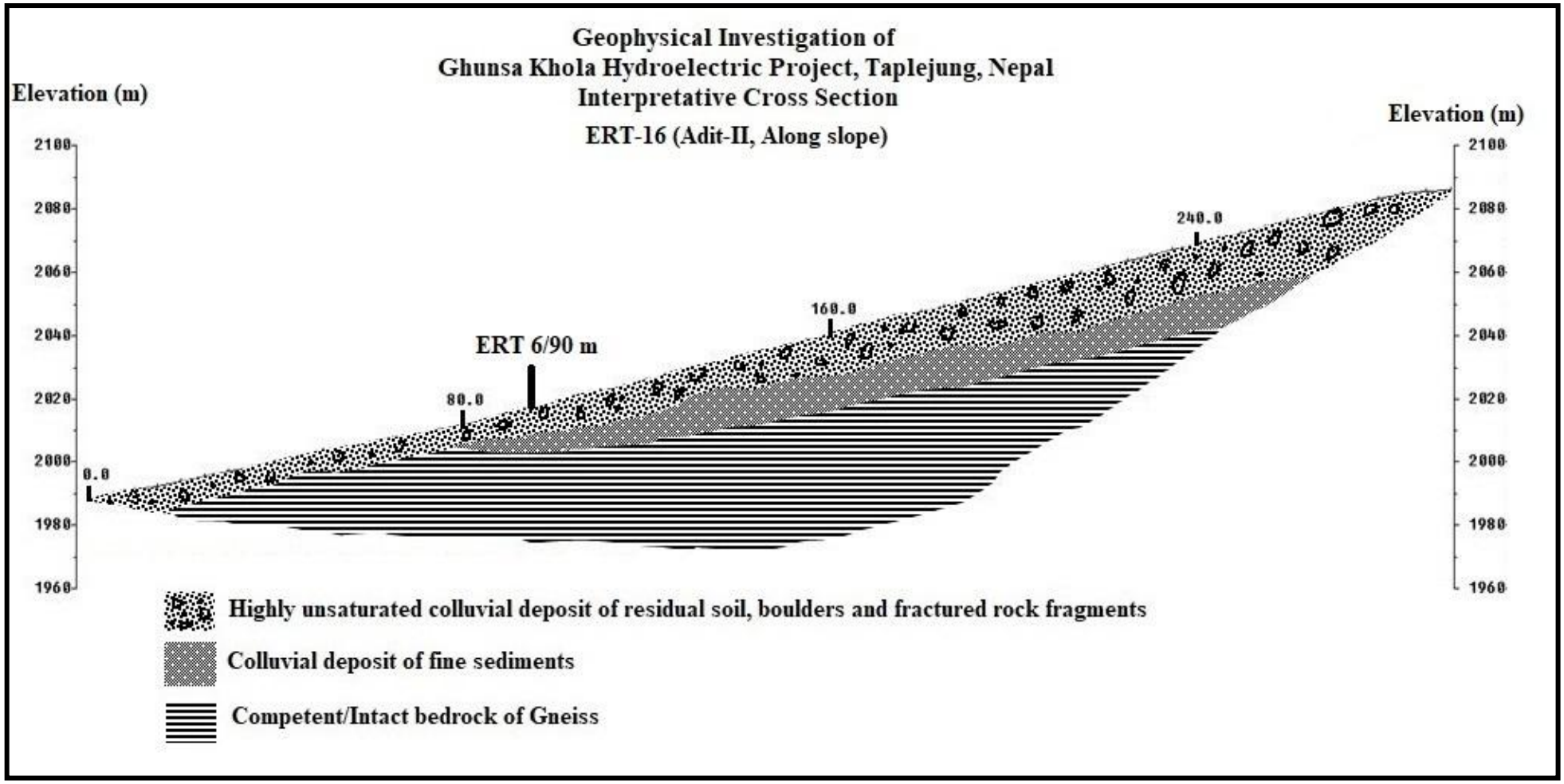


Figure 1-16: Interpretative cross-section of ERT-16

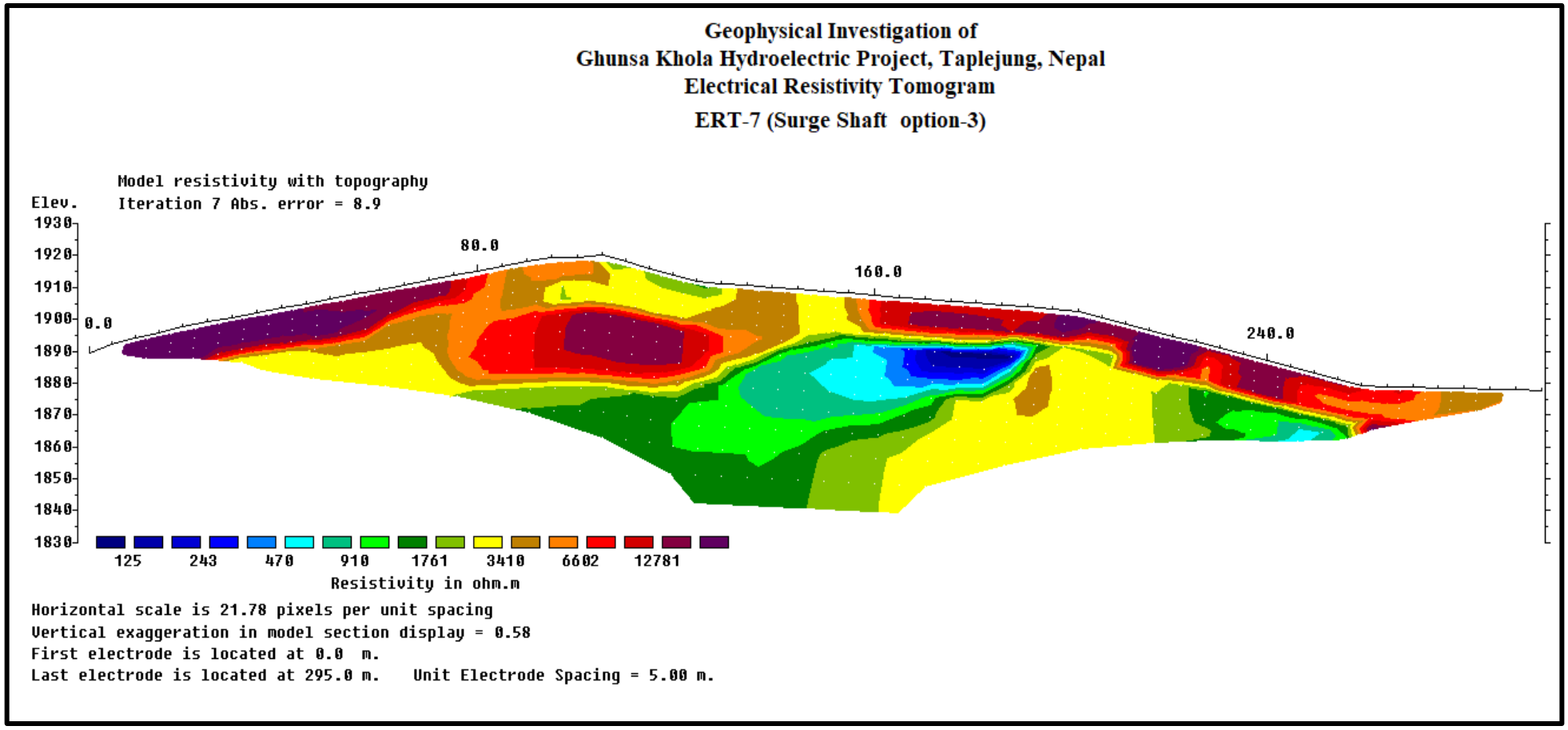


Figure 1-17: Electrical resistivity tomogram of ERT–7

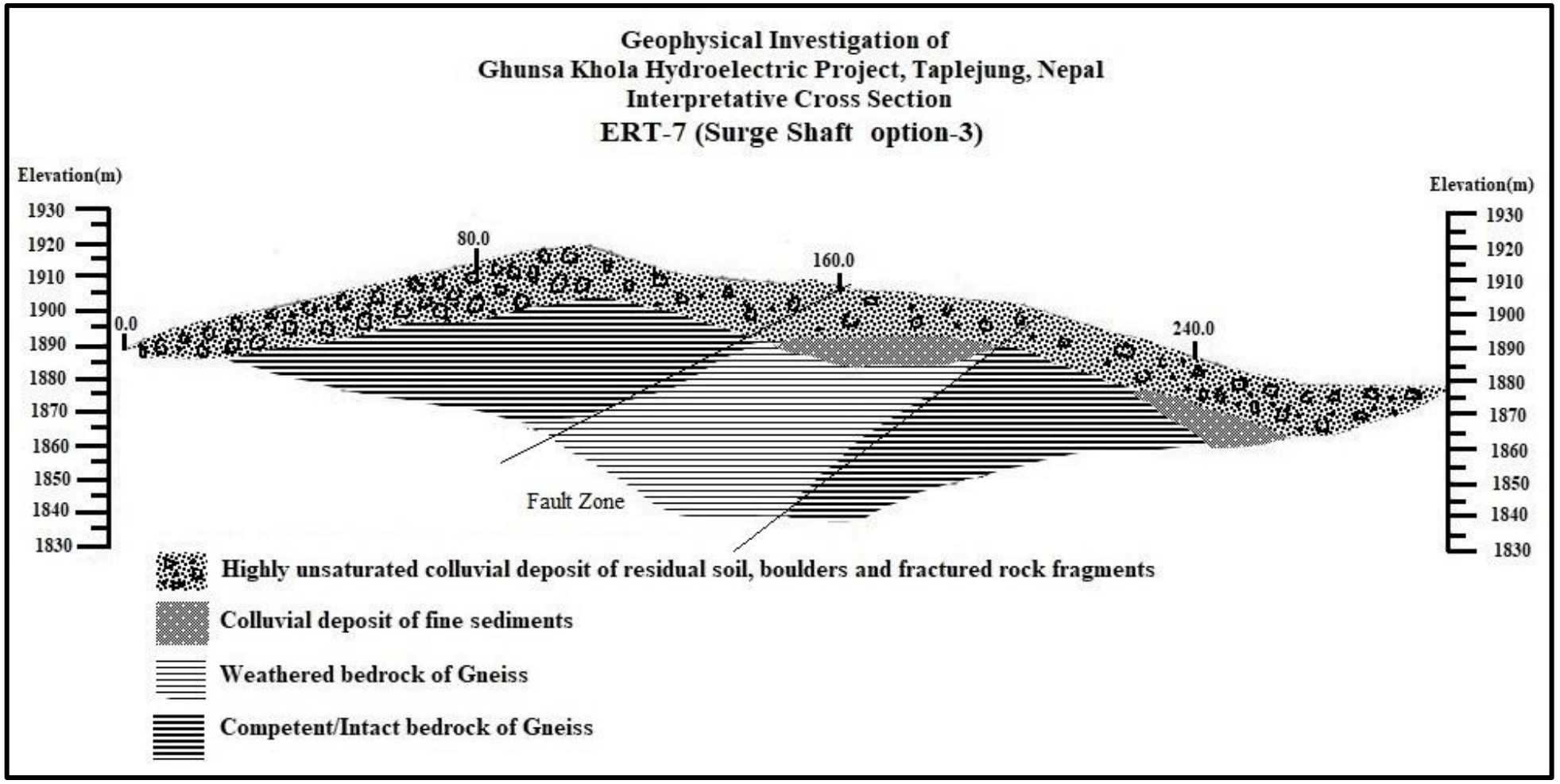


Figure 1-18: Interpretative cross-section of ERT–7

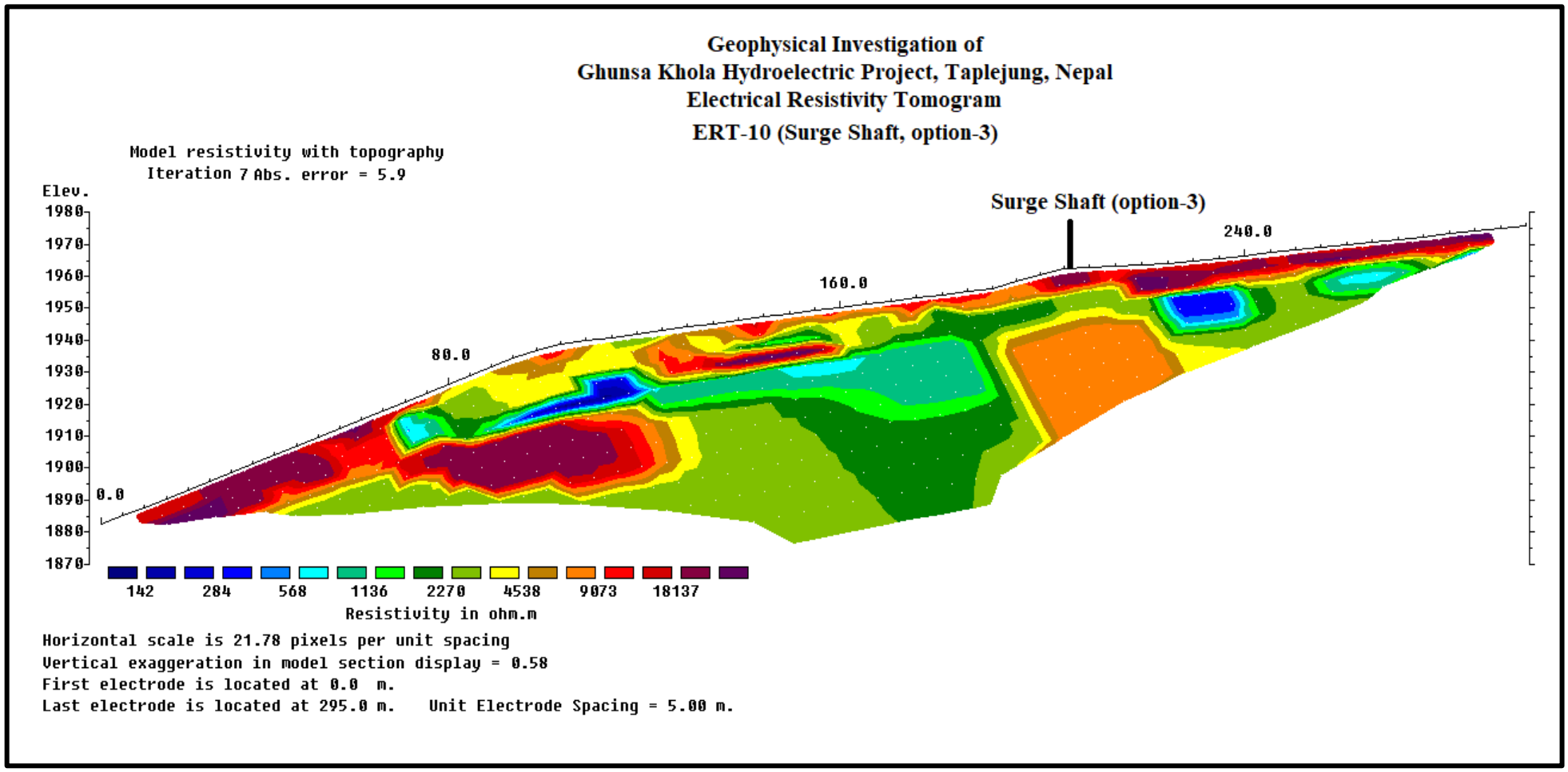


Figure 1-19: Electrical resistivity tomogram of ERT–10

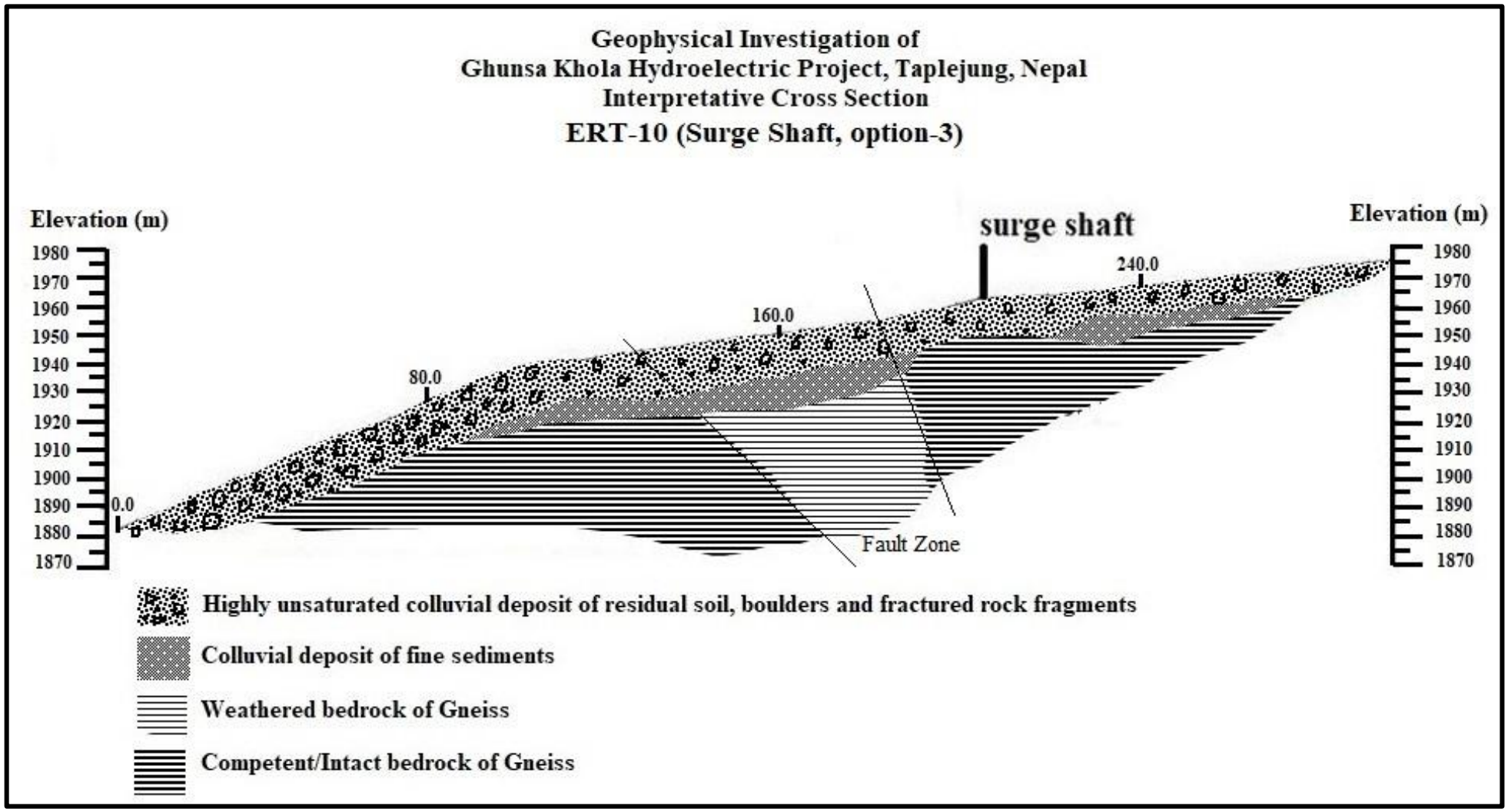


Figure 1-20: Interpretative cross-section of ERT–10

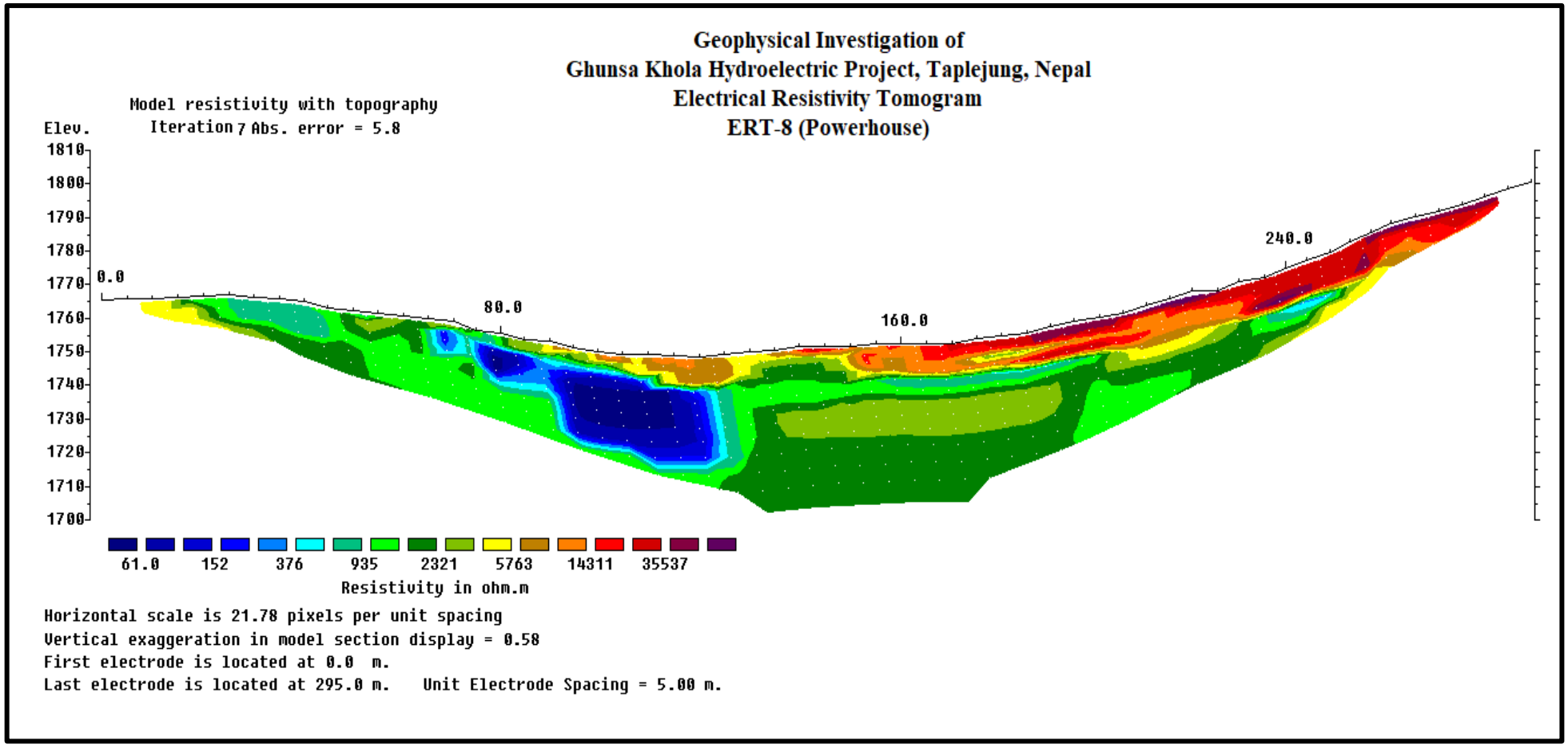


Figure 1-21: Electrical resistivity tomogram of ERT–8

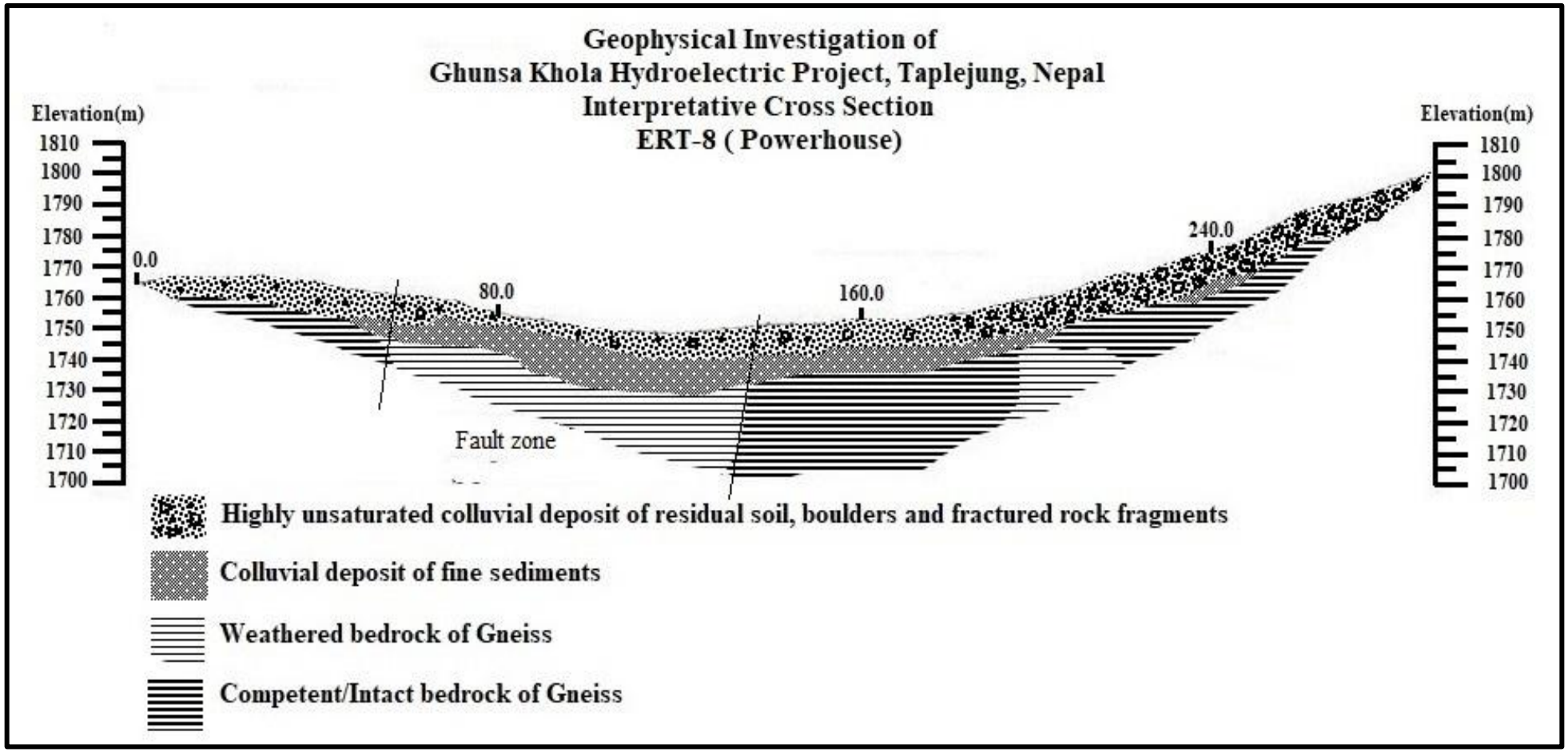


Figure 1-22: Interpretative cross-section of ERT –8

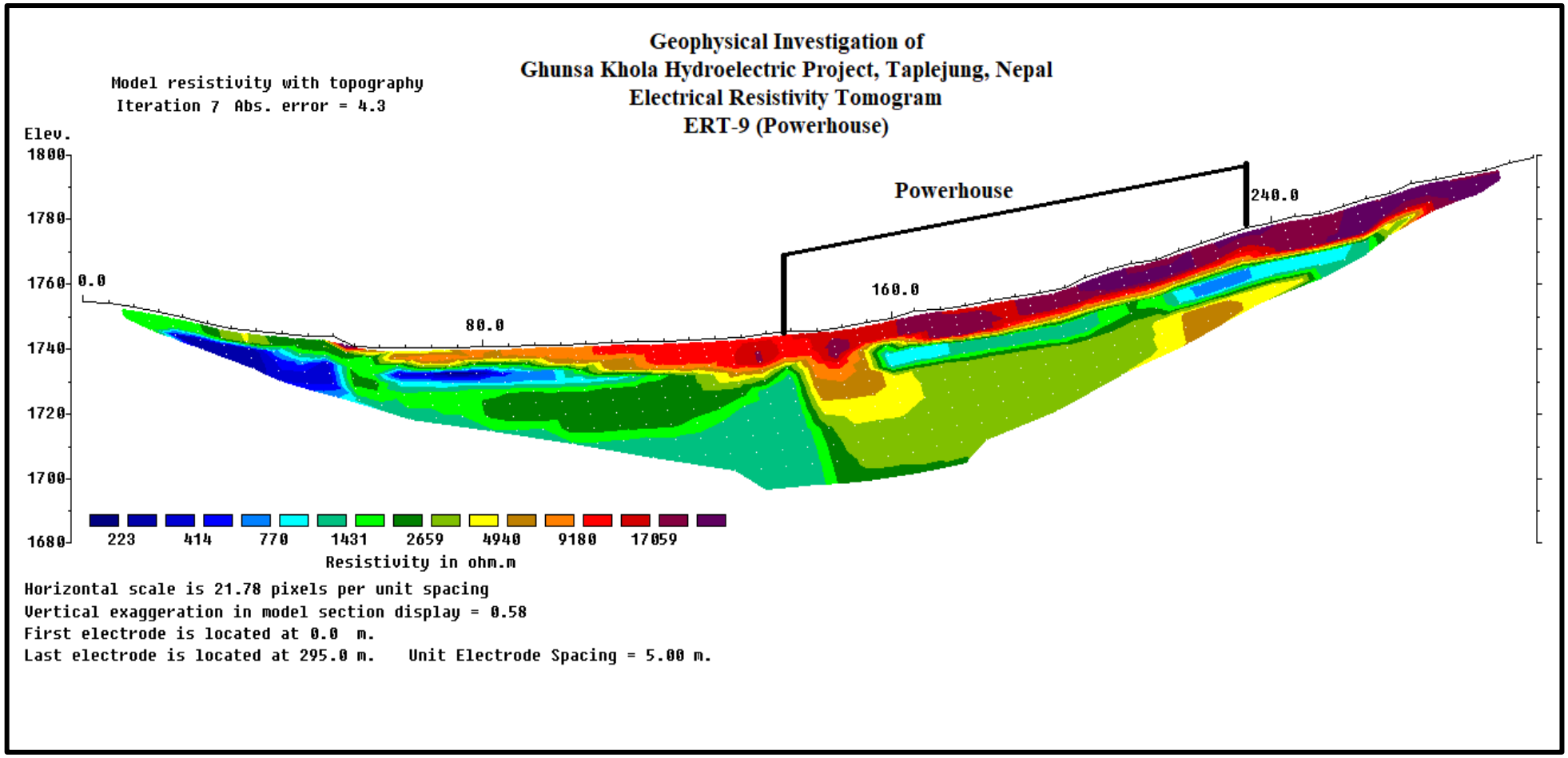


Figure 1-23: Electrical resistivity tomogram of ERT–9

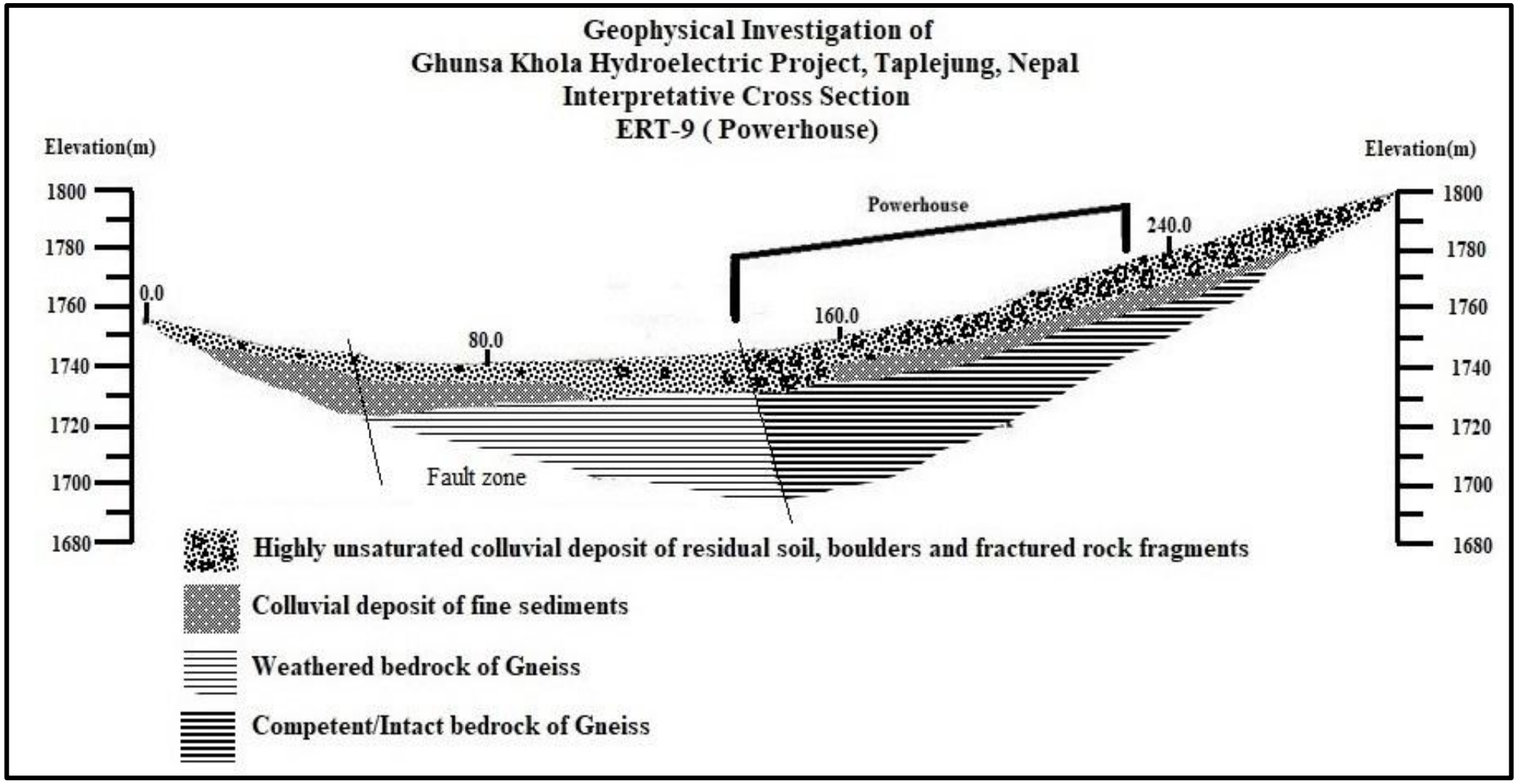


Figure 1-24: Interpretative cross-section of ERT –9

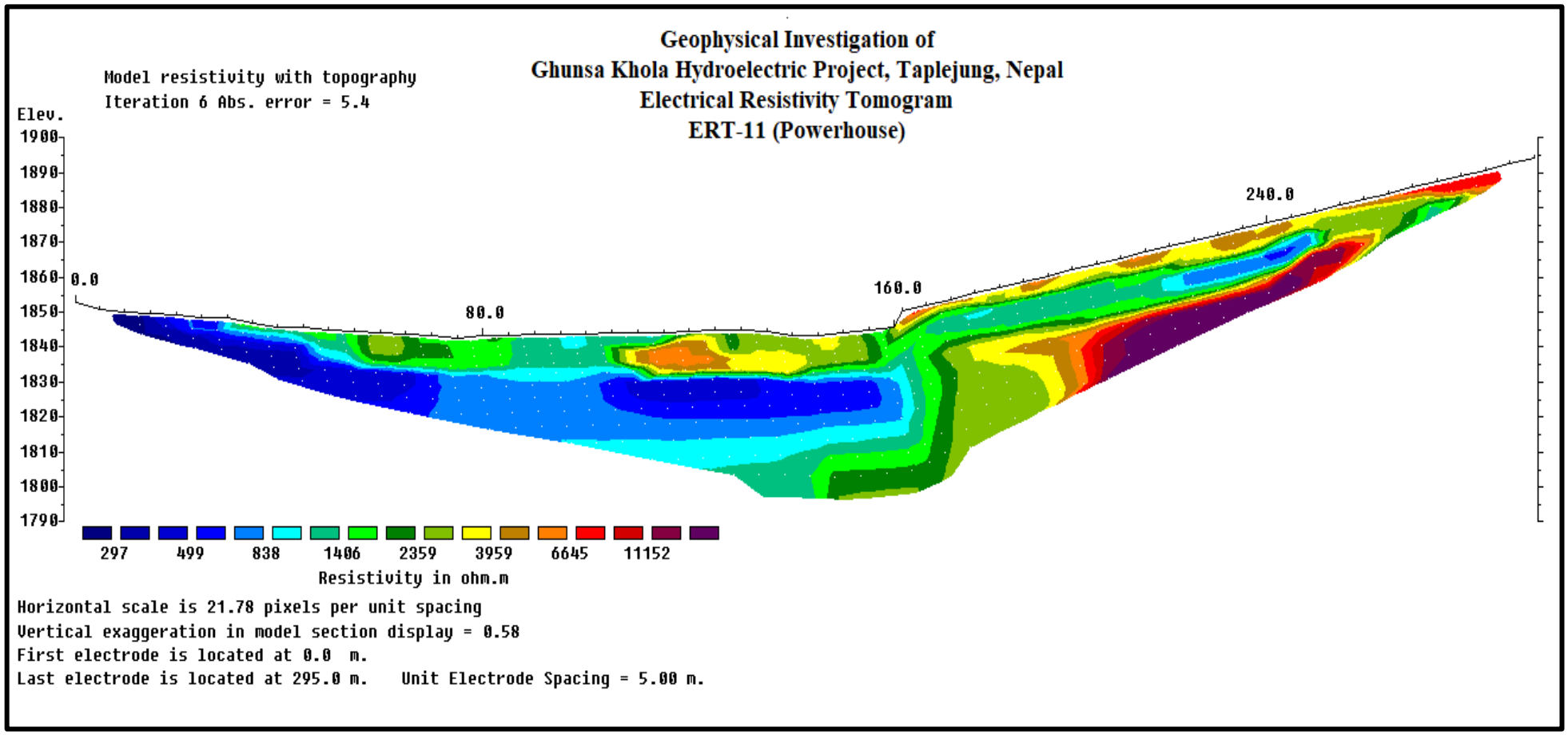


Figure 1-25: Electrical resistivity tomogram of ERT–11

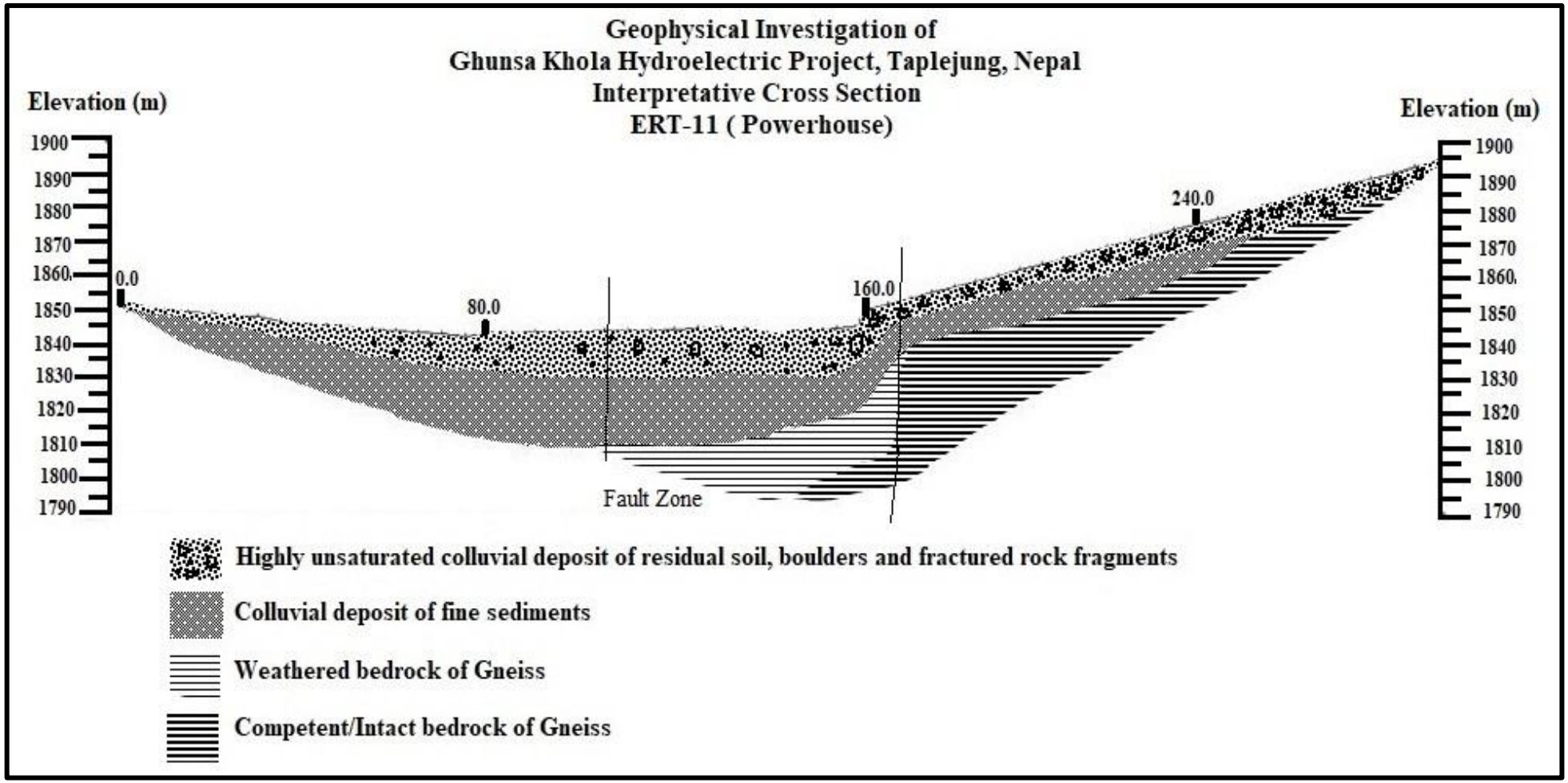


Figure 1-26: Interpretative cross-section of ERT –11

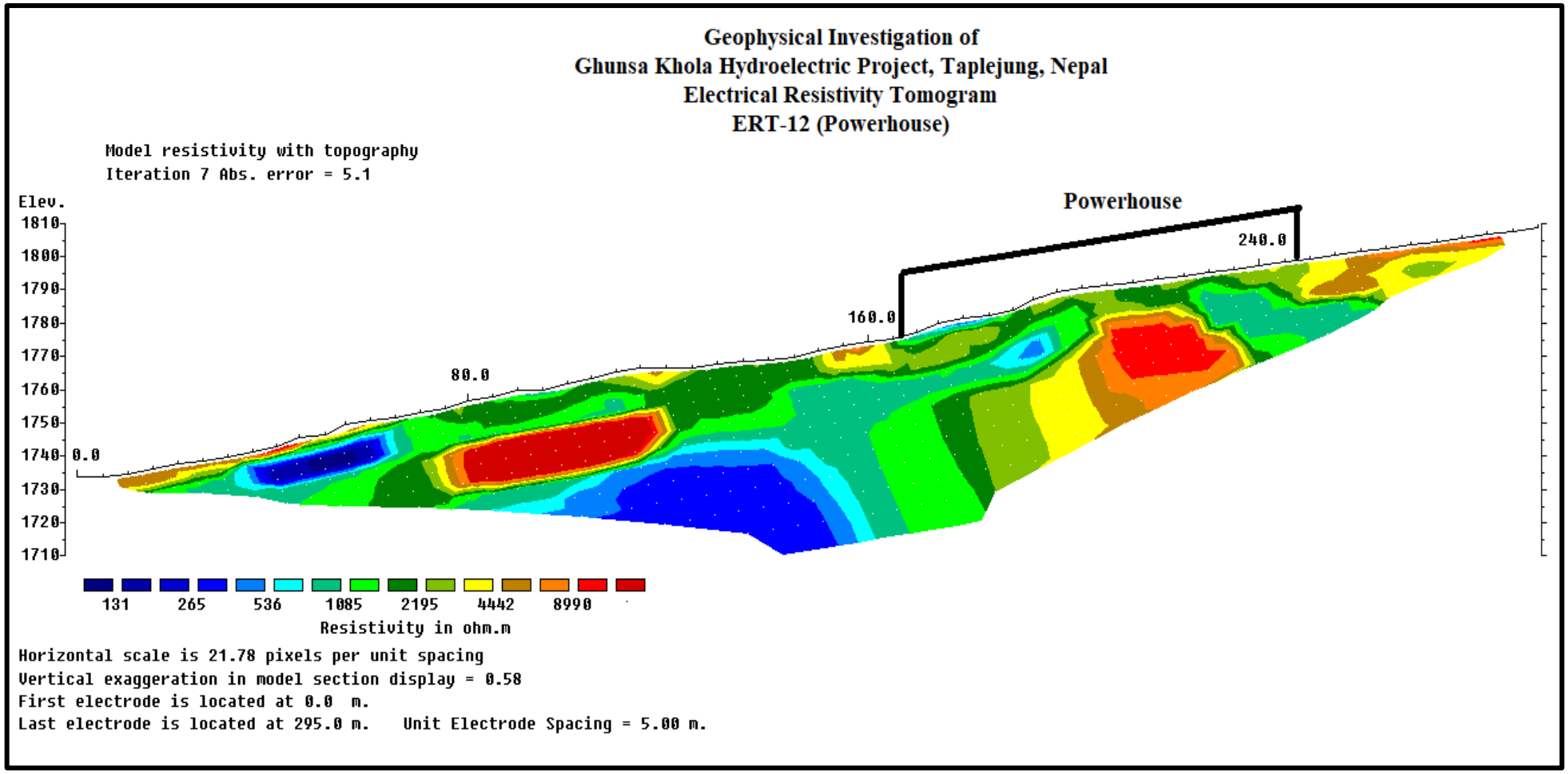


Figure 1-27: Electrical resistivity tomogram of ERT–12

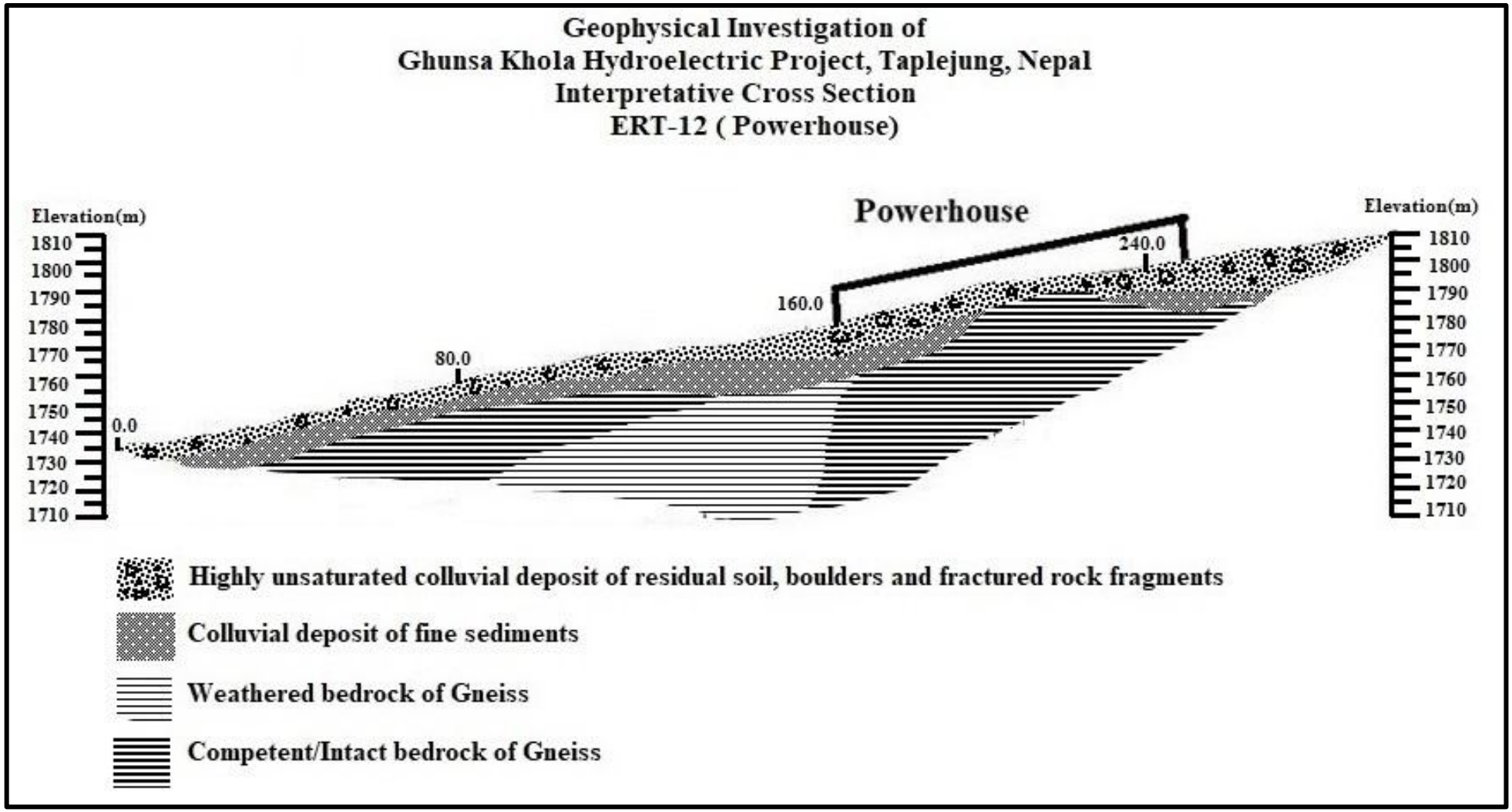


Figure 1-28: Interpretative cross-section of ERT–12

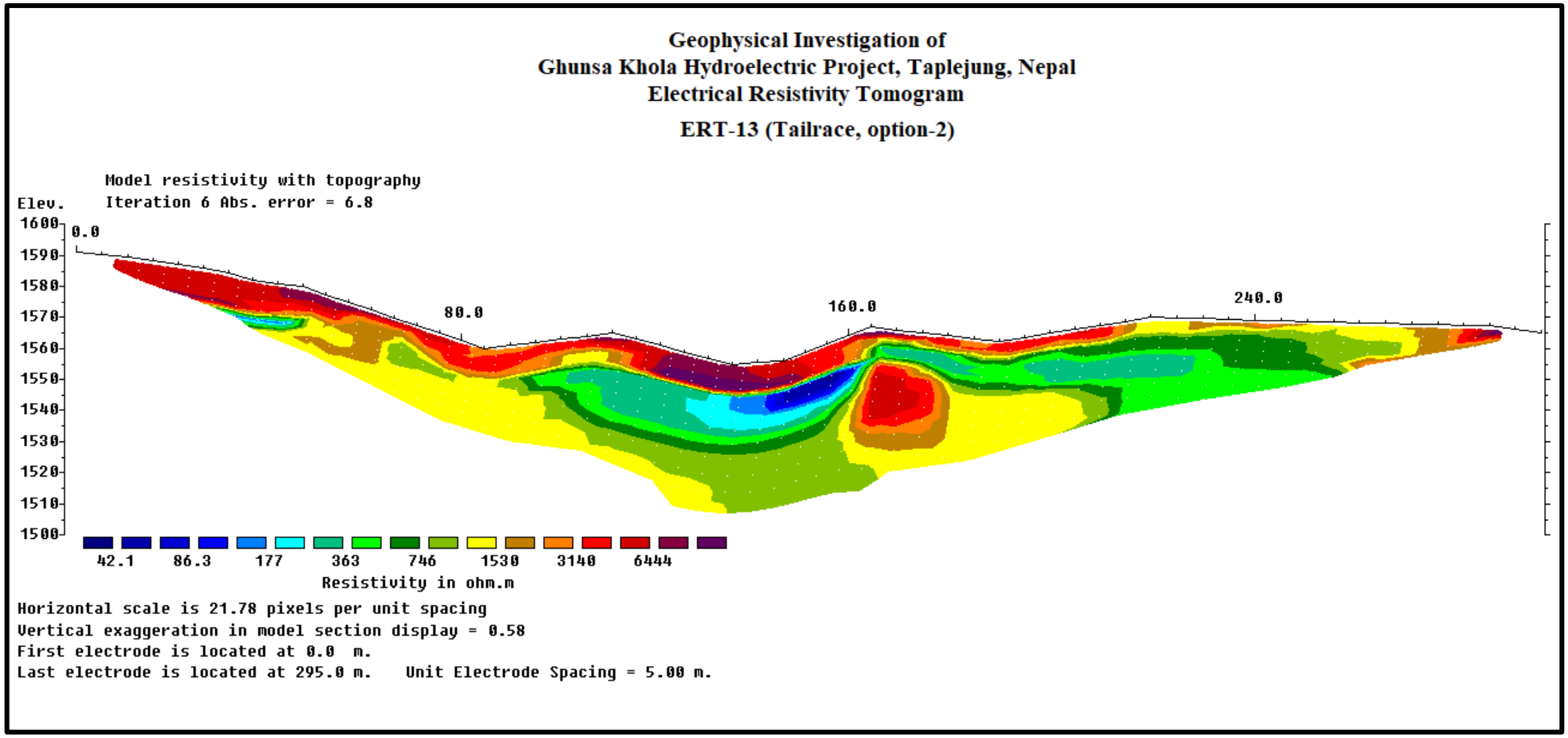


Figure 1-29: Electrical resistivity tomogram of ERT–13

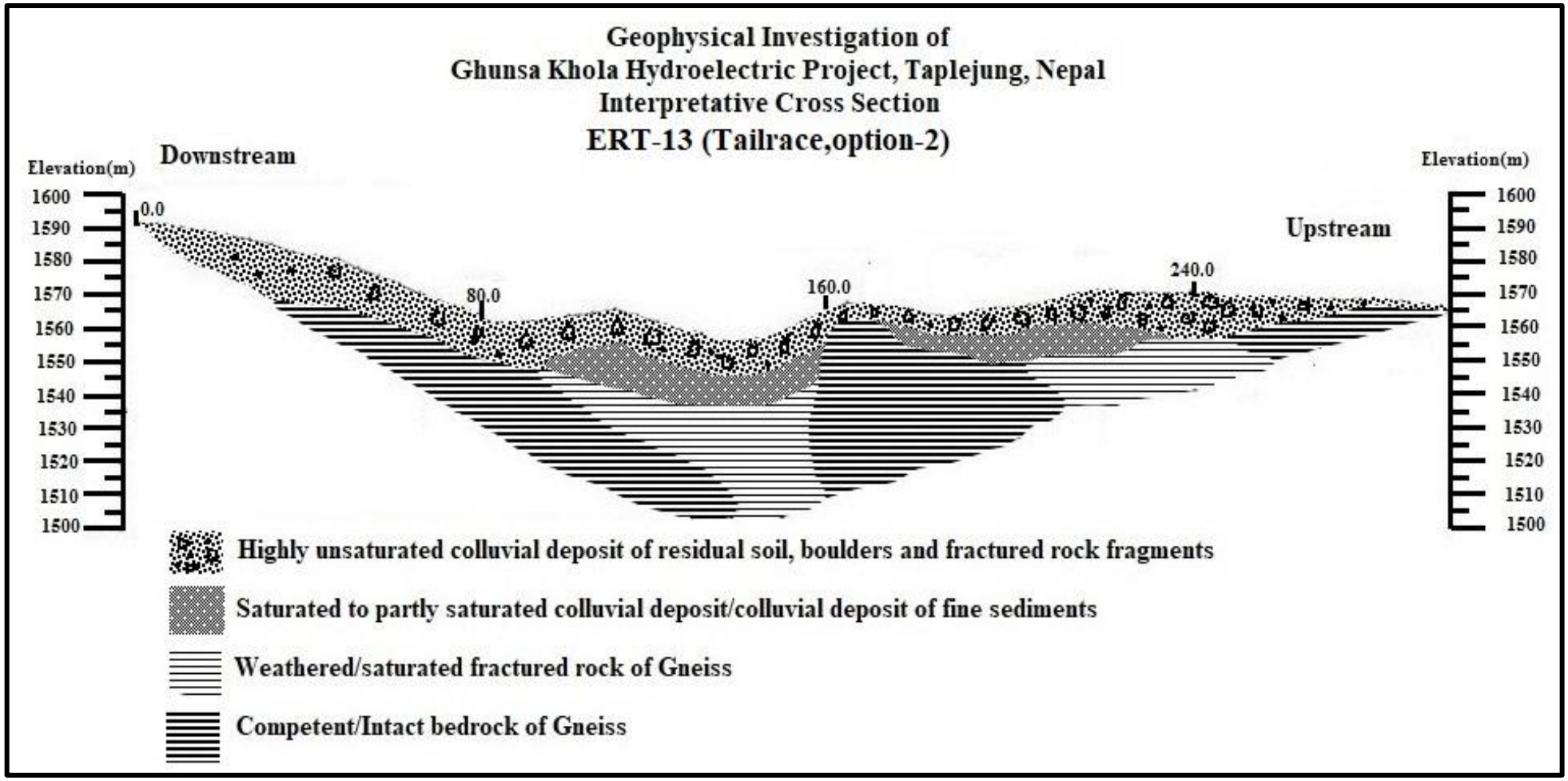


Figure 1-30: Interpretative cross-section of ERT –13

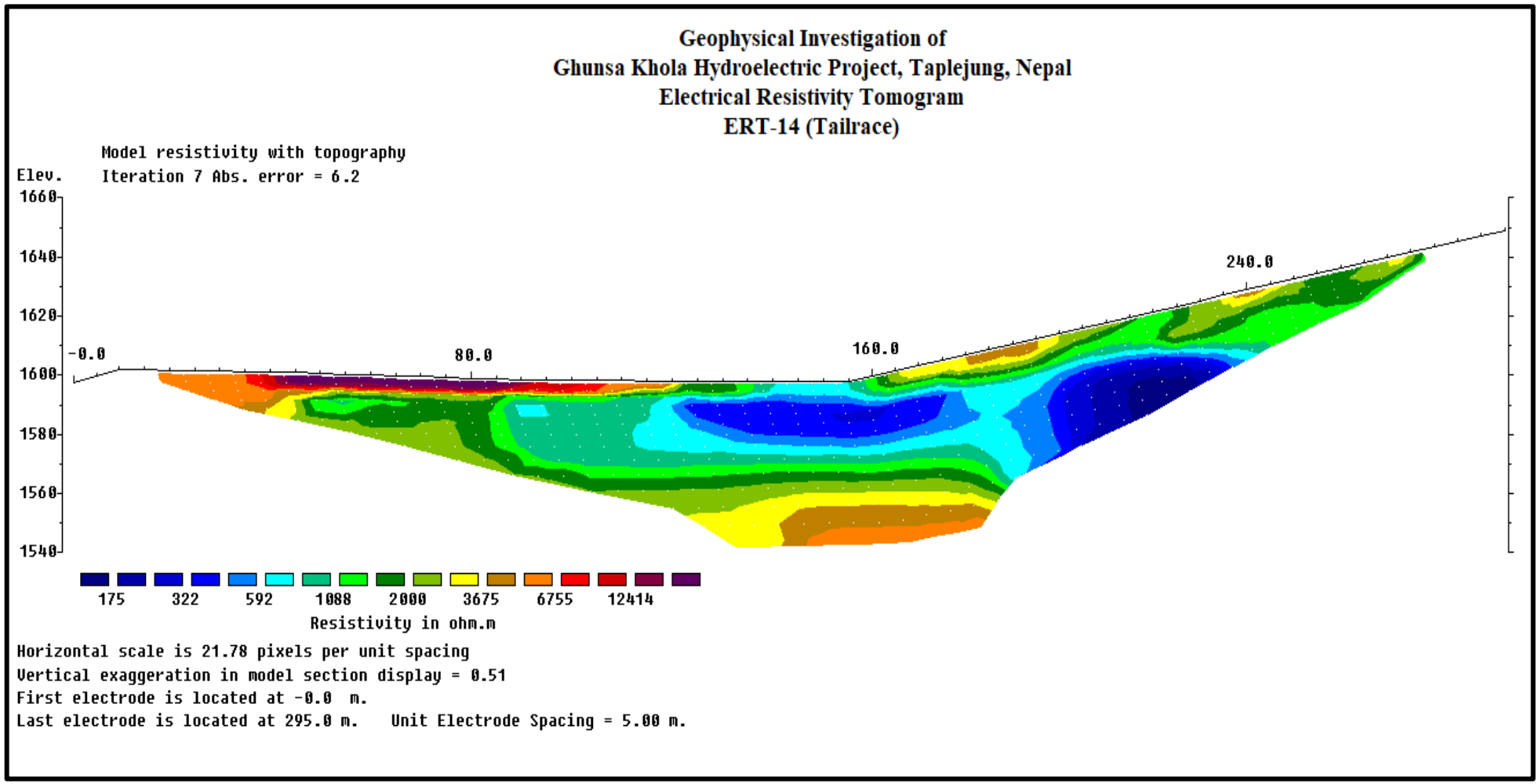


Figure 1-31: Electrical resistivity tomogram of ERT–14

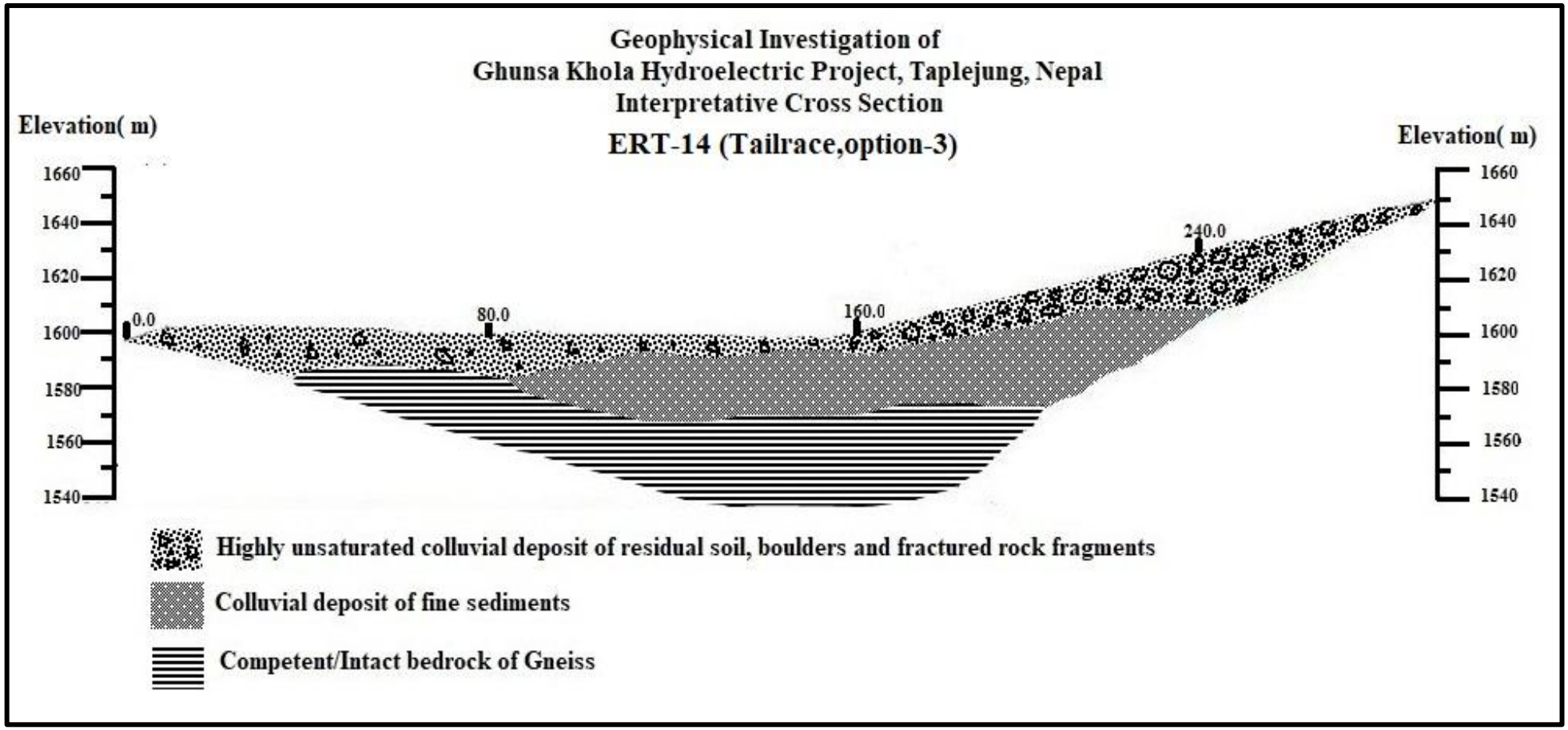


Figure 1-32: Interpretative cross-section of ERT –14

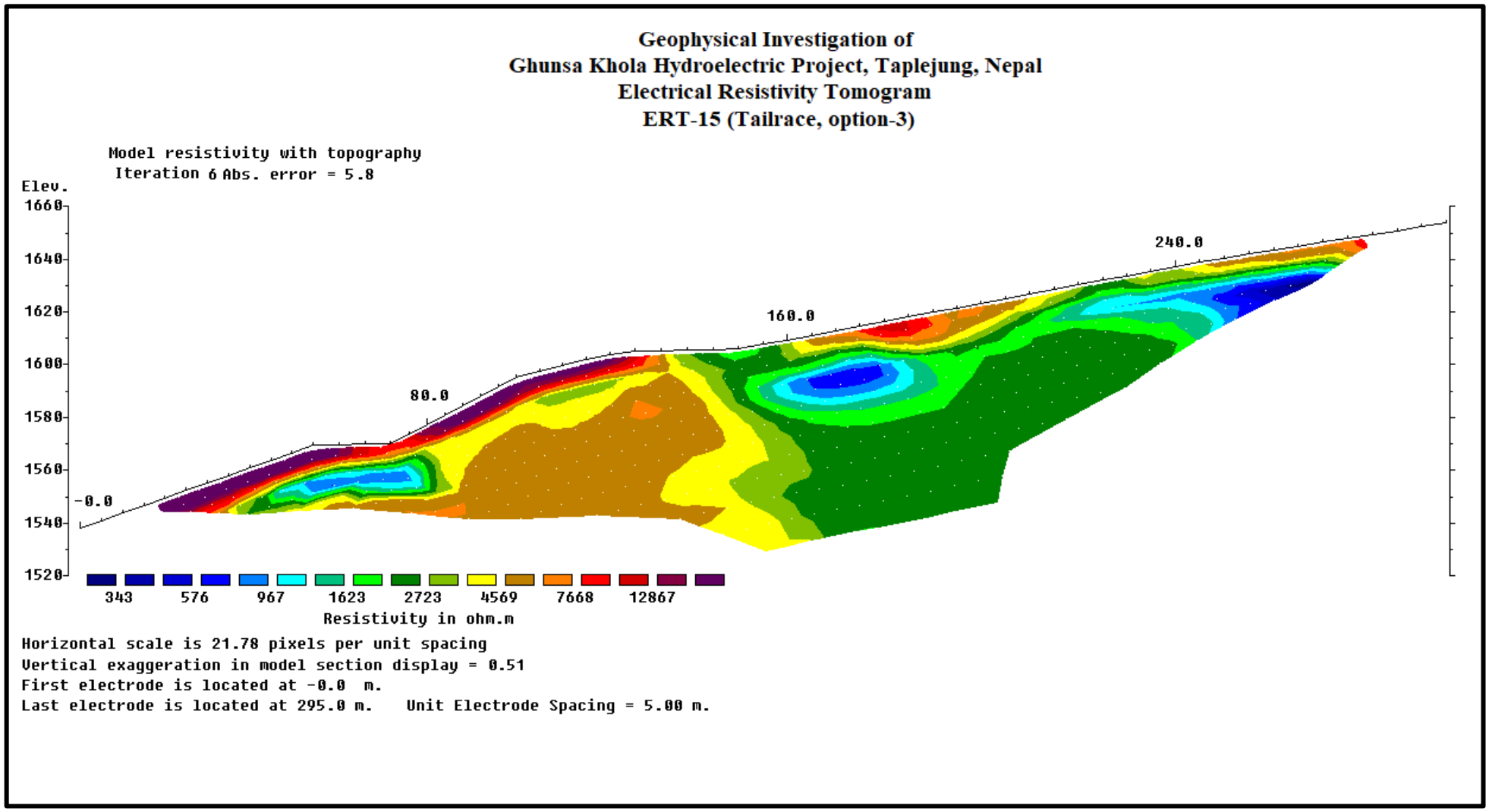


Figure 1-33: Electrical resistivity tomogram of ERT–15

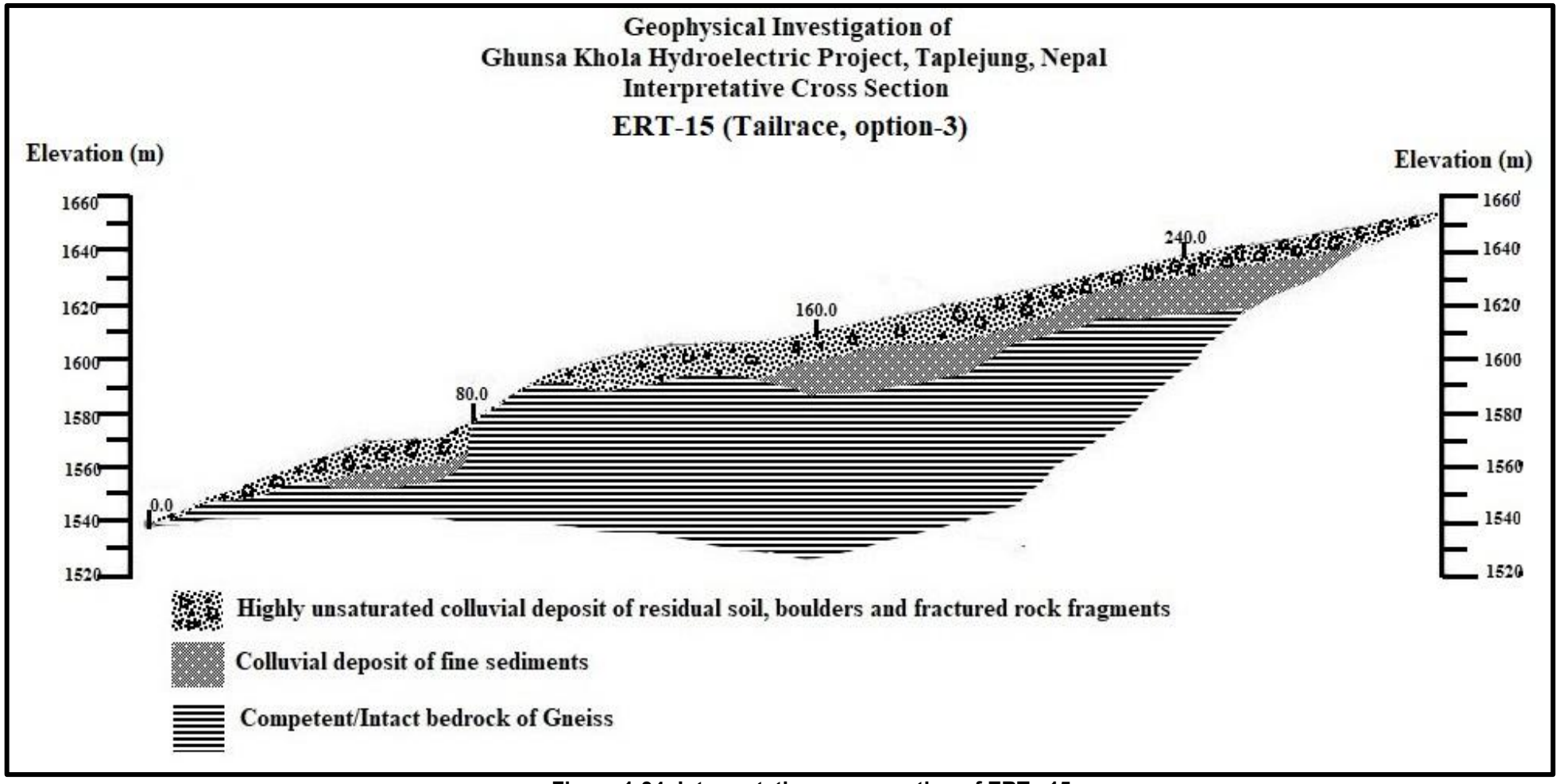


Figure 1-34: Interpretative cross-section of ERT –15

1.4 SUMMARY AND CONCLUSIONS

1.4.1 Summary

Detailed geophysical investigations were carried out at the sites of the project components, i.e. the headworks, adits, surge shaft, powerhouse and tailrace. General geological survey of these sites and the project area was also carried out during the geophysical survey, and the rock type, associate minerals, bedding structures and slope aspects and attitude of beds were studied.

1.4.1.1 Headworks Area

The headworks area lies in Lamatar, about 5.5 km from the confluence of Ghunsa Khola and Tamor River. It is located in a river valley with bedrock, residual soil and alluvial and colluvial deposits on both banks. At this site, the left bank consists of residual soil deposits with thick vegetation. Bedrock is exposed at some places on the steep slopes. The right bank consists of bedrocks of migmatite, migmatic gneiss and gneiss covered by alluvial/ colluvial materials. Alluvium deposit is observed along the stream. This deposit consists of medium to coarse grained sand to cobble size to huge blocks of boulders carried by the river.

Bedrock is expected to be found at a depth of 15 m at the weir axis on the left bank of Ghunsa Khola and at around 18 m on the right bank. It is expected to occur at varying depths of 10 to 18 m along the river on both banks. Bedrock is clearly exposed on the steep slope in the desanding basin area.

1.4.1.2 Adits

Adit-1 lies about 1 km south-west from the headworks, on the right bank of Ghunsa Khola. Adit-2 lies near Jongim village, about 1.6 km further south-west of Adit-1. Because of difficult site conditions, ERT surveys could not be performed in the planned locations for both adits, and, as such, were carried out at the nearest possible area from thier locations.

Adit-1 lies on a very steep slopes of bedrock gneiss. Very little vegetation is present on the hill slopes. The main rock type in this area consists of coarse foliated gneiss and migmatic gneiss with foliation dipping towards northeast. Bedrock is expected to occur at depths varying between 12 to 15 m in the surveyed area.

At Adit-2, the surficial deposit consists of residual soils with colluvial deposit and fractured rock fragments. Bedrock is expected to occur at depths between 10 to 20 m in the survey area.

1.4.1.3 Surge Shaft Areas

Two surge shaft sites, corresponding to Option-2 and Option-3 layout, are proposed for the project. The surge shaft for Option-3 mostly lies in residual soil deposits with huge angular boulders of gneiss and fractured rock fragments. Bedrock is found at a depth of around 8 to 10 m in this area.The surge shaft site for Option-2 lies in an area with bedrock exposure.

Based on the geological site conditions and presence of weathered and fractured rock mass between two competent bedrock masses identified in the ERT surveys, a narrow fault zone near the surge shaft area is expected.

1.4.1.4 Powerhouse Areas

The proposed powerhouse site lies on the right bank of Ghunsa Khola. Two powerhouse sites, viz. Option-1 and Option-2, were investigated during the present study.

Option-1 mostly lies in residual soil deposit with boulders and fractured rock fragments. Bedrock of gneiss is expected to occur at a depth of 15 to 20 m from the surface at this site.

Option-2 of the powerhouse lies in residual and colluvial soil deposits with steep slope. The area is covered by bushes and pine forest. Bedrock is expected to be found at depths varying between 10 and 15 m from the ground surface.

Based on the geological site conditions and presence of weathered and fractured rock mass between two competent bedrock masses identified in the ERT surveys, a narrow fault zone near the powerhouse area is expected.

1.4.1.5 Tailrace

Option-3 of the tailrace lies near the confluence of Tamor River and Ghunsa Khola while Option-2 lies near the Japantar area. Both tailrace areas are covered with colluvial deposits of residual soil with fractured rock fragments. Bedrock is found at varying depth of 15 to 25 m in the Option-3 tailrace area and depths of 5 to 18 m in the Option-2 area. Bedrock is found at shallow depths of less than 1 m in some parts of the tailrace area.

1.4.2 Conclusions

The geophysical subsurface studies at the project site reveal that the overburden thickness varies in different areas. Bedrocks are exposed at various sections in the project area. Most of the surveyed area at the headworks and the powerhouse are covered with alluvial/ colluvial deposits. The expected depth of bedrock at these varies.

Annex 4: Drill Core Logs

1. CORE DRILLING

1.1 GENERAL

A total of five drill holes, NDH-1 to NDH-5, were drilled at different locations in the project area (Table III-1-1). A description of the drilling activities, core logs and core photographs are presented in the following sections.

Table III-1-1: Details of drill holes

Drill hole	Location	Inclination	Total depth, m	Drilling machine
NDH-1	Weir axis	Vertical	40	HGY 200
NDH-2	Surge shaft	Vertical	40	HGY 200
NDH-3	Powerhouse	Vertical	105	HGY 200
NDH-4	Flat terrain upstream of powerhouse	Vertical	35	HGY 200
NDH-5	Along tailrace tunnel alignment	Vertical	30	HGY 200
Total length			**250**	

Direct Cone Penetration Tests (DCPT) were performed within alluvium deposits at different depths. In some cases, the cone could not be advanced due to the presence of large boulders. Permeability and water pressure tests were also conducted within the superficial deposits and the bed rocks, respectively.

1.2 CORE LOGS

The core logs of the five drill holes are presented at the end of the appendix. A description of the logs and associated issues are presented below.

Drill Hole NDH-1

Drill holeNDH-1 was drilled vertically on the right bank of Ghunsa Khola along the proposed weir axis. The depth of the hole was 40m.

Recent alluvial deposits, consisting mainly of pebbles, cobbles and boulder-sized gneisses and quartzite, were found up to a depth of 23.4 m. Bedrock was encountered at a depth of 23.4 m. From 23.4 to 40 m,bedrocks of different gneisses were recorded.

The penetration rate of the core drilling ranged from 6.5 to 27 mm/min. Core recovery ranged from 10% to 92% within the bedrocks,with RQD between 0 to 82%.

It is worth noting that the core recovered from NDH-1 is similar to that recovered from a 45 m vertical drill hole (DH-1) drilled on the left bank of Ghunsa Khola along the proposed weir axis during the 2012 feasibility study. In this drill hole, recent alluvial deposits consisting mainly of pebbles, cobbles and boulder-sized gneisses and quartzite, were found up to a depthof 20 m. From 20 to 25 m, colluviums with highly weathered rock fragments of gneisses were found. Bedrock was encountered at a depthof 25 m. From 25 m to 45 m, bedrocks of different gneisses were recorded.

For reference purposes, the core log of DH-1 has also been included after the core log of NDH-1.

Drill Hole NDH-2

Drill hole NDH-2 was drilled vertically in the surge tank area. The depth of the hole was 40 m.

Drill hole NDH-2 passed through colluvial deposits (overburden) of clay, silt, sand with rock fragments of gneisses up to 40 m. From 3 to 40mdepth, drilling water was totally lost. The colluvium was composed of fresh to slightly weathered, medium to coarse banded and augen gneisses with migmatities.

Very low core recoveries(9-19%) were achieved between 10 to 20 m and 35 to 40 m.

Drill Hole NDH-3

Drill hole NDH-3 was drilled vertically on the uphill slope of the right bank of Ghunsa Khola at Sekathum. The depth of the hole was 105 m.

Drill hole NDH-3 passed through colluvial deposits (overburden) upto a depth of 5 m. The colluvial deposits consist of boulder cobbles of gneisses and marbles embedded in fine sandy-

silty matrix. Bedrock was encountered at a depth of 5m. The core between depthof 5 to 105 m is mainly composed of marbles and calcareous gneisses with rare beds of quartzite.

The penetration rate of drilling ranged from 4 to 16 mm/min within the bedrocks up to a Depth: of 50 m. The RQD ranged from 8 to 94% and generally exceeds 40%. High drilling rate (33 to 34%) with very low core recovery (0-20%) and low RQD (0-5%) possibly indicate cavities filled with rock fragments.

Drill Hole NDH-4

Drill hole NDH-4 was drilled in a flat terrain upstream of the powerhouse site. The depth of the hole was 35 m.

Drill hole NDH-4 passes through colluvial deposits up to 19.5 m depth. Non-sticky, black carbonaceous material was encountered between 19.5 m and 20.5 m. Loose, dominantly finer clast material was encountered up to 29 m depth. This was followed by possible bed rock up to a depth of 35 m.

Metallic ore mineralization was encountered at depths of 30.8 m and 32.5 m. Mineralization was found in quartzites with alternating beds of gneisses and marbles.

Drill Hole NDH-5

Drill hole NDH-5 was drilled in the uphill slope on the right bank of Ghunsa Khola along the tailrace tunnel alignment. The depth of the hole was 30 m.

over sandy alluvium.

Drill holeNDH-5 passes through alluvium up to the depth of 13.5 m. The alluvium is composed of sand, silts, gravel and boulders of gneisses and quartzites. From 13.5 m to 30 m depth, bedrocks comprising different gneisses are developed.

The drilling rate within the bedrock ranged from 7 to 18 mm/min. The RQD of the core ranged from 21% to 91% and, in average, was above 46%.

Drill Hole NDH-1

	Depth: 0.0 – 5.0 m
	Depth: 5.0 – 10.0 m

	Depth: 10.0 – 15.0 m
	Depth: 15.0 – 20.0 m
	Depth: 20.0 – 25.0 m

	Depth: 25.0 – 30.0 m
	Depth:30.0 – 35.0 m
	Depth: 35.0 – 40.0 m

Photo III- 1-1: Core log for NDH-1

Drill Hole DH-1

(Drilled during 2012 feasibility study, included for reference purposes)

	Depth: 0.0 – 5.0 m
	Depth: 5.0 – 10.0 m
	Depth: 10.0 – 15.0 m

	Depth: 15.0 – 20.0 m
	Depth: 20.0 – 25.0 m
	Depth: 25.0 – 30.0 m

	Depth: 30.0 – 35.0 m
	Depth: 35.0 – 40.0 m
	Depth: 40.0 – 45.0 m

Photo III- 1-2: Core log for DH-1

Drill Hole NDH-2

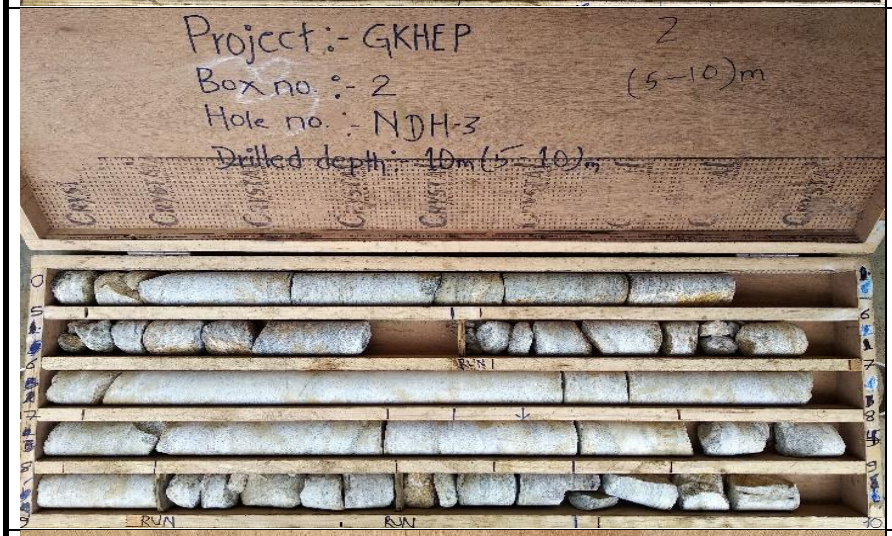
	Depth: 0.0 – 5.0 m
	Depth: 5.0 – 10.0 m
	Depth: 10.0 – 15.0 m

	Depth: 15.0 – 20.0 m
	Depth: 20.0 – 25.0 m
	Depth: 25.0 – 30.0 m

	Depth: 30.0 – 35.0 m
	Depth: 35.0 – 40.0 m

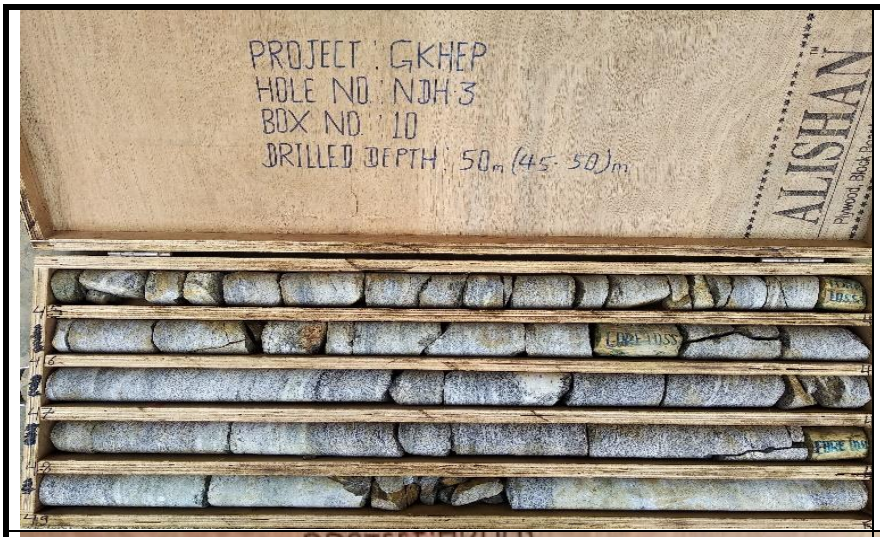
Photo III- 1-3: Core log for NDH-2

Drill Hole NDH-3

	Depth: 0.0 – 5.0 m
	**Depth: 5.0 – 10.0 m**
	Depth: 10.0 – 15.0 m

	Depth: 15.0– 20.0 m
	Depth: 20.0 – 25.0 m
	Depth: 25.0 – 30.0 m

	Depth: 30.0 – 35.0 m
	Depth: 35.0 – 40.0 m
	Depth: 40.0 – 45.0 m

	**Depth: 45.0 – 50.0 m**
	Depth: 50.0 – 55.0 m
	Depth: 55.0 – 60.0 m

	Depth: 60.0 – 65.0 m
	Depth: 65.0 – 70.0 m

	Depth: 70.0 – 75.0 m
	Depth: 75.0 – 80.0 m

	Depth: 80.0 – 85.0 m
	Depth:: 85.0 – 90.0 m
	Depth: 95.0 – 100.0 m

Depth:: 100.0 – 105.0 m

Photo III- 1-4: Core log for NDH-3

Drill Hole NDH-4

	Depth: 0.0 – 5.0 m
	Depth: 5.0 – 10.0 m
	Depth: 10.0 – 15.0 m

	Depth: 15.0– 20.0 m
	Depth: 20.0 – 25.0 m
	Depth: 25.0 – 30.0 m

Depth: 30.0 – 35.0 m

Photo III- 1-5: Core log for NDH-4

Drill Hole NDH-5

	Depth: 0.0 – 5.0 m
	Depth: 5.0 – 10.0 m
	Depth: 10.0 – 15.0 m

	Depth: 15.0– 20.0 m
	Depth: 20.0 – 25.0 m
	Depth: 25.0 – 30.0 m

Photo III- 1-6: Core log for NDH-5

Annex 5: Water Pressure and Permeability Tests

1. WATER PRESSURE & PERMEABILITY TESTS

1.1 WATER PRESSURE TESTS

Drill Hole NDH-1

Water table: 0.65m

Test depth: 25.0-28.0 m

P_0	H_1	H_2	P	Q	L	Lu
(kg/cm^2)	(m)	(m)	(kg/cm^2)	(l/min/m)	m	
1	1	0.65	1.165	27	3	77.253
3	1	0.65	3.165	25	3	26.330
5	1	0.65	5.165	21	3	13.553
3	1	0.65	3.165	24	3	25.276
1	1	0.65	1.165	26	3	74.392
					Representative Lugeon Value	**13.553**

Test depth: 28.0-31.0 m

P_0	H_1	H_2	P	Q	L	Lu
(kg/cm^2)	(m)	(m)	(kg/cm^2)	(l/min/m)	m	
1	1	0.65	1.165	26	3	74.392
3	1	0.65	3.165	25	3	26.330
5	1	0.65	5.165	23	3	14.843
3	1	0.65	3.165	26	3	27.383
1	1	0.65	1.165	27	3	77.253
					Representative Lugeon Value	**14.843**

Test depth: 31.0 –34.0 m

P_0	H_1	H_2	P	Q	L	Lu
(kg/cm^2)	**(m)**	**(m)**	**(kg/cm^2)**	**(l/min/m)**	**m**	
1	1	0.65	1.165	29	3	82.976
3	1	0.65	3.165	27	3	28.436
5	1	0.65	5.165	24	3	15.489
3	1	0.65	3.165	25	3	26.330
1	1	0.65	1.165	27	3	77.253
					Representative Lugeon Value	**15.489**

Test depth: 35.0 – 38.0 m

P_0	H_1	H_2	P	Q	L	Lu
(kg/cm^2)	(m)	(m)	(kg/cm^2)	(l/min/m)	m	
1	1	0.65	1.165	31	3	88.698
3	1	0.65	3.165	30	3	31.596
5	1	0.65	5.165	24	3	15.489
3	1	0.65	3.165	28	3	29.489
1	1	0.65	1.165	30	3	85.837
					Representative Lugeon Value	**15.489**

Drill Hole NDH-3

Test depth: 6.0 – 9.0m

P_0 (kg/cm^2)	H_1 (m)	H_2 (m)	P (kg/cm^2)	Q (l/min/m)	L (m)	Lu
1	1	7.5	1.85	43	3	77.477
2	1	7.5	2.85	42	3	49.123
3	1	7.5	3.85	40	3	34.632
2	1	7.5	2.85	32	3	37.427
1	1	7.5	1.85	42	3	75.676
					Representative Lugeon Value	**34.632**

Test depth: 9.0 – 12.0 m

P_0 (kg/cm^2)	H_1 (m)	H_2 (m)	P (kg/cm^2)	Q (l/min/m)	L (m)	Lu
1	1	10.5	2.15	23	3	35.659
2	1	10.5	3.15	20	3	21.164
3	1	10.5	4.15	10	3	8.032
2	1	10.5	3.15	18	3	19.048
1	1	10.5	2.15	24	3	37.209
					Representative Lugeon Value	**8.032**

Test depth: 13.0 – 16.0 m

P_0 (kg/cm^2)	H_1 (m)	H_2 (m)	P (kg/cm^2)	Q (l/min/m)	L (m)	Lu
1	1	14.5	2.55	47	3	61.438
1.5	1	14.5	3.05	49	3	53.552
2	1	14.5	3.55	47	3	44.131
1.5	1	14.5	3.05	50	3	54.645
1	1	14.5	2.55	51	3	66.667
					Representative Lugeon Value	**44.131**

Test depth: 16.0 – 19.0 m

P_0 (kg/cm^2)	H_1 (m)	H_2 (m)	P (kg/cm^2)	Q (l/min/m)	L (m)	Lu
1	1	17.5	2.85	35	3	40.935
2	1	17.5	3.85	39	3	33.766
3	1	17.5	4.85	40	3	27.491
2	1	17.5	3.85	42	3	36.363
1	1	17.5	2.85	33	3	38.596
					Representative Lugeon Value	**27.491**

Test depth: 19.0 – 22.0m

P_0 (kg/cm^2)	H_1 (m)	H_2 (m)	P (kg/cm^2)	Q (l/min/m)	L (m)	Lu
1	1	20.5	3.15	38	3	40.211
2	1	20.5	4.15	37	3	29.718
3	1	20.5	5.15	35	3	22.653
2	1	20.5	4.15	39	3	31.325
1	1	20.5	3.15	40	3	42.328
					Representative Lugeon Value	**22.653**

Test depth: 22.0 – 25.0 m

P_0	H_1	H_2	P	Q	L	Lu
(kg/cm^2)	(m)	(m)	(kg/cm^2)	(l/min/m)	(m)	
1	1	23.5	3.45	38	3	36.714
2	1	23.5	4.45	39	3	29.213
3	1	23.5	5.45	36	3	22.018
2	1	23.5	4.45	39	3	29.213
1	1	23.5	3.45	33	3	31.884
					Representative Lugeon Value	**22.018**

Tested depth: 25.0 - 28.0 m

P_0	H_1	H_2	P	Q	L	Lu
(kg/cm^2)	(m)	(m)	(kg/cm^2)	(l/min/m)	(m)	
1	1	26.5	3.75	44	3	39.111
3	1	26.5	5.75	40	3	23.188
5	1	26.5	7.75	32	3	13.763
3	1	26.5	5.75	40	3	23.188
1	1	26.5	3.75	45	3	40.000
					Representative Lugeon Value	**13.763**

Test depth: 28.0– 31.0 m

P_0	H_1	H_2	P	Q	L	Lu
(kg/cm^2)	**(m)**	**(m)**	**(kg/cm^2)**	**(l/min/m)**	**(m)**	
1	1	29.5	4.05	49	3	40.329
3	1	29.5	6.05	45	3	24.793
5	1	29.5	8.05	40	3	16.563
3	1	29.5	6.05	48	3	26.446
1	1	29.5	4.05	48	3	39.506
					Representative Lugeon Value	**16.563**

Test depth: 31.0 – 34.0 m

P_0	H_1	H_2	P	Q	L	Lu
(kg/ cm^2)	(m)	(m)	(kg/ cm^2)	(l/min/m)	(m)	
1	1	32.5	4.35	45	3	34.482
3	1	32.5	6.35	40	3	20.997
5	1	32.5	8.35	36	3	14.371
3	1	32.5	6.35	41	3	21.522
1	1	32.5	4.35	47	3	36.015
					Representative Lugeon Value	**14.371**

Test depth: 34.0 – 37.0 m

P_0	H_1	H_2	P	Q	L	Lu
(kg/ cm^2)	(m)	(m)	(kg/ cm^2)	(l/min/m)	(m)	
1	1	35.5	4.65	48	3	43.835
3	1	35.5	6.65	45	3	22.556
5	1	35.5	8.65	39	3	15.028
3	1	35.5	6.65	47	3	23.558
1	1	35.5	4.65	46	3	42.009
					Representative Lugeon Value	**15.028**

Test depth: 37.0 – 40.0 m

P_0	H_1	H_2	P	Q	L	Lu
(kg/cm^2)	(m)	(m)	(kg/cm^2)	(l/min/m)	(m)	
1	1	38.5	4.95	47	3	31.649
3	1	38.5	6.95	45	3	21.582
5	1	38.5	8.95	40	3	14.897
3	1	38.5	6.95	43	3	20.623
1	1	38.5	4.95	48	3	25.589
					Representative Lugeon Value	**14.897**

Test depth: 40.5 – 43.5 m

P_0	H_1	H_2	P	Q	L	Lu
(kg/cm^2)	(m)	(m)	(kg/cm^2)	(l/min/m)	(m)	
1	1	42.0	5.30	49	3	30.817
3	1	42.0	7.30	46	3	21.004
5	1	42.0	9.30	42	3	15.053
3	1	42.0	7.30	45	3	20.547
1	1	42.0	5.30	47	3	29.559
					Representative Lugeon Value	**15.053**

Test depth: 44.0 – 47.0 m

P_0	H_1	H_2	P	Q	L	Lu
(kg/cm^2)	(m)	(m)	(kg/cm^2)	(l/min/m)	(m)	
1	1	45.5	5.65	52	3	30.678
3	1	45.5	7.65	48	3	20.915
5	1	45.5	9.65	43	3	14.853
3	1	45.5	7.65	49	3	21.350
1	1	45.5	5.65	50	3	29.498
					Representative Lugeon Value	**14.853**

Test depth: 47.0 – 50.0 m

P_0	H_1	H_2	P	Q	L	Lu
(kg/cm^2)	(m)	(m)	(kg/cm^2)	(l/min/m)	(m)	
1	1	48.5	5.95	53	3	29.691
3	1	48.5	7.95	50	3	20.964
5	1	48.5	9.95	47	3	15.745
3	1	48.5	7.95	49	3	20.545
1	1	48.5	5.95	52	3	29.131
					Representative Lugeon Value	**15.745**

Test depth: 51.0 – 54.0 m

P_0	H_1	H_2	P	Q	L	Lu
(kg/cm^2)	(m)	(m)	(kg/cm^2)	(l/min/m)	(m)	
1	1	52.5	6.35	57	3	29.921
3	1	52.5	8.35	52	3	20.758
5	1	52.5	10.35	48	3	15.459
3	1	52.5	8.35	54	3	21.557
1	1	52.5	6.35	59	3	30.971
					Representative Lugeon Value	**15.459**

Test depth: 54.0 – 57.0 m

P_0	H_1	H_2	P	Q	L	Lu
(kg/cm^2)	(m)	(m)	(kg/cm^2)	(l/min/m)	(m)	
1	1	55.5	6.65	54	3	27.068
3	1	55.5	8.65	50	3	19.268
5	1	55.5	10.65	46.5	3	14.554
3	1	55.5	8.65	51	3	19.653
1	1	55.5	6.65	56	3	28.070
					Representative Lugeon Value	**14.554**

Test depth: 57.0 – 60.0 m

P_0	H_1	H_2	P	Q	L	Lu
(kg/cm^2)	(m)	(m)	(kg/cm^2)	(l/min/m)	(m)	
1	1	58.5	6.95	58	3	27.818
3	1	58.5	8.95	53.5	3	19.926
5	1	58.5	10.95	49	3	14.916
3	1	58.5	8.95	54	3	20.112
1	1	58.5	6.95	58	3	27.818
					Representative Lugeon Value	**14.916**

Test depth: 60.0 – 63.0 m

P_0	H_1	H_2	P	Q	L	Lu
(kg/cm^2)	(m)	(m)	(kg/cm^2)	(l/min/m)	(m)	
1	1	61.5	7.25	54	3	24.828
3	1	61.5	9.25	51	3	18.378
5	1	61.5	11.25	46	3	13.630
3	1	61.5	9.25	50	3	18.018
1	1	61.5	7.25	53	3	24.368
					Representative Lugeon Value	**13.630**

Test depth: 63.0 – 66.0 m

P_0	H_1	H_2	P	Q	L	Lu
(kg/cm^2)	(m)	(m)	(kg/cm^2)	(l/min/m)	(m)	
1	1	64.5	7.55	56	3	24.724
3	1	64.5	9.55	51	3	17.801
5	1	64.5	11.55	45	3	12.987
3	1	64.5	9.55	51	3	17.801
1	1	64.5	7.55	57	3	25.166
					Representative Lugeon Value	**12.987**

Test depth: 67.0 – 70.0 m

P_0	H_1	H_2	P	Q	L	Lu
(kg/cm^2)	(m)	(m)	(kg/cm^2)	(l/min/m)	(m)	
1	1	68.5	7.95	58	3	24.319
3	1	68.5	9.95	53	3	17.755
5	1	68.5	11.95	46.5	3	12.971
3	1	68.5	9.95	52	3	17.420
1	1	68.5	7.95	59	3	24.738
					Representative Lugeon Value	**12.971**

Drill Hole NDH-5

Test depth: 18.0 – 21.0 m

P_0 (kg/cm^2)	H_1 (m)	H_2 (m)	P (kg/cm^2)	Q (l/min/m)	L (m)	Lu
1	0	19.5	2.95	15	3	16.949
2	0	19.5	3.95	13	3	10.970
3	0	19.5	4.95	8	3	5.387
2	0	19.5	3.95	12	3	10.127
1	0	19.5	2.95	16	3	18.079
					Representative Lugeon Value	**5.387**

Test depth: 21.0 – 24.0 m

P_0 (kg/cm^2)	H_1 (m)	H_2 (m)	P (kg/cm^2)	Q (l/min/m)	L (m)	Lu
1	0	22.5	3.25	12	3	12.308
2	0	22.5	4.25	10	3	7.843
3	0	22.5	5.25	8.5	3	5.397
2	0	22.5	4.25	10	3	7.843
1	0	22.5	3.25	13	3	13.333
					Representative Lugeon Value	**5.397**

Test depth: 24.0 – 27.0 m

P_0 (kg/cm^2)	H_1 (m)	H_2 (m)	P (kg/cm^2)	Q (l/min/m)	L (m)	Lu
1	0	25.5	3.55	11	3	10.329
2	0	25.5	4.55	10	3	7.326
3	0	25.5	5.55	8	3	4.805
2	0	25.5	4.55	10.5	3	7.692
1	0	25.5	3.55	16	3	15.023
					Representative Lugeon Value	**4.805**

Test depth: 27.0 – 30.0 m

P_0 (kg/cm^2)	H_1 (m)	H_2 (m)	P (kg/cm^2)	Q (l/min/m)	L (m)	Lu
1	0	28.5	3.85	11	3	9.524
2	0	28.5	4.85	9	3	6.186
3	0	28.5	5.85	8	3	4.558
2	0	28.5	4.85	10	3	6.873
1	0	28.5	3.85	14	3	12.121
					Representative Lugeon Value	**4.558**

Drill Hole DH-C

Water table: 6m

Test depth: 4.0 – 7.0 m

P_0 (kg/cm^2)	H_1 (m)	H_2 (m)	P (kg/cm^2)	Q (l/min/m)	L (m)	Lu
1	0	5.5	1.55	20	3	43.020
2	0	5.5	2.55	14	3	18.310
3	0	5.5	3.55	4	3	3.760
2	0	5.5	2.55	15	3	19.610
1	0	5.5	1.55	21	3	45.170
					Representative Lugeon Value	**3.760**

Test depth: 7.0 – 10.0 m

P_0	H_1	H_2	P	Q	L	Lu
(kg/cm^2)	(m)	(m)	(kg/cm^2)	(l/min/m)	(m)	
1	0	5.5	1.55	20	3	43.020
2	0	5.5	2.55	14	3	18.310
3	0	5.5	3.55	4	3	3.760
2	0	5.5	2.55	15	3	19.610
1	0	5.5	1.55	21	3	45.170
					Representative Lugeon Value	**3.760**

Test depth: 10.0 – 13.0 m

P_0	H_1	H_2	P	Q	L	Lu
(kg/cm^2)	(m)	(m)	(kg/cm^2)	(l/min/m)	(m)	
1	0	6.00	1.60	31	3	64.583
2	0	6.00	2.60	14	3	17.950
3	0	6.00	3.60	8	3	7.410
2	0	6.00	2.60	15	3	19.230
1	0	6.00	1.60	23	3	47.920
					Representative Lugeon Value	**7.410**

Test depth: 14.0 – 17.0 m

P_0	H_1	H_2	P	Q	L	Lu
(kg/cm^2)	(m)	(m)	(kg/cm^2)	(l/min/m)	(m)	
1	0	6.00	1.60	26	3	54.160
2	0	6.00	2.60	18	3	23.080
3	0	6.00	3.60	11	3	10.180
2	0	6.00	2.60	17	3	21.790
1	0	6.00	1.60	34	3	70.833
					Representative Lugeon Value	**10.180**

Test depth: 19.0 – 22.0 m

P_0	H_1	H_2	P	Q	L	Lu
(kg/cm^2)	(m)	(m)	(kg/cm^2)	(l/min/m)	(m)	
1	0	6.00	1.60	26	3	54.160
2	0	6.00	2.60	21	3	26.920
3	0	6.00	3.60	13	3	12.030
2	0	6.00	2.60	20	3	25.640
1	0	6.00	1.60	24	3	50.000
					Representative Lugeon Value	**12.030**

Test depth: 22.0 – 25.0 m

P_0	H_1	H_2	P	Q	L	Lu
(kg/cm^2)	(m)	(m)	(kg/cm^2)	(l/min/m)	(m)	
1	0	6.00	1.60	21	3	43.750
2	0	6.00	2.60	21	3	26.920
3	0	6.00	3.60	22	3	20.370
2	0	6.00	2.60	27	3	34.620
1	0	6.00	1.60	21	3	43.750
					Representative Lugeon Value	**20.370**

Test depth: 25.0 – 28.0 m

P_0	H_1	H_2	P	Q	L	Lu
(kg/cm^2)	(m)	(m)	(kg/cm^2)	(l/min/m)	(m)	
1	0	6.00	1.60	28	3	58.330
2	0	6.00	2.60	20	3	25.640
3	0	6.00	3.60	14	3	12.960
2	0	6.00	2.60	21	3	26.920
1	0	6.00	1.60	29	3	60.410
					Representative Lugeon Value	**12.960**

1.2 PERMEABLITY TESTS

Location: NDH-1

Depth of water table: 0.65 m

SN	Test section (m)	Tube diameter, d_p (cm)	Hole diameter, 2r (cm)	Test length, L (cm)	Test time, t (s)	Initial head, h_0 (cm)	Head after time t, h_1 (cm)	Intake factor, F	Permeability, k (cm/sec)	Av. permeability, k_{av} (cm/sec)
1	2.0 – 3.0	10.01	7.31	100	60	65	53	189.8009727	0.001410	
		10.01	7.31	100	60	65	53	189.8009727	0.001410	
		10.01	7.31	100	60	65	53	189.8009727	0.001410	1.41E-03
		10.01	7.31	100	60	65	53	189.8009727	0.001410	
		10.01	7.31	100	60	65	53	189.8009727	0.001410	
2	5.5 - 6.0	10.01	7.31	50	60	65	60	119.8519329	0.000876	
		10.01	7.31	50	60	65	60	119.8519329	0.000876	8.76E-04
		10.01	7.31	50	60	65	60	119.8519329	0.000876	
3	8.0 – 9.0	10.01	7.31	100	60	65	54	189.8009727	0.001281	
		10.01	7.31	100	60	65	54	189.8009727	0.001281	
		10.01	7.31	100	60	65	57	189.8009727	0.000908	1.06E-03
		10.01	7.31	100	60	65	57	189.8009727	0.000908	
		10.01	7.31	100	60	65	57	189.8009727	0.000908	
4	11.0 – 12.0	7.64	7.31	100	120	65	59.5	189.8009727	0.000178	
		7.64	7.31	100	60	65	56	189.8009727	0.000600	
		7.64	7.31	100	60	65	55	189.8009727	0.000672	5.59E-04
		7.64	7.31	100	60	65	55	189.8009727	0.000672	
		7.64	7.31	100	60	65	55	189.8009727	0.000672	
5	15.0 -16.0	7.64	7.31	100	60	65	48	189.8009727	0.001220	
		7.64	7.31	100	60	65	52	189.8009727	0.000898	9.79E-04
		7.64	7.31	100	60	65	52	189.8009727	0.000898	
		7.64	7.31	100	60	65	52	189.8009727	0.000898	
6	19.0 – 20.0	7.64	7.31	100	60	65	51	189.8009727	0.000976	
		7.64	7.31	100	60	65	55	189.8009727	0.000672	
		7.64	7.31	100	60	65	53	189.8009727	0.000822	8.23E-04
		7.64	7.31	100	60	65	53	189.8009727	0.000822	
		7.64	7.31	100	60	65	53	189.8009727	0.000822	

Location: NDH-4

Depth of water table: *not encountered*

SN	Test section (m)	Tube diameter, d_p (cm)	Hole diameter, 2r (cm)	Test length, L (cm)	Test time, t (s)	Initial head, h_0 (cm)	Head after time t, h_1 (cm)	Intake factor, F	Permeability, k (cm/sec)	Av. permeability, k_{av} (cm/sec)
1	1.0 – 2.0	10.01	7.31	100	60	120	118	189.8009727	0.000116	1.09E-04
		10.01	7.31	100	60	120	118.5	189.8009727	0.000087	
		10.01	7.31	100	60	120	118	189.8009727	0.000116	
		10.01	7.31	100	60	120	118	189.8009727	0.000116	
2	4.0 – 5.0	10.01	7.31	100	60	520	516	189.8009727	0.000053	4.13E-05
		10.01	7.31	100	60	520	517.5	189.8009727	0.000033	
		10.01	7.31	100	60	520	517	189.8009727	0.000040	
		10.01	7.31	100	60	520	517	189.8009727	0.000040	
		10.01	7.31	100	60	520	517	189.8009727	0.000040	
3	7.0 - 8.0	10.01	7.31	100	60	910	907	189.8009727	0.000023	2.21E-05
		10.01	7.31	100	60	910	907.5	189.8009727	0.000019	
		10.01	7.31	100	60	910	907	189.8009727	0.000023	
		10.01	7.31	100	60	910	907	189.8009727	0.000023	
		10.01	7.31	100	60	910	907	189.8009727	0.000023	

Location: DH-A

Depth of water table: *not encountered*

SN	Test section (m)	Tube diameter, d_p (cm)	Hole diameter, 2r (cm)	Test length, L (cm)	Test time, t (s)	Initial head, h_0 (cm)	Head after time t, h_1 (cm)	Intake factor, F	Permeability, k (cm/sec)	Av. permeability, k_{av} (cm/sec)
1	10.0 – 12.0	7.64	3.655	200	60	1000	990	313.9587178	0.000024	2.15E-05
		7.64	3.655	200	60	1000	992	313.9587178	0.000020	
		7.64	3.655	200	60	1000	998	313.9587178	0.000005	
		7.64	3.655	200	60	1000	988	313.9587178	0.000029	
		7.64	3.655	200	60	1000	988	313.9587178	0.000029	

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Annex 6: Laboratory Tests on Rock Core Samples

1. LABORATORY TESTS ON CORE SAMPLES

1.1 POINT LOAD TESTS

Table 1-1: Details of point load tests on core specimen

Drill hole	Depth m	Sample Name	Test	Width mm	Ht mm	P, kN	De^2	De	Is MPa	F	Is (50) MPa
NDH-1	26-28	Banded Gneiss	Axial	54.57	81.83	17	2977.88	54.57	5.71	1.040	5.96
	38-40	Banded Gneiss	Axial	54.68	109.36	25	2989.90	54.68	8.36	1.041	8.74
NDH-3	8-9.15	White Marble	Axial	54.38	81.30	18	2,957.18	54.38	6.09	1.039	6.35
	10.3-11.5	Calcareous Gneiss	Axial	53.65	108.70	17.5	2,878.32	53.65	6.08	1.032	6.30
	34-36	Calcareous Gneiss	Axial	54.27	108.54	35	2,945.23	54.27	11.88	1.038	12.38
DH-A	18-19	Gneiss	Axial	55.00	69.00	15	4831.94	69.51	3.10	1.160	3.66
			Diametral	54.00	80.60	16	6496.36	80.60	2.46	1.240	3.13
DH-B	24-27	Gneiss	Axial	54.00	71.00	23	4881.60	69.87	4.71	1.162	5.57
			Diametral	54.00	100.00	14	10000.0	100.00	1.40	1.366	1.98
	27-30	Gneiss	Axial	52.00	68.00	15	4502.18	67.10	3.33	1.142	3.86
			Diametral	52.00	79.00	24	6241.00	79.00	3.85	1.229	4.83
DH-C	7-8	Banded Gneiss	Axial	56.00	75.30	21	5369.00	73.27	3.91	1.188	4.73
			Diametral	56.00	85.80	24	7361.64	85.80	3.26	1.275	4.27
	22-25	Banded Gneiss	Axial	53.00	69.00	13	4656.24	68.24	2.79	1.150	3.26
			Diametral	53.00	85.00	23	7225.00	85.00	3.18	1.270	4.15

1.2 UNIAXIAL COMPRESSIVE STRENGTH TESTS

Table 1-2: Details of uniaxial compressive strength tests on core specimen

Drill Hole No.	Depth m	Sample			Uniaxial Compressive Strength kg/cm^2
		Diameter mm	Height mm	Weight gm	
NDH-1	12.00-14.00	54.61	111.50	720.90	361.38
	26.88-28.00	54.58	113.50	750.00	261.85
	38.00-40.00	54.64	112.17	741.40	322.99
NDH-3	8.00-9.15	54.53	114.34	737.00	238.45
	10.30-11.50	54.27	112.75	750.50	510.37
	22.00-23.75	54.58	112.09	730.70	380.82
	28.00-29.50	54.64	111.75	744.60	341.99
	30.80-32.50	40.85	89.20	382.90	577.86
	34.00-36.00	54.34	110.61	804.60	326.56
DH-B	24.00-27.00	54.00	120.00	754.90	486.55
	27.00-30.00	54.00	120.00	752.90	836.87
DH-C	7.00-8.00	54.00	118.50	752.20	330.86
	22.00-25.00	54.00	118.00	745.30	992.57

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Uniaxial Compression Test

Client:

Sample Name : Augen Gneiss Date:2076.07.29

S.No.	Drill Hole No.	Depth m	Sample		Weight gm	Uniaxial Compressive Strength, kg/cm^2
			diameter mm	height mm		
1	NDH-1	12.00- 14.00 m	54.61	111.50	720.90	361.38

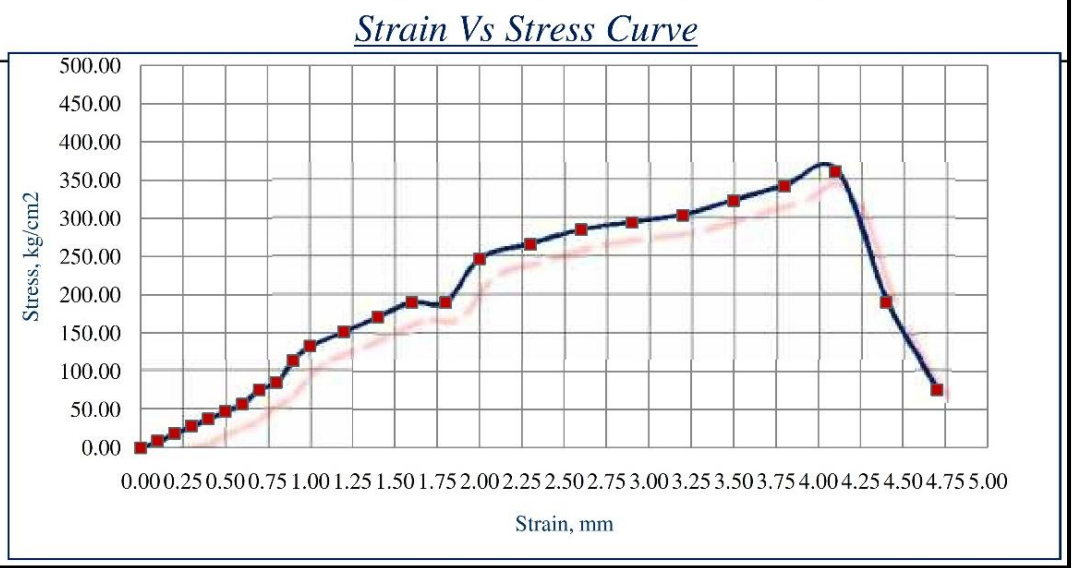


Figure 1-1: Stress-Strain Curve for core samples from drill hole NDH-1

Soil Rock & Concrete Laboratory NEA.	Uniaxial Compression Test

Client:Sakhuwa Engineering Consultancy Pvt.Ltd.

Sample Name : Bandded gneiss Date: 2076.07.29

S.No.	Drill Hole No.	Depth m	Sample diameter mm	Sample height mm	Weight gm	Uniaxial Compressive Strength, kg/cm^2
1	NDH-1	26.88 -28.00 m	54.58	113.50	750.00	261.85

Strain Vs Stress Curve

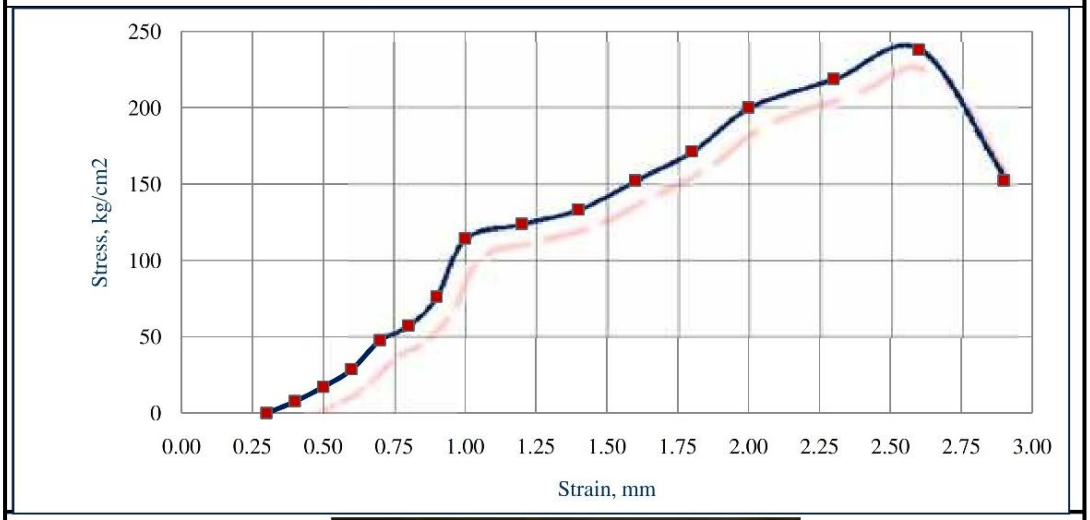


Figure 1-2: Stress-Strain Curve for core samples from drill hole NDH-1

Soil Rock & Concrete Laboratory NEA.

Uniaxial Compression Test

Client:Sakhuwa Engineering Consultancy Pvt.Ltd.

Sample Name :Banded Gneiss Date: 2076.07.29

S.No.	Drill Hole No.	Depth m	Sample diameter mm	Sample height mm	Weight gm	Uniaxial Compressive Strength,
1	NDH-1	38.00- 40.00 m	54.64	112.17	741.40	322.99

Strain Vs Stress Curve

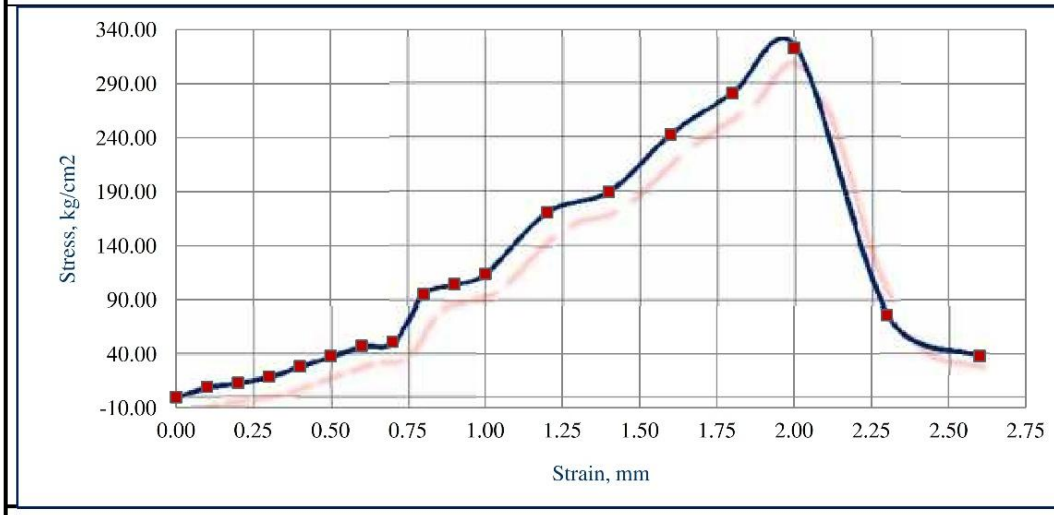


Figure 1-3: Stress-Strain Curve for core samples from drill hole NDH-1

Soil Rock & Concrete Laboratory NEA.

Uniaxial Compression Test

Client:Sakhuwa Engineering Consultancy Pvt.Ltd.

Sample Name :White Marble

Date: 2076.07.29

S.No.	Drill Hole No.	Depth m	Sample diameter mm	Sample height mm	Weight gm	Uniaxial Compressive Strength, kg/cm^2
1	NDH-3	8.00- 9.15.00 m	54.53	114.34	737.00	238.45

Strain Vs Stress Curve

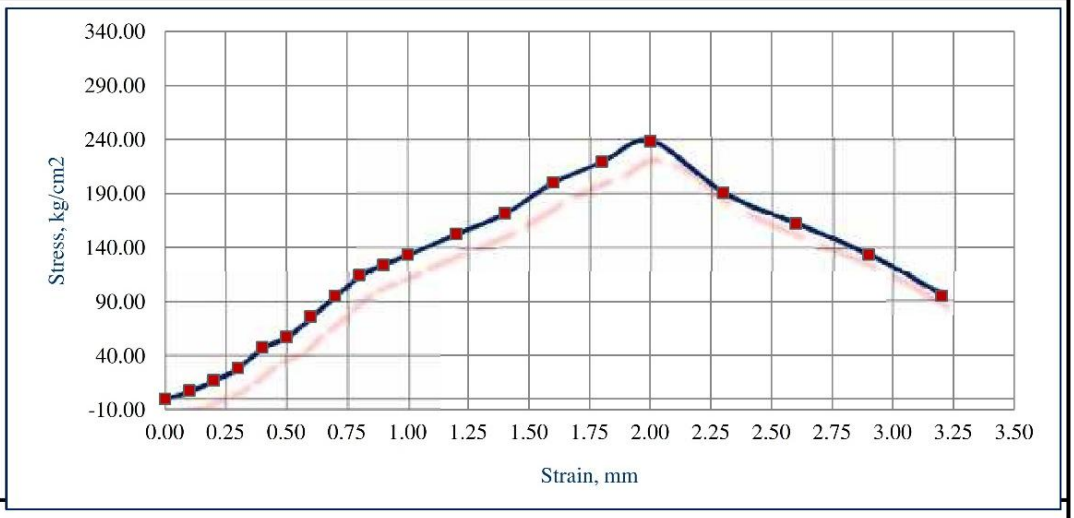


Figure 1-4: Stress-Strain Curve for core samples from drill hole NDH-3

Soil Rock & Concrete Laboratory NEA.

Uniaxial Compression Test

Client:Sakhuwa Engineering Consultancy Pvt.Ltd.

Sample Name :Calcarious Gneiss

Date: 2076.07.29

S.No.	Drill Hole No.	Depth m	Sample diameter mm	Sample height mm	Weight gm	Uniaxial Compressive Strength, kg/cm^2
1	NDH-3	10.30 m - 11.50 m	54.27	112.75	750.50	510.37

Strain Vs Stress Curve

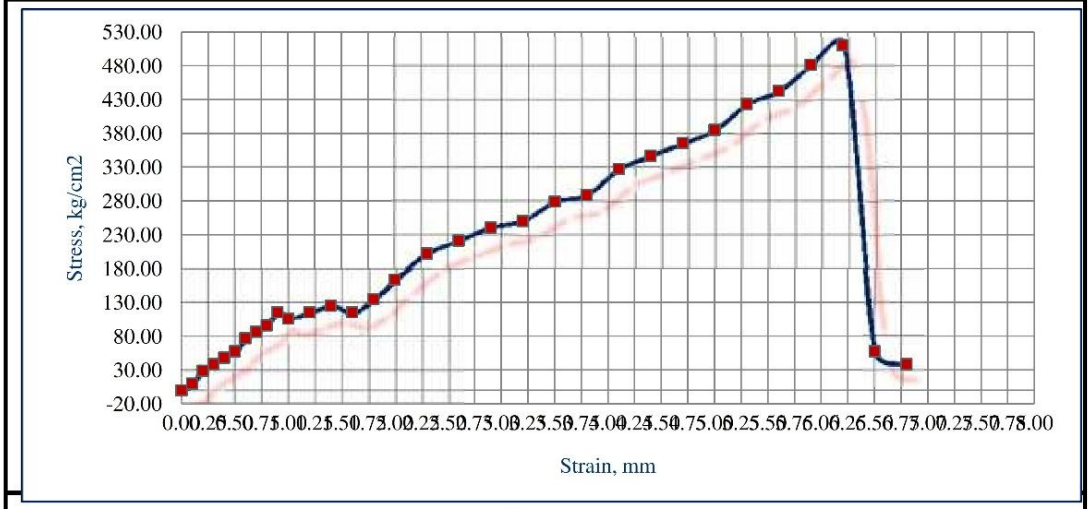


Figure 1-5: Stress-Strain Curve for core samples from drill hole NDH-3

Soil Rock & Concrete Laboratory NEA.

Uniaxial Compression Test

Client:Sakhuwa Engineering Consultancy Pvt.Ltd.

Sample Name : White Marble

Date: 2076.07.29

S.No.	Drill Hole No.	Depth m	Sample diameter mm	Sample height mm	Weight gm	Uniaxial Compressive Strength, kg/cm^2
1	NDH-3	22.00 m - 23.75 m	54.58	112.09	730.70	380.82

Strain Vs Stress Curve

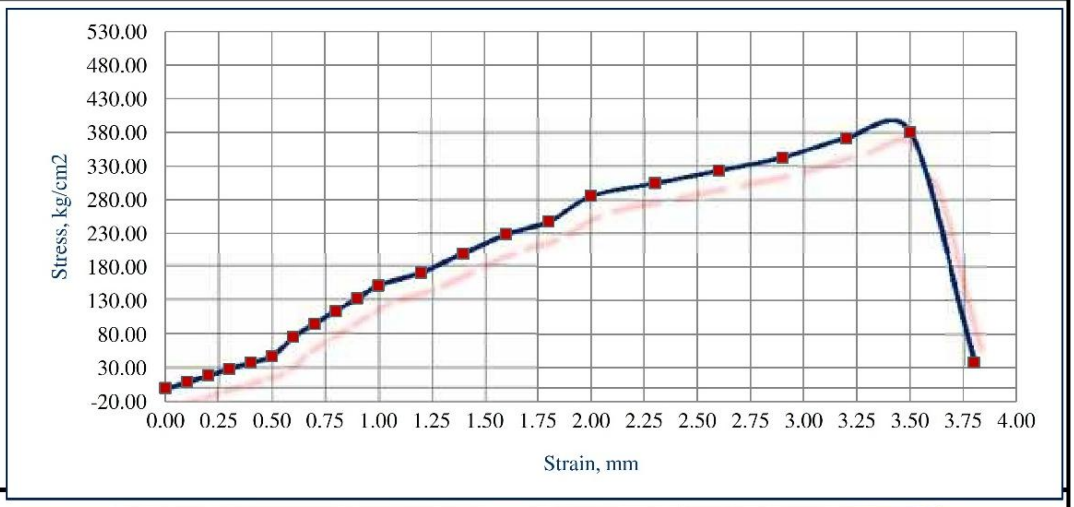


Figure 1-6: Stress-Strain Curve for core samples from drill hole NDH-3

Soil Rock & Concrete Laboratory NEA.

Uniaxial Compression Test

Client:Sakhuwa Engineering Consultancy Pvt.Ltd.

Sample Name : Quartzite

Date: 2076.07.29

S.No.	Drill Hole No.	Depth m	Sample diameter mm	Sample height mm	Weight gm	Uniaxial Compressive Strength, kg/cm^2
1	NDH-3	28.00 m - 29.50 m	54.64	111.75	744.60	341.99

Strain Vs Stress Curve

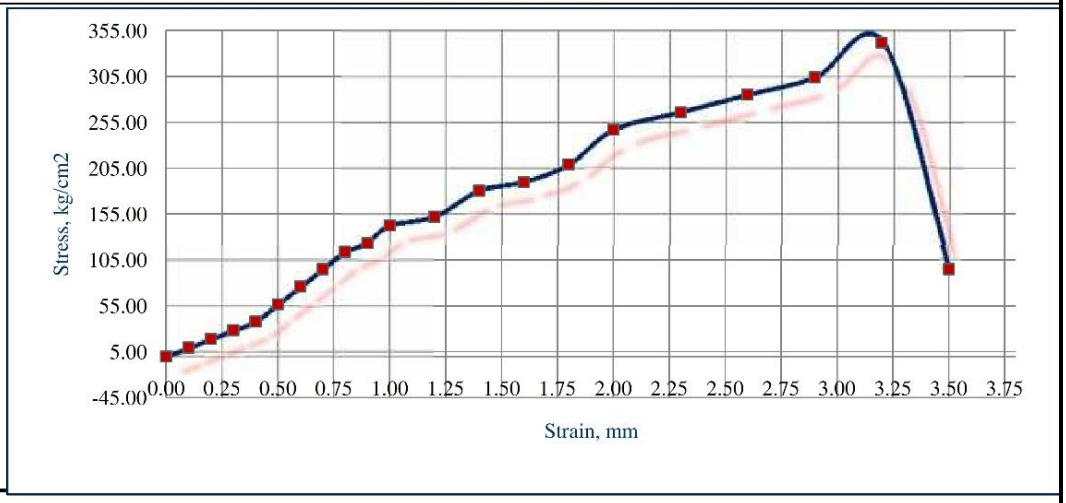


Figure 1-7: Stress-Strain Curve for core samples from drill hole NDH-3

Soil Rock & Concrete Laboratory NEA.

Uniaxial Compression Test

Client:Sakhuwa Engineering Consultancy Pvt.Ltd.

Sample Name : Metalic Core

Date: 2076.07.29

S.No.	Drill Hole No.	Depth m	Sample diameter mm	Sample height mm	Weight gm	Uniaxial Compressive Strength, kg/cm^2
1	NDH-3	30.80 - 32.50 m	40.85	89.20	382.90	577.86

Strain Vs Stress Curve

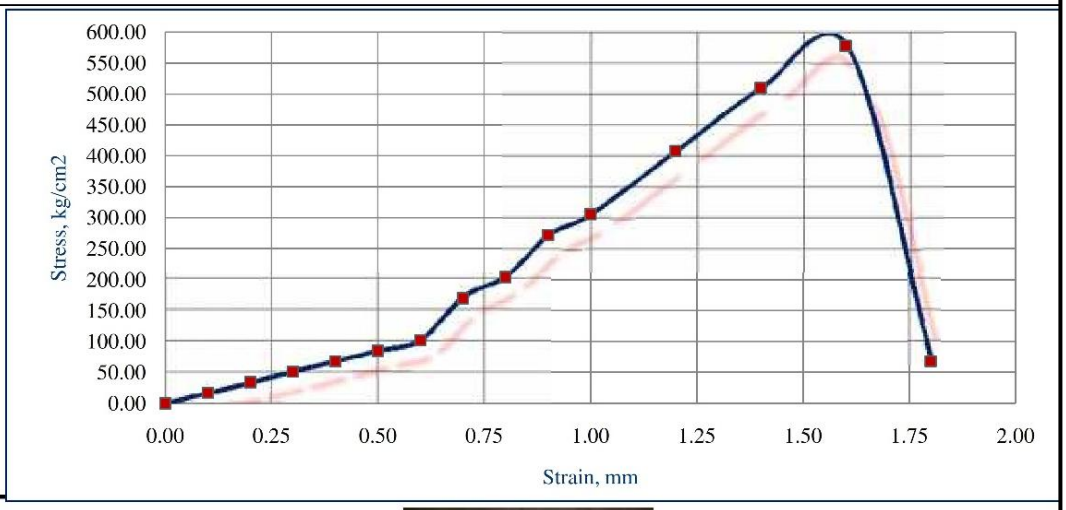


Figure 1-8: Stress-Strain Curve for core samples from drill hole NDH-3

Soil Rock & Concrete Laboratory NEA.

Uniaxial Compression Test

Client:Sakhuwa Engineering Consultancy Pvt.Ltd.

Sample Name : Calcarious Gneiss

Date: 2076.07.29

S.No.	Drill Hole No.	Depth m	Sample diameter mm	Sample height mm	Weight gm	Uniaxial Compressive Strength, kg/cm^2
1	NDH-3	34.00 m -36.00 m	54.34	110.61	804.60	326.56

Strain Vs Stress Curve

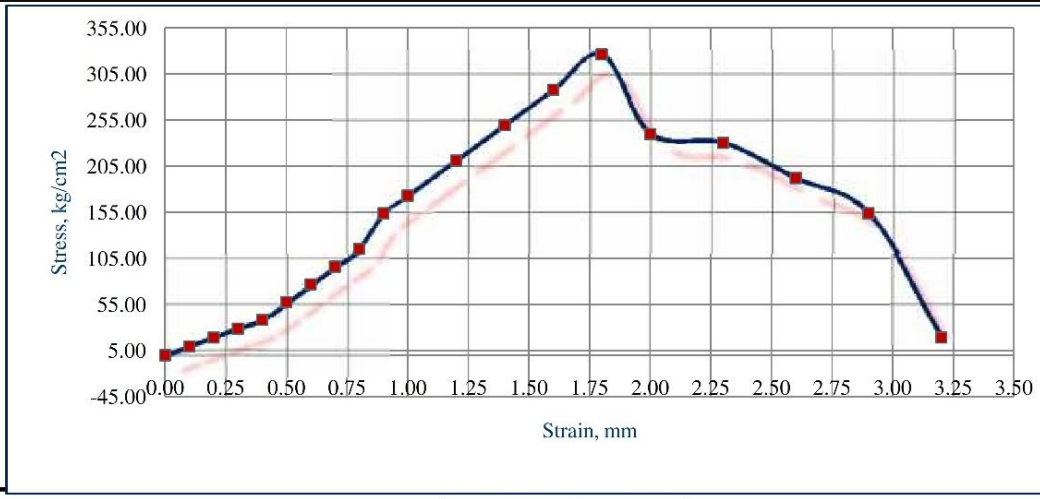


Figure 1-9: Stress-Strain Curve for core samples from drill hole NDH-3

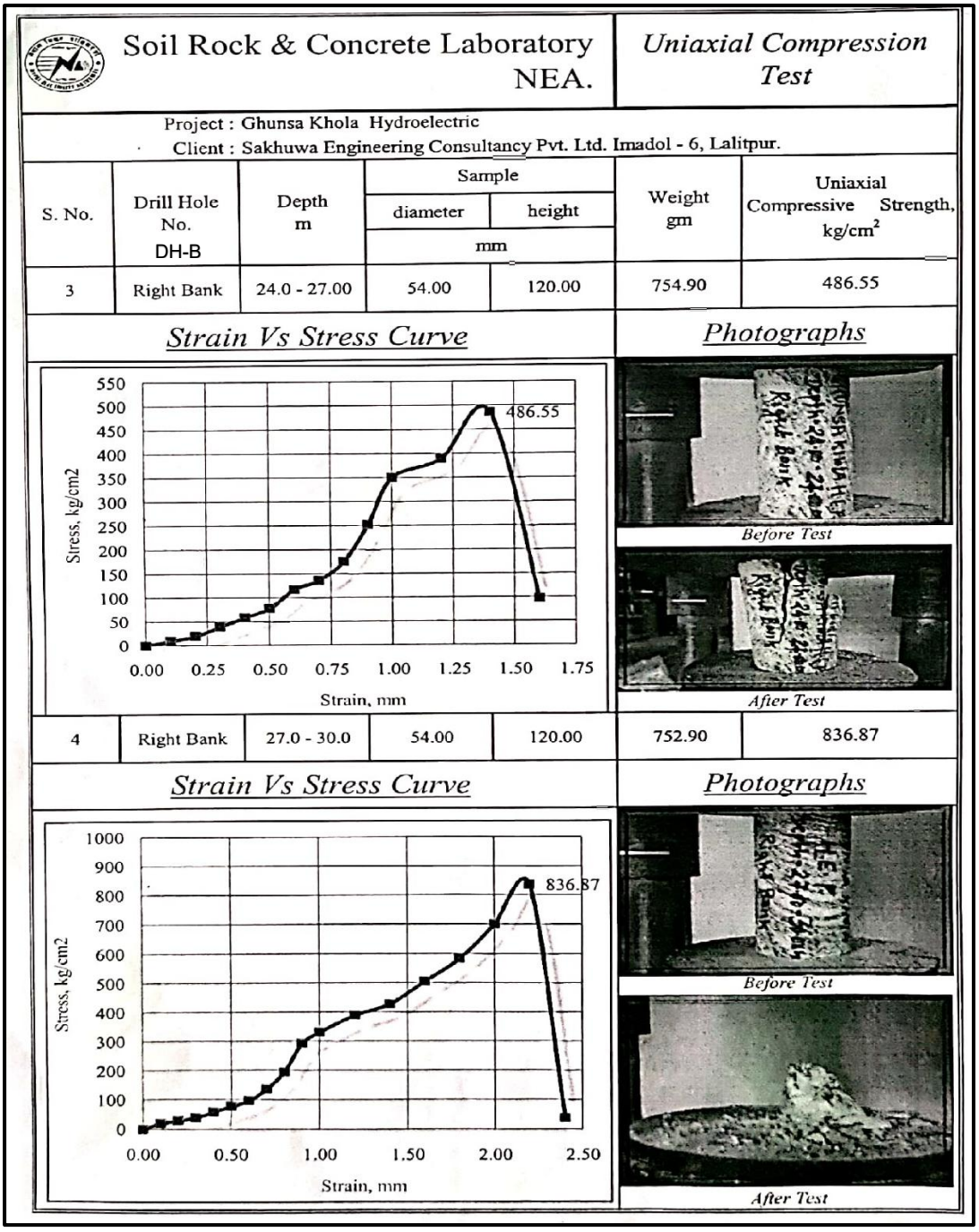
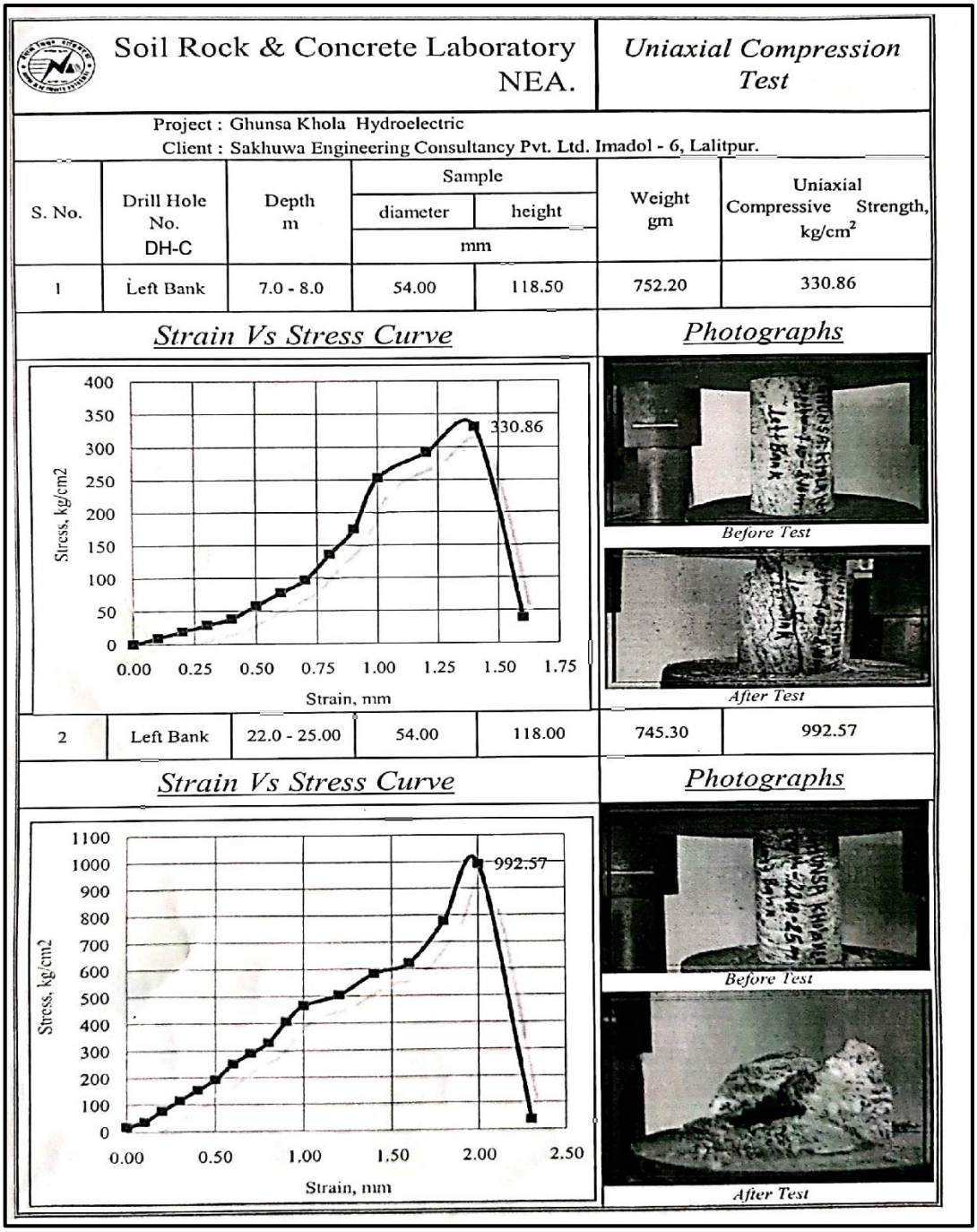


Figure 1-10: Stress-Strain Curve for core samples from drill hole DH-B



Soil Rock & Concrete Laboratory
NEA.

Uniaxial Compression Test

Project : Ghunsa Khola Hydroelectric
Client : Sakhuwa Engineering Consultancy Pvt. Ltd. Imadol - 6, Lalitpur.

S. No.	Drill Hole No. DH-C	Depth m	Sample diameter mm	Sample height mm	Weight gm	Uniaxial Compressive Strength, kg/cm²
1	Left Bank	7.0 - 8.0	54.00	118.50	752.20	330.86

Strain Vs Stress Curve

Photographs

Before Test

After Test

2	Left Bank	22.0 - 25.00	54.00	118.00	745.30	992.57

Strain Vs Stress Curve

Photographs

Before Test

After Test

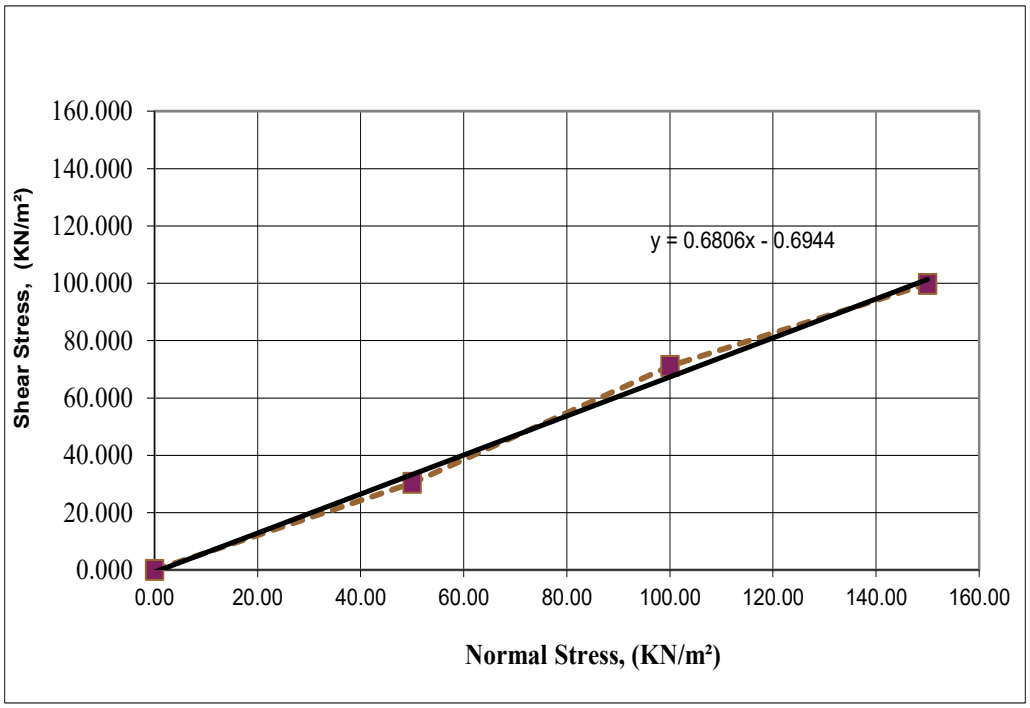
Figure 1-11: Stress-Strain Curve for core samples from drill hole DH-C

1.3 LABORATORY TESTS ON DH-A

DIRECT SHEAR TEST

Project:	Ghunsa Khola H.E.P.	**Location:**	Taplejung
Bore Hole No.:	DH-A(RB of Tamor Khola)	**Depth (m):**	(9-10) m
Size of box (m) :	0.06	**Height of Box (m):**	0.025
Strain Rate (mm/min):	1.25	**Strain Dial Const. (mm):**	0.01
Shear Dial Const. (KN/div):	0.3125	**Area of Sample (sq.m):**	0.0036
Volume (m^3):	0.00009		

Test No	Normal Load	Failure Load	Shear Stress	Wt of Sample	Water Content	Unit Weight KN/m^3			
		Load Dial Reading							
	kN/m^2	Div	kN/m^2	KN	%	Bulk	Av.	Dry	Av
1	0	0	0						
2	50.00	**35**	30	0.00137	14%	15.20		15.18	
3	100.00	**82**	71	0.00128	14%	14.18	14.74	14.16	14.72
4	150.00	**115**	100	0.00133	14%	14.83		14.81	



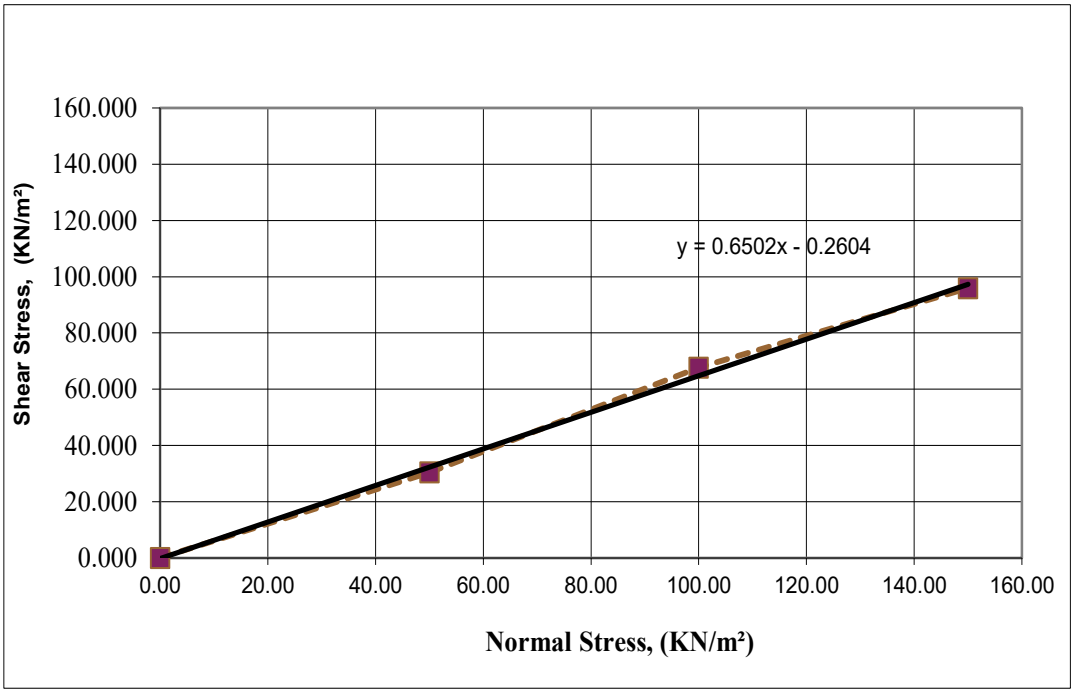
Results:

Angle of internal friction: **34**

Cohesion (KN/m²) : **0**

Project:	Ghunsa Khola H.E.P.	**Location:**	Taplejung
Bore Hole No.:	DH-A(RB of Tamor Khola)	**Depth (m):**	(23-24) m
Size of box (m) :	0.06	**Height of Box (m):**	0.025
Strain Rate (mm/min):	1.25	**Strain Dial Const(mm):**	0.01
Shear Dial Const. (KN/div):	0.3125	**Area of Sample (sq.m):**	0.0036
Volume (m^3):	0.00009		

Test No	Normal Load	Failure Load	Shear Stress	Wt of Sample	Water Content	Unit Weight KN/m^3			
		Load Dial Reading							
	kN/m^2	Div	kN/m^2	KN	%	Bulk	Av.	Dry	Av
1	0	0	0						
2	50.00	**35**	30	0.00119	13%	13.18		13.16	
3	100.00	**78**	68	0.00130	13%	14.49	14.15	14.48	14.13
4	150.00	**110.5**	96	0.00133	13%	14.78		14.76	



Results:

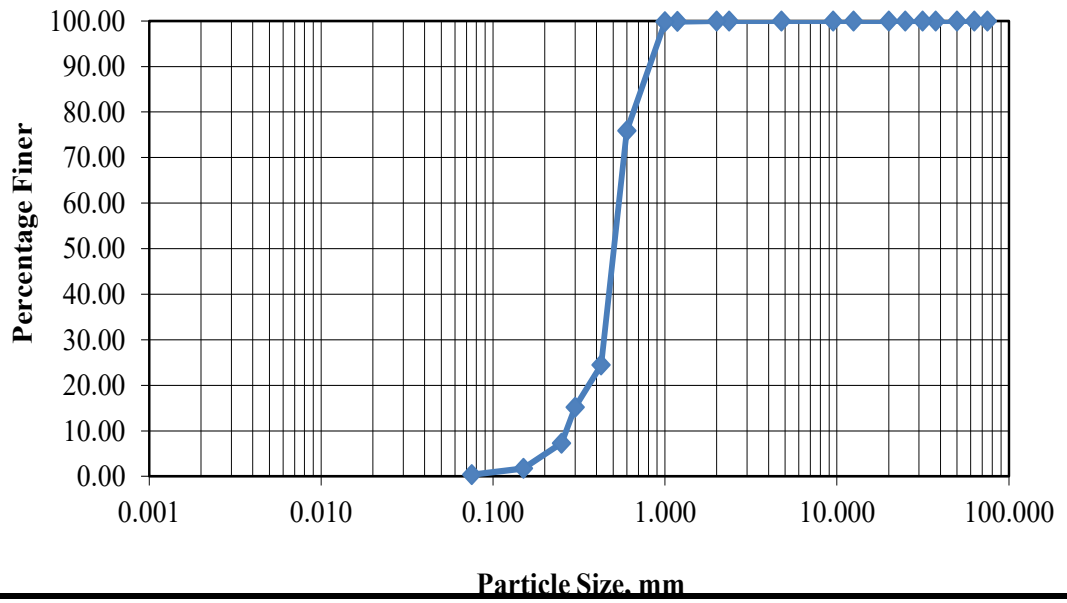
Angle of internal friction: **33**

Cohesion (KN/m²) : **0**

SIEVE ANALYSIS

Wt. of Sample (gms): 1,413.91 **Date :** 1/11/2076
Depth (m): (3-5) m

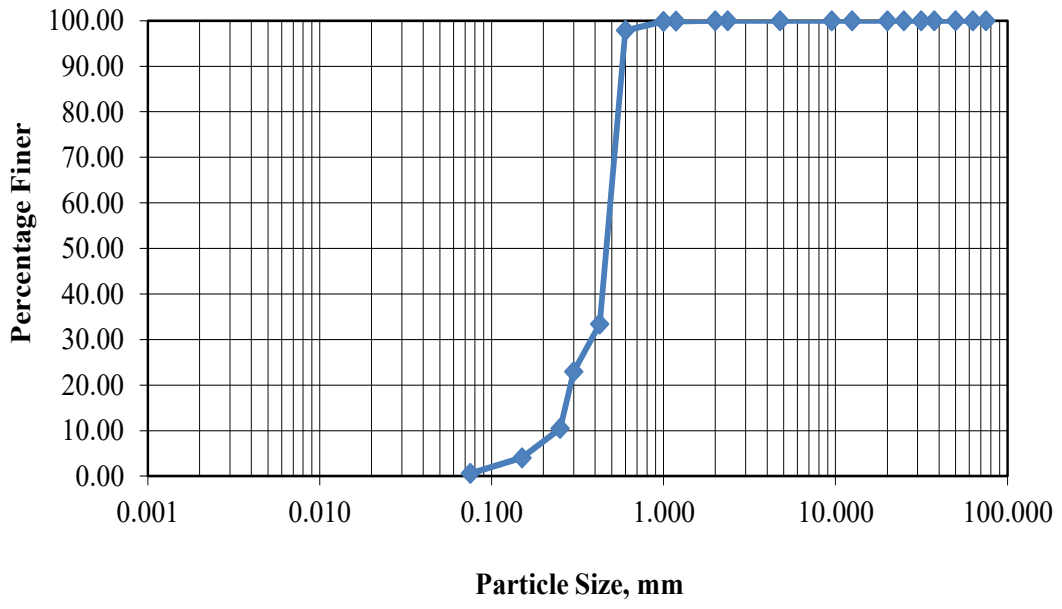
Seive Size mm	Wt. of Soil Retained gms	Cumulative Weight Retained gms	Cumulative Percentage Retained %	Percent Passing %	Remarks
75.000	0.00	0.00	0.00	100.00	
63.000	0.00	0.00	0.00	100.00	
50.000	0.00	0.00	0.00	100.00	
37.500	0.00	0.00	0.00	100.00	
31.500	0.00	0.00	0.00	100.00	
25.000	0.00	0.00	0.00	100.00	
20.000	0.00	0.00	0.00	100.00	Gravel 0.00 %
12.500	0.00	0.00	0.00	100.00	Coarse Sand 75.54 %
9.500	0.00	0.00	0.00	100.00	Medium Sand 24.10 %
4.750	0.00	0.00	0.00	100.00	Silt & Clay 0.36 %
2.360	0.00	0.00	0.00	100.00	
2.000	0.00	0.00	0.00	100.00	
1.180	1.72	1.72	0.12	99.88	
1.000	1.07	2.79	0.20	99.80	
0.600	338.39	341.18	24.13	75.87	
0.425	726.82	1068.00	75.54	24.46	
0.300	131.35	1199.35	84.83	15.17	
0.250	111.49	1310.84	92.71	7.29	
0.150	78.19	1389.03	98.24	1.76	
0.075	19.77	1408.80	99.64	0.36	
Pan	5.11	1413.91	100.00	0.00	



Wt. of Sample (gms): 1,076.97 **Date :** 1/11/2076

Depth (m): (9-10) m

Seive Size mm	Wt. of Soil Retained gms	Cumulative Weight Retained gms	Cumulative Percentage Retained %	Percent Passing %	Remarks
75.000	0.00	0.00	0.00	100.00	
63.000	0.00	0.00	0.00	100.00	
50.000	0.00	0.00	0.00	100.00	
37.500	0.00	0.00	0.00	100.00	
31.500	0.00	0.00	0.00	100.00	
25.000	0.00	0.00	0.00	100.00	
20.000	0.00	0.00	0.00	100.00	Gravel 0.00 %
12.500	0.00	0.00	0.00	100.00	Coarse Sand 66.57 %
9.500	0.00	0.00	0.00	100.00	Medium Sand 32.82 %
4.750	0.00	0.00	0.00	100.00	Silt & Clay 0.61 %
2.360	0.00	0.00	0.00	100.00	
2.000	0.00	0.00	0.00	100.00	
1.180	1.09	1.09	0.10	99.90	
1.000	0.52	1.61	0.15	99.85	
0.600	21.35	22.96	2.13	97.87	
0.425	694.00	716.96	66.57	33.43	
0.300	113.21	830.17	77.08	22.92	
0.250	134.29	964.46	89.55	10.45	
0.150	69.01	1033.47	95.96	4.04	
0.075	36.97	1070.44	99.39	0.61	
Pan	6.53	1076.97	100.00	0.00	

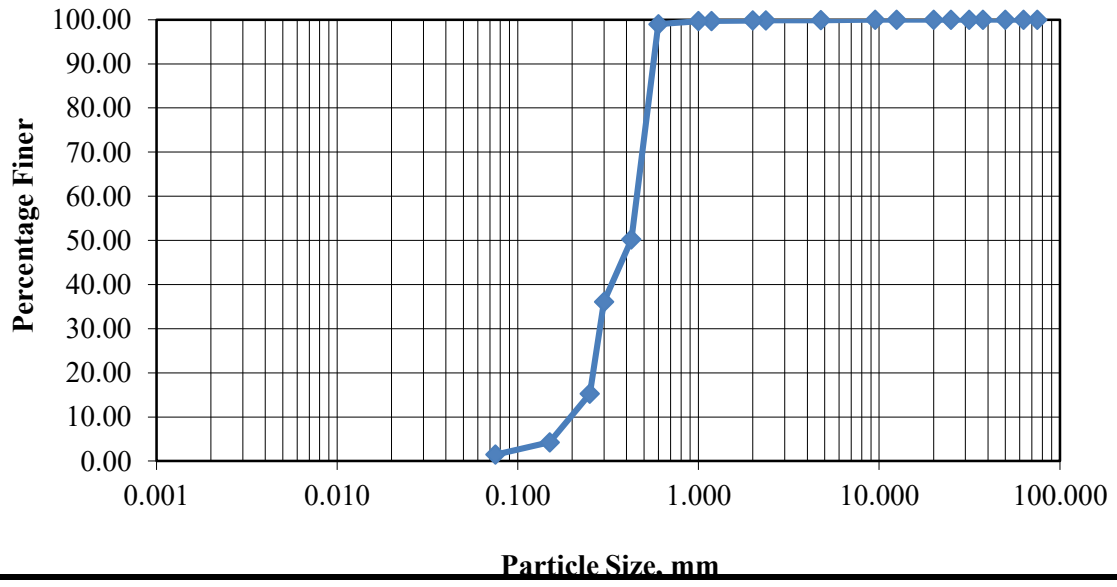


DH-A

Wt. of Sample (gms): 1,235.82 **Date :** 1/11/2076

Depth (m): (13-15) m

Seive Size mm	Wt. of Soil Retained gms	Cumulative Weight Retained gms	Cumulative Percentage Retained %	Percent Passing %	Remarks
75.000	0.00	0.00	0.00	100.00	
63.000	0.00	0.00	0.00	100.00	
50.000	0.00	0.00	0.00	100.00	
37.500	0.00	0.00	0.00	100.00	
31.500	0.00	0.00	0.00	100.00	
25.000	0.00	0.00	0.00	100.00	
20.000	0.00	0.00	0.00	100.00	Gravel 0.11 %
12.500	0.00	0.00	0.00	100.00	Coarse Sand 49.69 %
9.500	0.00	0.00	0.00	100.00	Medium Sand 48.74 %
4.750	1.31	1.31	0.11	99.89	Silt & Clay 1.46 %
2.360	0.73	2.04	0.17	99.83	
2.000	0.25	2.29	0.19	99.81	
1.180	1.27	3.56	0.29	99.71	
1.000	0.53	4.09	0.33	99.67	
0.600	8.69	12.78	1.03	98.97	
0.425	602.65	615.43	49.80	50.20	
0.300	174.13	789.56	63.89	36.11	
0.250	257.99	1047.55	84.77	15.23	
0.150	135.82	1183.37	95.76	4.24	
0.075	34.37	1217.74	98.54	1.46	
Pan	18.08	1235.82	100.00	0.00	

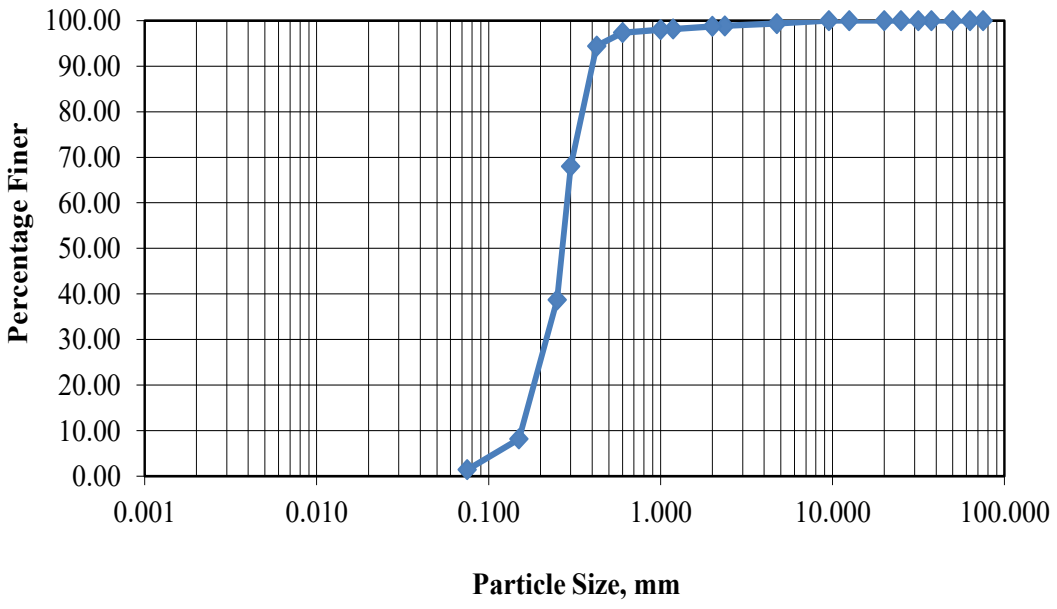


DH-A

Wt. of Sample (gms): 538.47 **Date :** 1/11/2076

Depth (m): (23-24) m

Seive Size mm	Wt. of Soil Retained gms	Cumulative Weight Retained gms	Cumulative Percentage Retained %	Percent Passing %	Remarks
75.000	0.00	0.00	0.00	100.00	
63.000	0.00	0.00	0.00	100.00	
50.000	0.00	0.00	0.00	100.00	
37.500	0.00	0.00	0.00	100.00	
31.500	0.00	0.00	0.00	100.00	
25.000	0.00	0.00	0.00	100.00	
20.000	0.00	0.00	0.00	100.00	Gravel 0.66 %
12.500	0.00	0.00	0.00	100.00	Coarse Sand 4.93 %
9.500	0.00	0.00	0.00	100.00	Medium Sand 92.96 %
4.750	3.57	3.57	0.66	99.34	Silt & Clay 1.45 %
2.360	2.74	6.31	1.17	98.83	
2.000	0.57	6.88	1.28	98.72	
1.180	3.15	10.03	1.86	98.14	
1.000	0.65	10.68	1.98	98.02	
0.600	3.57	14.25	2.65	97.35	
0.425	15.89	30.14	5.60	94.40	
0.300	142.05	172.19	31.98	68.02	
0.250	157.89	330.08	61.30	38.70	
0.150	164.22	494.30	91.80	8.20	
0.075	36.38	530.68	98.55	1.45	
Pan	7.79	538.47	100.00	0.00	

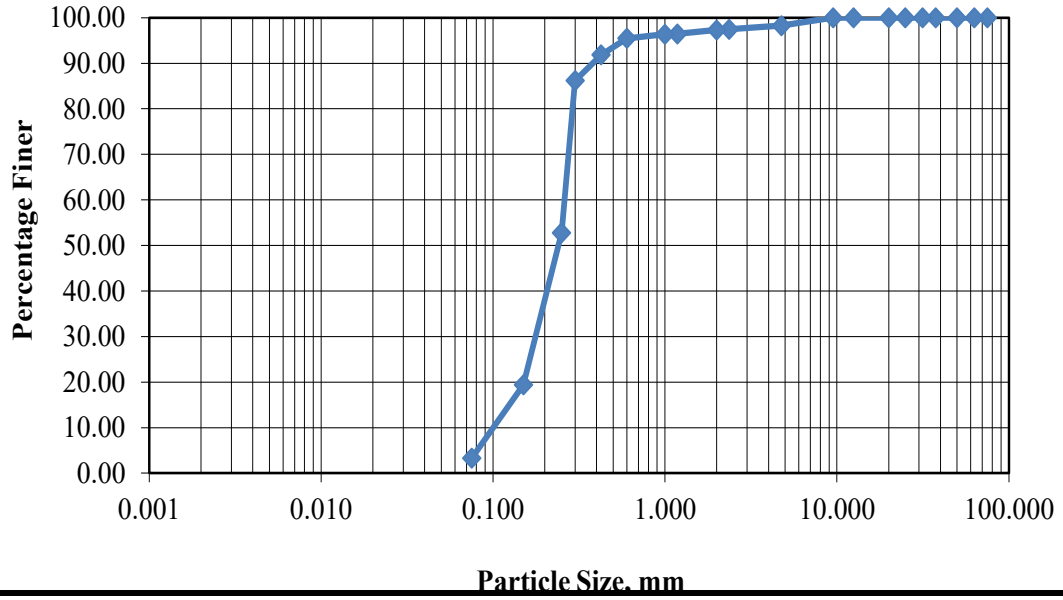


DH-A

Wt. of Sample (gms): 1,068.10 **Date :** 1/11/2076

Depth (m): (26-28) m

Seive Size mm	Wt. of Soil Retained gms	Cumulative Weight Retained gms	Cumulative Percentage Retained %	Percent Passing %	Remarks
75.000	0.00	0.00	0.00	100.00	
63.000	0.00	0.00	0.00	100.00	
50.000	0.00	0.00	0.00	100.00	
37.500	0.00	0.00	0.00	100.00	
31.500	0.00	0.00	0.00	100.00	
25.000	0.00	0.00	0.00	100.00	
20.000	0.00	0.00	0.00	100.00	Gravel 1.75 %
12.500	0.00	0.00	0.00	100.00	Coarse Sand 6.42 %
9.500	0.00	0.00	0.00	100.00	Medium Sand 88.57 %
4.750	18.65	18.65	1.75	98.25	Silt & Clay 3.27 %
2.360	8.64	27.29	2.56	97.44	
2.000	1.29	28.58	2.68	97.32	
1.180	9.41	37.99	3.56	96.44	
1.000	1.00	38.99	3.65	96.35	
0.600	9.52	48.51	4.54	95.46	
0.425	38.71	87.22	8.17	91.83	
0.300	60.57	147.79	13.84	86.16	
0.250	356.68	504.47	47.23	52.77	
0.150	356.84	861.31	80.64	19.36	
0.075	171.89	1033.20	96.73	3.27	
Pan	34.90	1068.10	100.00	0.00	



WATER CONTENT DETERMINATION

DH-A							
S.N.	Description		Depth				
			(3-5)m	(9-10)m	(13-15)m	(23-24)m	(26-28)m
1	Container No.		R13	R14	R8	R5	R11
2	Wt of saturated soil+container	grms	71.15	79.82	109.22	86.92	118.97
3	Wt of dry soil+container	grms	65.84	71.06	95.18	76.02	103.55
4	Wt. of container	grms	30.41	29.25	33.41	28.74	30.04
5	Wt. of water	grms	5.31	8.76	14.04	10.9	15.42
6	Wt. of dry sample	grms	35.43	41.81	61.77	47.28	73.51
7	Moisture content	%	14.99	20.95	22.73	23.05	20.98
Moisture Content(%)= (Weight of water/ Weight of dry sample)*100							

SPECIFIC GRAVITY (PYCNOMETER METHOD)

DH-A			
Specific Gravity			
Depth = 3-5 m			
Sampling Numbers		1	2
Pycnometer no:		7	9
Weight of Pycnometer + water + soil	(W1)grm	179.31	179.69
Weight of Pycnometer + water	(W2)grm	166.81	160.94
Dry Weight of Sample,	(Ws)grm	20.00	30.00
Temperature of Water, T deg C.		20.00	20.00
Specific Gravity of Water, at T deg C.	(Gt)	1.00	1.00
Specific Gravity of Soil Sample,	(Gs)	2.67	2.67
Average Sp. Gravity		2.67	
Gs = (Gt* Ws)/(Ws-W1-W2)			

DH-A			
Specific Gravity			
Depth = 9-10 m			
Sampling Numbers		1.00	2.00
Pycnometer no:		11.00	5.00
Weight of Pycnometer + water + soil	(W1)grm	176.56	181.48
Weight of Pycnometer + water	(W2)grm	164.08	162.74
Dry Weight of Sample,	(Ws)grm	20.00	30.00
Temperature of Water, T deg C.		20.00	20.00
Specific Gravity of Water, at T deg C.	(Gt)	1.00	1.00
Specific Gravity of Soil Sample,	(Gs)	2.66	2.66
Average Sp. Gravity		2.66	
Gs = (Gt* Ws)/(Ws-W1-W2)			

DH-A			
Specific Gravity			
Depth = 13-15 m			
Sampling Numbers		1.00	2.00
Pycnometer no:		6.00	8.00
Weight of Pycnometer + water + soil	(W1)grm	173.71	179.58
Weight of Pycnometer + water	(W2)grm	161.24	160.88
Dry Weight of Sample,	(Ws)grm	20.00	30.00
Temperature of Water, T deg C.		20.00	20.00
Specific Gravity of Water, at T deg C.	(Gt)	1.00	1.00
Specific Gravity of Soil Sample,	(Gs)	2.66	2.65
Average Sp. Gravity		2.66	
Gs = (Gt* Ws)/(Ws-W1-W2)			

DH-A			
Specific Gravity			
Depth = 23-24 m			
Sampling Numbers		1.00	2.00
Pycnometer no:		6.00	8.00
Weight of Pycnometer + water + soil	(W1)grm	173.69	179.54
Weight of Pycnometer + water	(W2)grm	161.24	160.88
Dry Weight of Sample,	(Ws)grm	20.00	30.00
Temperature of Water, T deg C.		13.00	13.00
Specific Gravity of Water, at T deg C.	(Gt)	1.00	1.00
Specific Gravity of Soil Sample,	(Gs)	2.65	2.65
Average Sp. Gravity		2.65	
Gs = (Gt* Ws)/(Ws-W1-W2)			

DH-A			
Specific Gravity			
Depth = 26-28 m			
Sampling Numbers		1.00	2.00
Pycnometer no:		13.00	19.00
Weight of Pycnometer + water + soil	(W1)grm	184.57	190.28
Weight of Pycnometer + water	(W2)grm	172.15	171.63
Dry Weight of Sample,	(Ws)grm	20.00	30.00
Temperature of Water, T deg C.		13.00	13.00
Specific Gravity of Water, at T deg C.	(Gt)	1.00	1.00
Specific Gravity of Soil Sample,	(Gs)	2.64	2.64
Average Sp. Gravity		2.64	
Gs = (Gt* Ws)/(Ws-W1-W2)			

1.4 LABORATORY TESTS ON DH-B

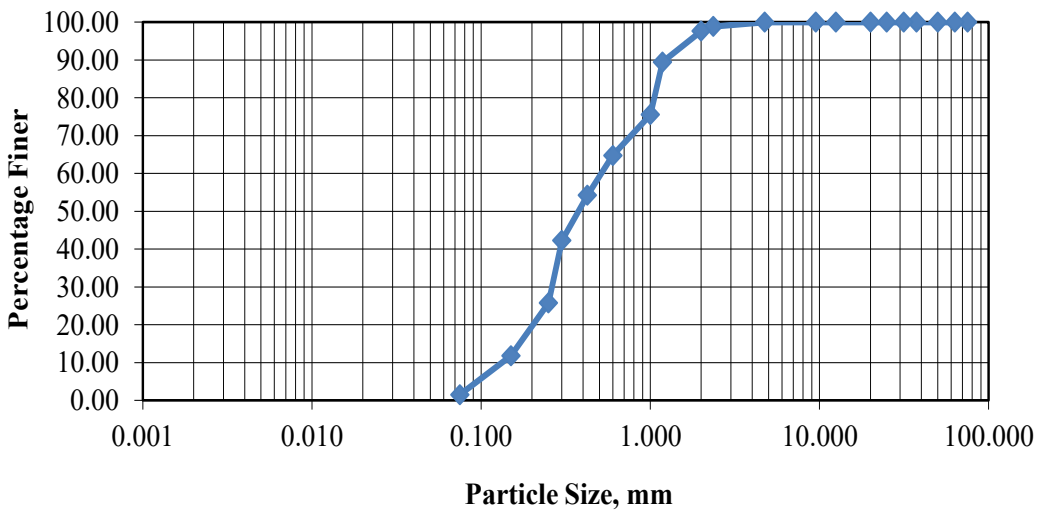
SIEVE ANALYSIS

DH-B

Wt. of Sample (gms): 928.78 **Date :** 03/03/2077

Depth (m): (3-4) m

Seive Size mm	Wt. of Soil Retained gms	Cumulative Weight Retained gms	Cumulative Percentage Retained %	Percent Passing %	Remarks
75.000	0.00	0.00	0.00	100.00	
63.000	0.00	0.00	0.00	100.00	
50.000	0.00	0.00	0.00	100.00	
37.500	0.00	0.00	0.00	100.00	
31.500	0.00	0.00	0.00	100.00	
25.000	0.00	0.00	0.00	100.00	
20.000	0.00	0.00	0.00	100.00	Gravel 0.00 %
12.500	0.00	0.00	0.00	100.00	Coarse Sand 45.74 %
9.500	0.00	0.00	0.00	100.00	Medium Sand 52.72 %
4.750	0.00	0.00	0.00	100.00	Silt & Clay 1.54 %
2.360	10.36	10.36	1.12	98.88	
2.000	11.29	21.65	2.33	97.67	
1.180	75.73	97.38	10.48	89.52	
1.000	129.57	226.95	24.44	75.56	
0.600	100.28	327.23	35.23	64.77	
0.425	97.56	424.79	45.74	54.26	
0.300	111.39	536.18	57.73	42.27	
0.250	153.28	689.46	74.23	25.77	
0.150	129.63	819.09	88.19	11.81	
0.075	95.37	914.46	98.46	1.54	
Pan	14.32	928.78	100.00	0.00	

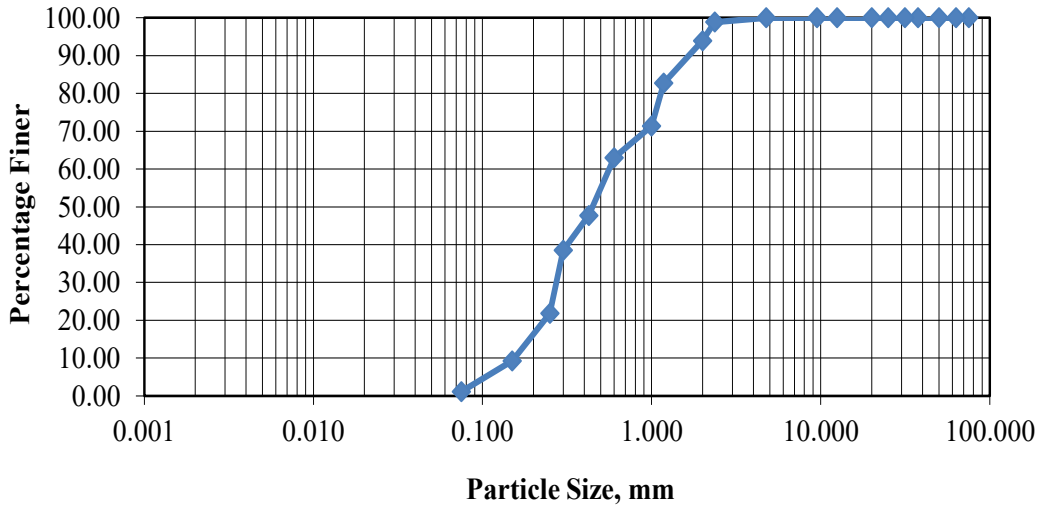


DH-B

Wt. of Sample (gms): 1,207.93 **Date :** 03/03/2077

Depth (m): (9-10) m

Seive Size mm	Wt. of Soil Retained gms	Cumulative Weight Retained gms	Cumulative Percentage Retained %	Percent Passing %	Remarks
75.000	0.00	0.00	0.00	100.00	
63.000	0.00	0.00	0.00	100.00	
50.000	0.00	0.00	0.00	100.00	
37.500	0.00	0.00	0.00	100.00	
31.500	0.00	0.00	0.00	100.00	
25.000	0.00	0.00	0.00	100.00	
20.000	0.00	0.00	0.00	100.00	Gravel 0.00 %
12.500	0.00	0.00	0.00	100.00	Coarse Sand 52.31 %
9.500	0.00	0.00	0.00	100.00	Medium Sand 46.56 %
4.750	0.00	0.00	0.00	100.00	Silt & Clay 1.13 %
2.360	13.56	13.56	1.12	98.88	
2.000	59.38	72.94	6.04	93.96	
1.180	135.15	208.09	17.23	82.77	
1.000	137.26	345.35	28.59	71.41	
0.600	101.37	446.72	36.98	63.02	
0.425	185.19	631.91	52.31	47.69	
0.300	111.25	743.16	61.52	38.48	
0.250	201.57	944.73	78.21	21.79	
0.150	151.35	1096.08	90.74	9.26	
0.075	98.26	1194.34	98.87	1.13	
Pan	13.59	1207.93	100.00	0.00	

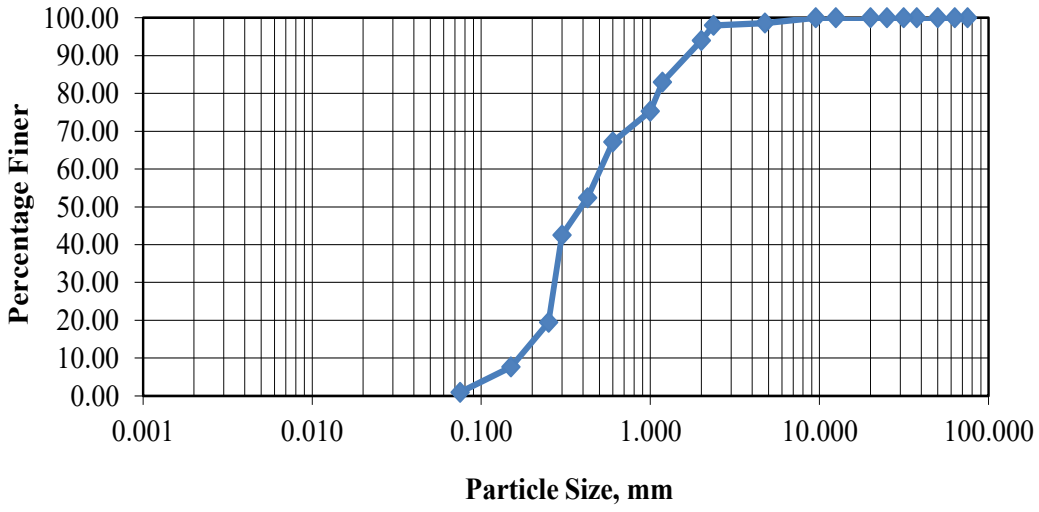


DH-B

Wt. of Sample (gms): 1,255.97 **Date :** 03/03/2077

Depth (m): (11-13) m

Seive Size mm	Wt. of Soil Retained gms	Cumulative Weight Retained gms	Cumulative Percentage Retained %	Percent Passing %	Remarks
75.000	0.00	0.00	0.00	100.00	
63.000	0.00	0.00	0.00	100.00	
50.000	0.00	0.00	0.00	100.00	
37.500	0.00	0.00	0.00	100.00	
31.500	0.00	0.00	0.00	100.00	
25.000	0.00	0.00	0.00	100.00	
20.000	0.00	0.00	0.00	100.00	Gravel 1.40 %
12.500	0.00	0.00	0.00	100.00	Coarse Sand 46.21 %
9.500	0.00	0.00	0.00	100.00	Medium Sand 51.43 %
4.750	17.57	17.57	1.40	98.60	Silt & Clay 0.96 %
2.360	7.59	25.16	2.00	98.00	
2.000	50.21	75.37	6.00	94.00	
1.180	138.06	213.43	16.99	83.01	
1.000	96.54	309.97	24.68	75.32	
0.600	102.00	411.97	32.80	67.20	
0.425	186.00	597.97	47.61	52.39	
0.300	124.00	721.97	57.48	42.52	
0.250	290.00	1011.97	80.57	19.43	
0.150	148.00	1159.97	92.36	7.64	
0.075	84.00	1243.97	99.04	0.96	
Pan	12.00	1255.97	100.00	0.00	

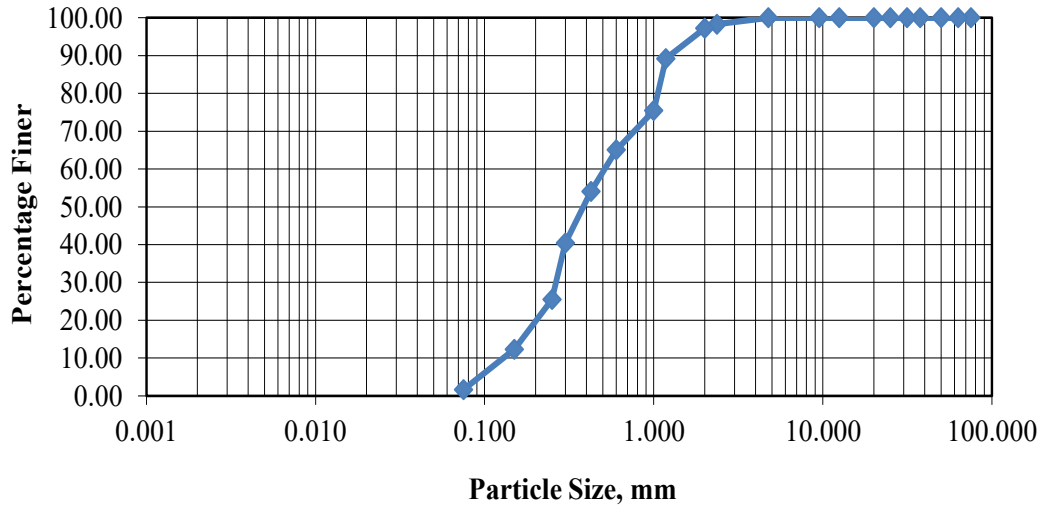


DH-B

Wt. of Sample (gms): 914.87 **Date :** 03/03/2077

Depth (m): (18-20) m

Seive Size mm	Wt. of Soil Retained gms	Cumulative Weight Retained gms	Cumulative Percentage Retained %	Percent Passing %	Remarks
75.000	0.00	0.00	0.00	100.00	
63.000	0.00	0.00	0.00	100.00	
50.000	0.00	0.00	0.00	100.00	
37.500	0.00	0.00	0.00	100.00	
31.500	0.00	0.00	0.00	100.00	
25.000	0.00	0.00	0.00	100.00	
20.000	0.00	0.00	0.00	100.00	Gravel 0.00 %
12.500	0.00	0.00	0.00	100.00	Coarse Sand 45.94 %
9.500	0.00	0.00	0.00	100.00	Medium Sand 52.40 %
4.750	0.00	0.00	0.00	100.00	Silt & Clay 1.67 %
2.360	15.32	15.32	1.67	98.33	
2.000	10.19	25.51	2.79	97.21	
1.180	73.25	98.76	10.79	89.21	
1.000	125.63	224.39	24.53	75.47	
0.600	95.21	319.60	34.93	65.07	
0.425	100.65	420.25	45.94	54.06	
0.300	125.12	545.37	59.61	40.39	
0.250	136.39	681.76	74.52	25.48	
0.150	120.50	802.26	87.69	12.31	
0.075	97.35	899.61	98.33	1.67	
Pan	15.26	914.87	100.00	0.00	

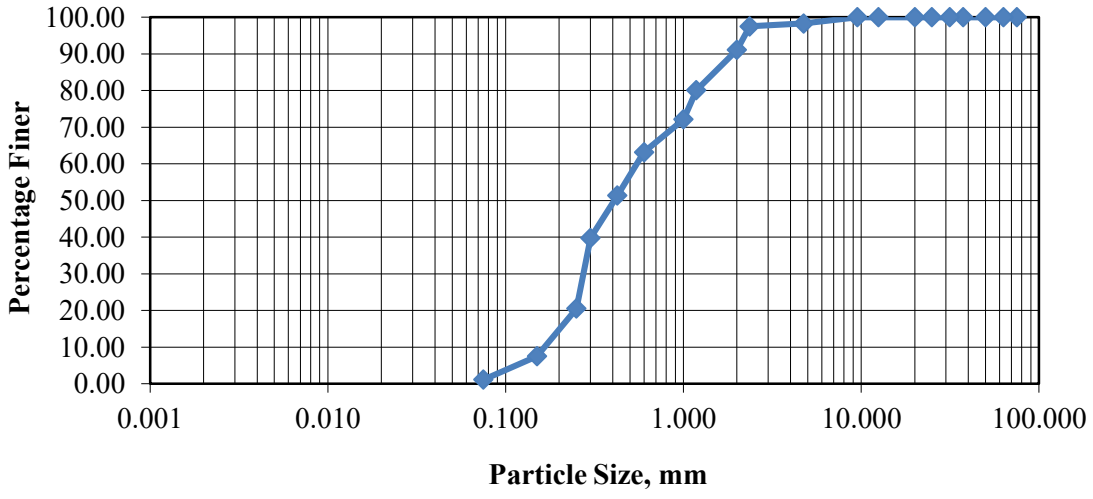


DH-B

Wt. of Sample (gms): 1,174.70 **Date :** 03/03/2077

Depth (m): (21-24) m

Seive Size mm	Wt. of Soil Retained gms	Cumulative Weight Retained gms	Cumulative Percentage Retained %	Percent Passing %	Remarks
75.000	0.00	0.00	0.00	100.00	
63.000	0.00	0.00	0.00	100.00	
50.000	0.00	0.00	0.00	100.00	
37.500	0.00	0.00	0.00	100.00	
31.500	0.00	0.00	0.00	100.00	
25.000	0.00	0.00	0.00	100.00	
20.000	0.00	0.00	0.00	100.00	Gravel 1.67 %
12.500	0.00	0.00	0.00	100.00	Coarse Sand 46.98 %
9.500	0.00	0.00	0.00	100.00	Medium Sand 50.22 %
4.750	19.57	19.57	1.67	98.33	Silt & Clay 1.14 %
2.360	9.25	28.82	2.45	97.55	
2.000	75.39	104.21	8.87	91.13	
1.180	129.69	233.90	19.91	80.09	
1.000	93.12	327.02	27.84	72.16	
0.600	105.06	432.08	36.78	63.22	
0.425	139.35	571.43	48.64	51.36	
0.300	135.60	707.03	60.19	39.81	
0.250	226.32	933.35	79.45	20.55	
0.150	153.00	1086.35	92.48	7.52	
0.075	75.00	1161.35	98.86	1.14	
Pan	13.35	1174.70	100.00	0.00	



Annex 7: Construction Material Investigations

1. CONSTRUCTION MATERIALS SURVEY

1.1 GENERAL

Eight test pits in early feasibility stage and four new test pits were excavated during this study to evaluate the quality of construction materials during previous feasibility study. Four new test pits are NTP-1, NTP-2, NTP-3 and NTP-4. These pits were logged, and representative samples were collected from their walls for laboratory tests. Details of the pits and the results of laboratory tests are presented in the following sections.

1.2 TEST PIT LOGGING

Test Pit TP-1 was excavated up to 3 m depth ona 500 x 50 m flood plainin the left bank of the Ghunsa Khola opposite to the portal of Main Access Tunnel (MAT) at Japantar.Mostly boulders and cobbles with some gravels of quartzite, and gneiss with fine to medium grained sand were found upto 3 m depth.Samples for laboratory testing were collected for fine and coarse aggregates.

Test Pit TP-2 was excavated up to 3 m depth on a 300x 50mflood plain developed in the left bank of the Ghunsa Khola opposite to the portal of cable cum ventilation tunnel at Japantar. The plain sediments contain considerable amount of gravels, cobbles and boulders of gneiss and quartzite. Samples for laboratory testing were collected for coarse aggregates.

Test Pit TP-3 was excavated up to 3 m depth on a 250 x 30 m gravelly bar in the left bank of the Ghunsa Khola at about 350 m upstream of suspension bridge near pothead yard at Japantar. The bar consists of cobbles and boulders of quartzite and gneiss with fine to medium grained light grey coloured sands.

Test Pit TP-4 was excavated up to 3 m depth on a 300 x 30m gravelly bar with considerable amount of sand with well sorted gravels, cobbles and boulders in the left bank of the Ghunsa Khola at about 550 m upstream of suspension bridge near pothead yard at Japantar. Representative sample was collected for laboratory testing of fine and coarse aggregates.

Test Pit TP-5 was excavated on a 300 x 40m sandy longitudinal bar in the left bank of Ghunsa Khola at about 300 m up from the suspension bridge at Lamatar. The deposit is dominated by gravels, cobbles and boulders of gneiss and quartzite with fine to coarse grained sands. Samples for laboratory testing were collected for coarse and fine aggregates.

Test Pit TP-6 was excavated on 300 ×250mwide colluvium located in the left bank along Ghunsa Kholaat about 200 m downstream of the weir axis at Lamatar. It has a considerable amount of cohesive material with sands and gravels. Samples for laboratory testing were collected for impervious core material.

Test Pit TP-7 was excavated up to 3 m depth on a 250 x 30m flood plain located in the left bank along Ghunsa Khola at about 475 m downstream of the weir axis at Lamatar. The entire surface of the plain is mostly consisting of fine materials, black clays with some pebbles and cobbles along with medium to coarse grained sand.

Test Pit TP-8 was excavated up to 3 m depth on a 300 x 30 m silty flood plain with colluvial deposits located in the left bank along Ghunsa Khola at about 800 m downstream of the weir axis at Lamatar. It was found to be containing mostly gravels with sands and silts.

New Test Pit NTP-1 was excavated on a triangular sand deposits developed within terrace deposits on the right bank of Ghunsa Khola around drill hole NDH-5. About 4-5 m thick entirely sand deposits are developed at the pit site. Sample was taken to evaluate the quality of sands.

New Test Pit NTP-2 was excavated on a circular sand deposit developed within the terrace deposits of Ghunsa Khola on its right bank about 200 m downstream of Japantar. It mostly contains sands with some gravels. Sample was taken to evaluate the quality of sands as fine aggregates.

New Test Pit NTP-3 was excavated on a huge alluvial fan deposit developed on the left bank of Ghunsa Khola at Lamatar. It is mostly composed of sandy gravels. Sample was taken to evaluate the quality of fine and coarse aggregates.

New Test Pit NTP-4 was excavated on an alluvial land deposit developed on the left bank of Ghunsa Khola, opposite to the proposed portal of access tunnel, about 300 m down from the weir axis. It is also mostly composed of sandy gravels. Sample was taken to evaluate the quality of fine and coarse aggregates.

Test pit logs and photos are presented in Photo 1-1 to Photo 1-22 respectively.

TP-1

GEOLOGICAL LOG OF TEST PIT

PROJECT: GHUNSA KHOLA HEP, TAPLEJUNG, NEPAL
PIT NO.: TP-1
LOCATION: 579836 E, 3046433 N
ELEVATION:
PIT SIZE 2.0 X 2.0 X 3.0 M
EXPECTED BOUNDARY AREA:
THICKNESS (APPROX.):
LOGGED BY: MALIRAM CHAUDHARY

DEPTH (M)	LITHOLOGY: DESCRIPTION	LOG	TEST RESULTS: LAA %	FI %	EI %	Sp. Gr.	WA %	ACV %	Bulk Density kg/m^3	SSD %	AAR- Chem. mmole/ lt.	AAR- Mortar bar mmole/ lt.	REMARKS
0.0 – 2.0	Mostly boulders and cobbles of quartzite and gneiss with sands.		26.2	-	-	2.610	0.936	-	1656	1.26	-	-	R_c= 62.9 mmole/lt. S_c= 18.2 mmole/lt. R_c= Reduction in Alkanity S_c= Reactive Silica
2.0 – 3.0	Fine to medium grained sand with some pebbles, cobbles and boulders of quartzite and gneiss												

Photo 1-1: Test Pit TP-1

1.3 LABORATORY TESTS ON TEST PIT TP-1

UNIT WEIGHT

TP-1

Date :-2011-8-23

Determination Procedure	Compact			Loose			Units
Code number	1	2	3	1	2	3	
Measure Volume	1316.70	1316.70	1316.70	1316.70	1316.70	1316.70	cc
Weight of measure + Sample	4095.0	4110.0	4100.0	3781.0	3788.0	3780.0	gm
Weight of clean dry measure	1602.0	1602.0	1602.0	1602.0	1602.0	1602.0	gm
Net weight Sample	2493.0	2508.0	2498.0	2179.0	2186.0	2178.0	gm
Loose unit wt. of Sample	1.893	1.905	1.897	1.655	1.660	1.654	gm/cc
Loose unit wt. of Sample	1893.37	1904.76	1897.17	1654.89	1660.21	1654.14	kg/m^3
Unit weight Average	1898.433			1656.414			kg/m^3

Compaction Unit Weight= 1898.433 kg/m^3

Loose Unit Weight = 1656.414 kg/m^3

SPECIFIC GRAVITY AND WATER ABSORPTION

TP -1

Date: 2011-08-24

	Determination No.	1	2	Unit
A	Wt. of Oven Dry Sample in Air	1229	1228	gms
B	Wt. of SSD Sample in Air	1242	1238	gms
C_1	Wt. of Sample + Container in Water	1521	1517	gms
C_2	Wt. of Container in Water	749.6	749.6	gms
$C=(C_1 - C_2)$	Wt. of SSD Sample in Water	771.4	767.4	gms
A/(B - C)	Bulk Specific Gravity (Oven Dry)	2.612	2.609	
B/(B - C)	Bulk Specific Gravity (SSD)	2.639	2.631	
A/(A - C)	Apparent Specific Gravity	2.686	2.666	
(B-A)/Ax100	Absorption %	1.058	0.814	
Average Bulk Specific Gravity (Oven Dry)		2.610		
Average Bulk Specific Gravity (SSD)		2.635		
Average Bulk Specific Gravity (Appa)		2.676		
Average Absorption %		0.936		

LOS ANGELES ABRASION TEST

TP-1

Date: 2011-08-24

S.No.	Description	Quantity	Remarks
1	Total weight of aggregate sample (gms)	5000	
2	Weight of aggregate retained on 1.7mm sieve after test (gms)	3690	< 35%
3	Aggregate wear in %	26.2	
	Reported LAA value %	26.2	

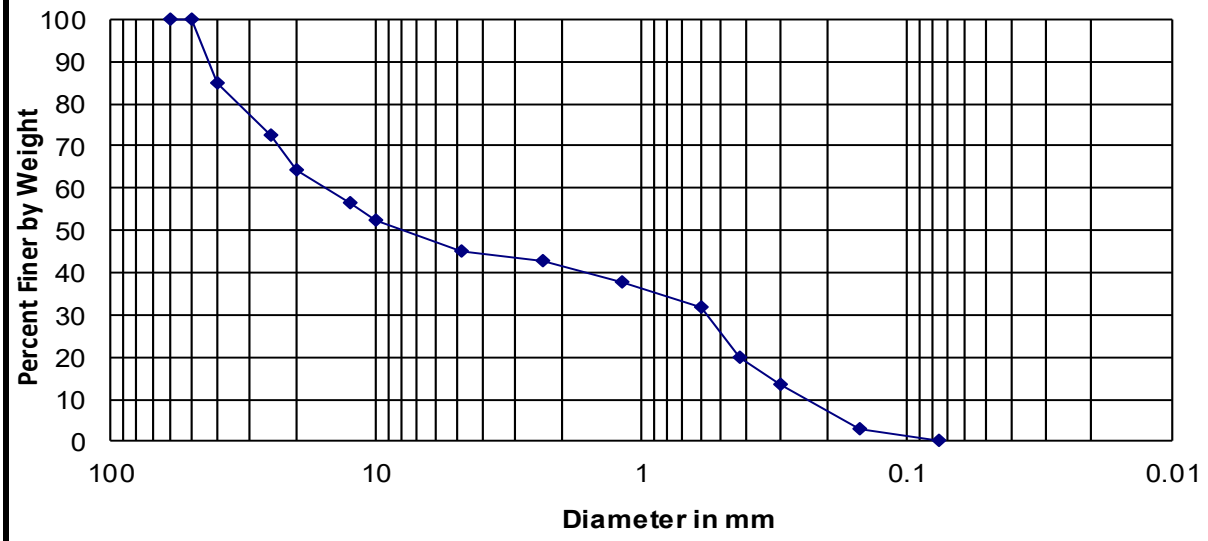
SIEVE ANALYSIS

TP-1

Wt. of Materials : 3620 gms **Date:-2011-8-22**

Sieve Size mm	Wt. Retained	Wt. Passing gm	% Passing	Remarks		
Total weight	3620.00					
60	0.00	3620.00	100.00			
50	0.00	3620.00	100.00			
40	550.00	3070.00	84.81			
25	437.00	2633.00	72.73			
20	314.00	2319.00	64.06			
12.5	281.00	2038.00	56.30	Gravel	55.11	%
10	136.00	1902.00	52.54	Coarse Sand	25.04	%
4.75	277.00	1625.00	44.89	Medium Sand	19.46	%
2.36	74.35	1550.65	42.84	Silt & Clay	0.39	%
1.18	180.56	1370.10	37.85			
0.6	223.04	1147.06	31.69			
0.425	428.38	718.68	19.85			
0.3	233.66	485.02	13.40			
0.15	385.89	99.13	2.74			
0.075	84.97	14.16	0.39			
PAN	14.16	0.00	0.00			
WASH	0.00					
TOTAL WT.	3620.00					

GRAIN SIZE DISTRIBUTION CURVE



TP-1				Solution used:	Na_2SO_4
Weight of air dry sample before washing(A ,gm) =				7850.00	
Weight of air dry sample after washing(B ,gm) =				7840.00	
SIEVE TEST BEFORE SOAKING				Date:	2011-09-15
Sieve	C	D	Fraction	E	Sample required
		% Ret	Number		for test
(mm)	Wt. Ret.	= C/Ax100%		Fraction % Ret.	if col. E ≥ 5%
75					3000 K 300
63			5		3000 K 300
50					3000 K 300
37.5	5005.00	63.8	4	63.8	2000 K 200
25	1010.00	12.9			1000 K 50
20	511.00	6.5	3	19.4	500 K 30
12.5	670.00	8.5			670 K 10
10	330.00	4.2	2	12.7	330 K 5
4.75	300.00	3.8	1		300 K 5
<4.75		0.0	X	3.8	None Tested
Pan	0.00				
Total	7826.00				
SIEVE TEST AFTER SOAKING					
FRACTION	F	Sieve	G	H	Wt.Loss = (HxE)/100
	TOTAL	(mm)	FR. WT.	% loss (F-G)/Fx100%	(%)
	FR. WT.		After Seiving		
5					
4	5005.00	31	4940	1.30	0.651
3	1521.00	16	1492	1.91	0.369
2	1000.00	8	985	1.50	0.191
1	300.00	4	296	1.33	0.051
Total	7826.00		7713		1.262
Percentage of loss of weight=			1.26	%	

SULPHATE SOUNDNESS OF COARSE AGGREGATES
ALKALI REACTIVITY - CHEMICAL METHOD

Sample: Aggregate		Received: 2011-08-23
		Completed:2011-08-28
Sample	**Observed Values,(%)**	
	Reduction in Alkalinity Rc(mmole/lt)	**Reactive Silica Sc(mmole/lt)**
TP1	62.9	18.20

TP-2

GEOLOGICAL LOG OF TEST PIT

PROJECT: GHUNSA KHOLA HEP, TAPLEJUNG, NEPAL
PIT NO.: TP-2
LOCATION: 580259 E, 3046819 N
ELEVATION:
PIT SIZE 2.0 X 2.0 X 3.0 M
EXPECTED BOUNDARY AREA:
THICKNESS (APPROX.):
LOGGED BY: MALIRAM CHAUDHARY

DEPTH (M)	LITHOLOGY: DESCRIPTION	LOG	TEST RESULTS: LAA %	FI %	EI %	Sp. Gr.	WA %	ACV %	Bulk Density kg/m^3	SSD %	AAR- Chem. mmole/ lt.	AAR- Mortar bar mmole/ lt.	REMARKS
0.0	Fine to medium grained sand with some pebbles and cobbles of granite and gneiss.												
1.0	Pebbles, cobbles and boulders of gneisses and quartzite.		34.0	-	-	2.591	0.669	-	1602	1.10	-	-	R_c= 55.3 mmole/lt. S_c= 14.5 mmole/lt. R_c= Reduction in Alkanity S_c= Reactive Silica
2.0	Fine to medium grained sand with some pebbles, cobbles and boulders of quartzite and gneiss												
3.0													

Photo 1-2: Test Pit 2

1.4 LABORATORY TESTS ON TEST PIT TP-2

UNIT WEIGHT

TP-2

Date :-2011-8-23

Determination Procedure	Compact			Loose			Units
Code number	1	2	3	1	2	3	
Measure Volume	1316.70	1316.70	1316.70	1316.70	1316.70	1316.70	cc
Weight of measure + Sample	4012.0	4020.0	4015.0	3700.0	3710.0	3725.0	gm
Weight of clean dry measure	1602.0	1602.0	1602.0	1602.0	1602.0	1602.0	gm
Net weight Sample	2410.0	2418.0	2413.0	2098.0	2108.0	2123.0	gm
Loose unit wt. of Sample	1.830	1.836	1.833	1.593	1.601	1.612	gm/cc
Loose unit wt. of Sample	1830.33	1836.41	1832.61	1593.38	1600.97	1612.36	kg/m^3
Unit weight Average	1833.118			1602.238			kg/m^3

Compaction Unit Weight= 1833.118 kg/m^3

Loose Unit Weight = 1602.238 kg/m^3

SPECIFIC GRAVITY AND WATER ABSORPTION

TP -2

Date: 2011-08-24

	Determination No.	1	2	Unit
A	Wt. of Oven Dry Sample in Air	1216	1176	gms
B	Wt. of SSD Sample in Air	1224	1184	gms
C_1	Wt. of Sample + Container in Water	1507	1477	gms
C_2	Wt. of Container in Water	749.6	749.6	gms
$C=(C_1 - C_2)$	Wt. of SSD Sample in Water	757.4	727.4	gms
A/(B - C)	Bulk Specific Gravity (Oven Dry)	2.606	2.576	
B/(B - C)	Bulk Specific Gravity (SSD)	2.623	2.593	
A/(A - C)	Apparent Specific Gravity	2.652	2.621	
(B-A)/A x 100	Absorption %	0.658	0.680	
Average Bulk Specific Gravity (Oven Dry)		2.591		
Average Bulk Specific Gravity (SSD)		2.608		
Average Bulk Specific Gravity (Appa)		2.637		
Average Absorption %		0.669		

LOS ANGELES ABRASION TEST

TP-2

Date: 2011-08-24

S.No.	Description	Quantity	Remarks
1	Total weight of aggregate sample (gms)	5000	< 35%
2	Weight of aggregate retained on 1.7mm sieve after test (gms)	3300	
3	Aggregate wear in %	34	
	Reported LAA value %	34	

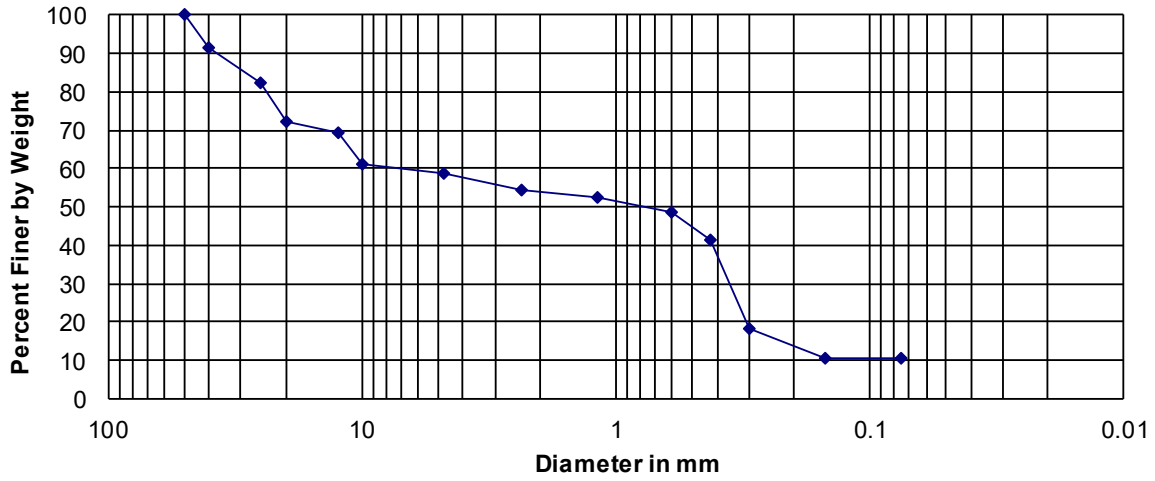
SIEVE ANALYSIS

TP-2

Wt. of Materials : 3223 gms **Date:-2011-8-22**

Sieve Size mm	Wt. Retained	Wt. Passing gm	% Passing	Remarks
Total weight	3223.00			
60	0.00	3223.00	100.00	
50	271.00	2952.00	91.59	
40	295.00	2657.00	82.44	
25	616.00	2336.00	72.48	
20	102.00	2234.00	69.31	
12.5	256.00	1978.00	61.37	Graval 45.45 %
10	89.00	1889.00	58.61	Corse Sand 36.13 %
4.75	131.00	1758.00	54.55	Medium Sand 9.27 %
2.36	64.27	1693.73	52.55	Silt & Clay 9.15 %
1.18	128.53	1565.20	48.56	
0.6	226.82	1338.38	41.53	
0.425	744.73	593.65	18.42	
0.3	257.06	336.58	10.44	
0.15	0.00	336.58	10.44	
0.075	41.58	295.00	9.15	
PAN	0.00	295.00	9.15	
WASH	0.00			
TOTAL WT.	3223.00			

GRAIN SIZE DISTRIBUTION CURVE



SULPHATE SOUNDNESS OF COARSE AGGREGATES

TP-2				Solution used:	Na_2SO_4
Weight of air dry sample before washing(A ,gm) =				7830.00	
Weight of air dry sample after washing(B ,gm) =				7821.00	
SIEVE TEST BEFORE SOAKING				Date:	2011-09-15
Sieve	C	D	Fraction	E	Sample required
		% Ret	Number		for test
(mm)	Wt. Ret.	= C/Ax100%		Fraction % Ret.	if col. E ≥ 5%
75					3000 K 300
63			5		3000 K 300
50	3000.00	38.3			3000 K 300
37.5	2002.00	25.6	4	63.9	2000 K 200
25	1005.00	12.8			1000 K 50
20	507.00	6.5	3	19.3	500 K 30
12.5	670.00	8.6			670 K 10
10	320.00	4.1	2	12.6	330 K 5
4.75	300.00	3.8	1		300 K 5
<4.75		0.0	X	3.8	None Tested
Pan	0.00				
Total	7804.00				
SIEVE TEST AFTER SOAKING					
FRACTION	F	Sieve	G	H	Wt.Loss = (HxE)/100
	TOTAL	(mm)	FR. WT.	loss (F-G)/Fx10	(%)
	FR. WT.		After Seiving		
5					
4	5002.00	31	4955	0.94	0.648
3	1512.00	16	1490	1.46	0.281
2	990.00	8	983	0.71	0.089
1	300.00	4	294	2.00	0.077
Total	7804.00		7722		1.095
Percentage of loss of weight=			1.10	%	

ALKALI REACTIVITY - CHEMICAL METHOD

Sample: Aggregate		Received: 2011-08-23
		Completed:2011-08-28
Sample	**Observed Values,(%)**	
	Reduction in Alkalinity Rc(mmole/lt)	**Reactive Silica Sc(mmole/lt)**
TP2	55.3	14.50

TP-3

GEOLOGICAL LOG OF TEST PIT

PROJECT: GHUNSA KHOLA HEP, TAPLEJUNG, NEPAL
PIT NO.: TP-3
LOCATION: 580472 E, 3046985 N
ELEVATION:

PIT SIZE: 2.0 X 2.0 X 3.0 M
EXPECTED BOUNDARY AREA:
THICKNESS (APPROX.):
LOGGED BY: MALIRAM CHAUDHARY

DEPTH (M)	LITHOLOGY: DESCRIPTION	LOG	TEST RESULTS: LAA %	FI %	EI %	Sp. Gr.	WA %	ACV %	Bulk Density kg/m^3	SSD %	AAR- Chem. mmole/ lt.	AAR- Mortar bar mmole/ lt.	REMARKS
0.0	Gravelly sand (pebbles, cobbles and boulders of quartzite and gneiss)												
1.0	Sandy gravel (mostly cobbles and boulders of quartzite and gneiss)		29.8	-	-	2.551	0.752	-	1551	2.87	-	-	R_c= 60.4 mmole/lt. S_c= 14.3 mmole/lt. R_c= Reduction in Alkanity S_c= Reactive Silica
2.0	Gravelly sand (fine to medium grained light grey coloured sand with fine gravels and some boulders)												
3.0													

Photo 1-3: Test Pit TP-3

1.5 LABORATORY TESTS ON TEST PIT TP-3

UNIT WEIGHT

TP-3

Date :-2011-8-23

Determination Procedure	Compact			Loose			Units
Code Number	1	2	3	1	2	3	
Measure Volume	1316.70	1316.70	1316.70	1316.70	1316.70	1316.70	cc
Weight of measure + Sample	4024	3965.000	4005.000	3664	3620	3650	gm
Weight of clean dry measure	1602.0	1602.0	1602.0	1602.0	1602.0	1602.0	gm
Net weight Sample	2422.0	2363.0	2403.0	2062.0	2018.0	2048.0	gm
Loose unit wt. of Sample	1.839	1.795	1.825	1.566	1.533	1.555	gm/cc
Loose unit wt. of Sample	1839.447	1794.638	1825.017	1566.036	1532.619	1555.404	kg/m^3
Unit weight Average	1819.701			1551.353			kg/m^3

Compaction Unit Weight= 1819.701 kg/m^3

Loose Unit Weight = 1551.353 kg/m^3

SPECIFIC GRAVITY AND WATER ABSORPTION

TP -3

Date: 2011-08-24

	Determination No.	1	2	Unit
A	Wt. of Oven Dry Sample in Air	1214	1176	gms
B	Wt. of SSD Sample in Air	1224	1184	gms
C_1	Wt. of Sample + Container in Water	1494	1476	gms
C_2	Wt. of Container in Water	749.6	749.6	gms
$C=(C_1 - C_2)$	Wt. of SSD Sample in Water	744.4	726.4	gms
A/(B - C)	Bulk Specific Gravity (Oven Dry)	2.531	2.570	
B/(B - C)	Bulk Specific Gravity (SSD)	2.552	2.587	
A/(A - C)	Apparent Specific Gravity	2.585	2.616	
(B-A)/Ax100	Absorption %	0.824	0.680	
Average Bulk Specific Gravity (Oven Dry)		2.551		
Average Bulk Specific Gravity (SSD)		2.570		
Average Bulk Specific Gravity (Appa)		2.600		
Average Absorption %		0.752		

LOS ANGELES ABRASION TEST

TP-3

Date: 2011-08-24

S.No.	Description	Quantity	Remarks
1	Total weight of aggregate sample (gms)	5000	< 35%
2	Weight of aggregate retained on 1.7mm sieve after test (gms)	3510	
3	Aggregate wear in %	29.8	
	Reported LAA value %	29.8	

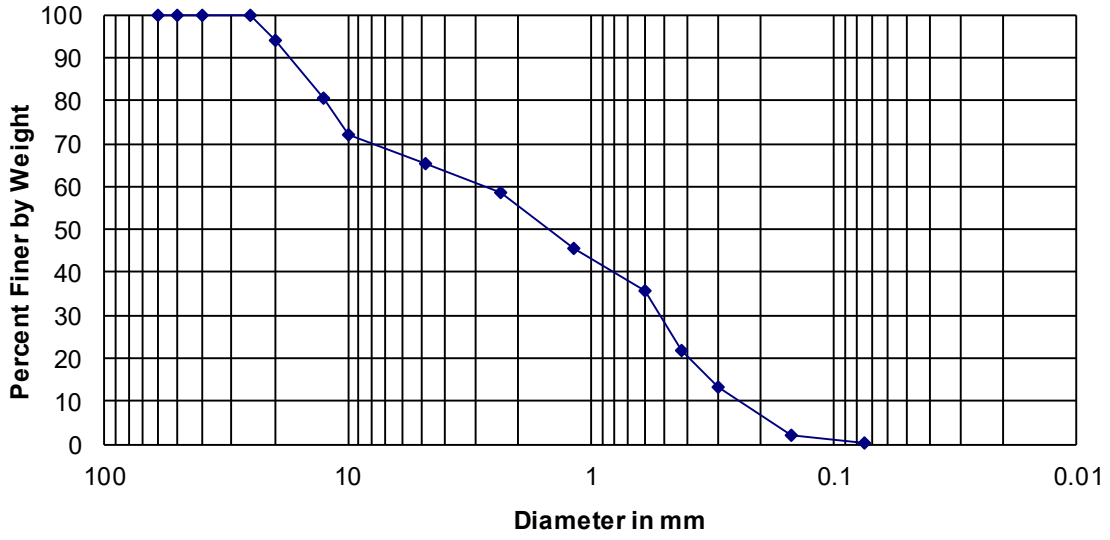
SIEVE ANALYSIS

TP-3

Wt. of Materials : 1937 gms **Date:-2011-8-22**

Sieve Size mm	Wt. Retained gm	Wt. Passing gm	% Passing	Remarks
Total weight	1937.00			
60	0.00	1937.00	100.00	
50	0.00	1937.00	100.00	
40	0.00	1937.00	100.00	
25	0.00	1937.00	100.00	
20	117.00	1820.00	93.96	
12.5	262.00	1558.00	80.43	Gravel 34.54 %
10	160.00	1398.00	72.17	Coarse Sand 43.83 %
4.75	130.00	1268.00	65.46	Medium Sand 21.48 %
2.36	134.00	1134.00	58.54	Silt & Clay 0.15 %
1.18	247.00	887.00	45.79	
0.6	199.00	688.00	35.52	
0.425	269.00	419.00	21.63	
0.3	159.00	260.00	13.42	
0.15	221.00	39.00	2.01	
0.075	36.00	3.00	0.15	
PAN	3.00	0.00	0.00	
WASH	0.00			
TOTAL WT.	1937.00			

GRAIN SIZE DISTRIBUTION CURVE



SULPHATE SOUNDNESS OF COARSE AGGREGATES

TP-3				Solution used:	Na_2SO_4
Weight of air dry sample before washing(A ,gm) =				2925.00	
Weight of air dry sample after washing(B ,gm) =				2912.00	
SIEVE TEST BEFORE SOAKING				Date:	2011-09-15
Sieve	C	D	Fraction	E	Sample required
		% Ret	Number		for test
(mm)	Wt. Ret.	= C/Ax100%		Fraction % Ret.	if col. E ≥ 5%
75					3000 K 300
63			5		3000 K 300
50					3000 K 300
37.5			4		2000 K 200
25	1015.00	34.7			1000 K 50
20	515.00	17.6	3	52.3	500 K 30
12.5	674.00	23.0			670 K 10
10	325.00	11.1	2	34.2	330 K 5
4.75	300.00	10.3	1		300 K 5
<4.75			X	10.3	None Tested
Pan	0.00				
Total	2829.00				
SIEVE TEST AFTER SOAKING					
FRACTION	F	Sieve	G	H	Wt.Loss = (HxE)/100
	TOTAL	(mm)	FR. WT.	loss (F-G)/Fx100	(%)
	FR. WT.		After Seiving		
5					
4					
3	1530.00	16	1477	3.46	1.812
2	999.00	8	980	1.90	0.650
1	300.00	4	288	4.00	0.410
Total	2829.00		2745		2.872
Percentage of loss of weight=			2.87	%	

ALKALI REACTIVITY - CHEMICAL METHOD

Sample: Aggregate		Received: 2011-08-23
		Completed:2011-08-28
Sample	**Observed Values,(%)**	
	Reduction in Alkalinity Rc(mmole/lt)	**Reactive Silica Sc(mmole/lt)**
TP3	60.4	14.30

TP-4

GEOLOGICAL LOG OF TEST PIT

PROJECT: GHUNSA KHOLA HEP, TAPLEJUNG, NEPAL
PIT NO.: TP-4
LOCATION: 580558 E, 3047064 N
ELEVATION:
PIT SIZE 2.0 X 2.0 X 3.0 M
EXPECTED BOUNDARY AREA:
THICKNESS (APPROX.):
LOGGED BY: MALIRAM CHAUDHARY

DEPTH (M)	LITHOLOGY: DESCRIPTION	LOG	TEST RESULTS: LAA %	FI %	EI %	Sp. Gr.	WA %	ACV %	Bulk Density kg/m^3	SSD %	AAR- Chem. mmole/ lt.	AAR- Mortar bar mmole/ lt.	REMARKS
0.0	Sandy gravels (pebbles, cobbles and mostly boulders of quartzite)												
1.0	Gravelly sand (well sorted pebbles and cobbles of quartzite and gneiss)		32.0	-	-	2.625	0.925	-	1807	1.37	-	-	R_c= 47.8 mmole/lt. S_c= 16.5 mmole/lt. R_c= Reduction in Alkanity S_c= Reactive Silica
2.0													
3.0													

Photo 1-4: Test Pit TP-4

1.6 LABORATORY TESTS ON TEST PIT TP-4

UNIT WEIGHT

TP-4

Date :-2011-8-23

Determination Procedure	Compact			Loose			Units
Code Number	1	2	3	1	2	3	
Measure Volume	1316.70	1316.70	1316.70	1316.70	1316.70	1316.70	cc
Weight of measure + Sample	4.255	4.275	4.265	3.895	3.970	3.990	gm
Weight of clean dry measure	1.602	1.602	1.602	1.602	1.602	1.602	gm
Net weight Sample	2.653	2.673	2.663	2.293	2.368	2.388	gm
Loose unit wt. of Sample	2.015	2.030	2.022	1.741	1.798	1.814	gm/cc
Loose unit wt. of Sample	2014.886	2030.075	2022.480	1741.475	1798.435	1813.625	kg/m^3
Unit weight Average	2022.480			1784.512			kg/m^3

Compaction Unit Weight= 2022.480 kg/m^3

Loose Unit Weight = 1784.512 kg/m^3

SPECIFIC GRAVITY AND WATER ABSORPTION

TP -4

Date: 2011-08-24

	Determination No.	1	2	Unit
A	Wt. of Oven Dry Sample in Air	1125	1146	gms
B	Wt. of SSD Sample in Air	1136	1156	gms
C_1	Wt. of Sample + Container in Water	1455	1471	gms
C_2	Wt. of Container in Water	749.6	749.6	gms
$C=(C_1 - C_2)$	Wt. of SSD Sample in Water	705.4	721.4	gms
A/(B - C)	Bulk Specific Gravity (Oven Dry)	2.613	2.637	
B/(B - C)	Bulk Specific Gravity (SSD)	2.638	2.660	
A/(A - C)	Apparent Specific Gravity	2.681	2.699	
(B-A)/Ax100	Absorption %	0.978	0.873	
Average Bulk Specific Gravity (Oven Dry)		2.625		
Average Bulk Specific Gravity (SSD)		2.649		
Average Bulk Specific Gravity (Appa)		2.690		
Average Absorption %		0.925		

LOS ANGELES ABRASION TEST

TP-4

Date: 2011-08-24

S.No.	Description	Quantity	Remarks
1	Total weight of aggregate sample (gms)	5000	
2	Weight of aggregate retained on 1.7mm sieve after test (gms)	3400	< 35%
3	Aggregate wear in %	32	
	Reported LAA value %	32	

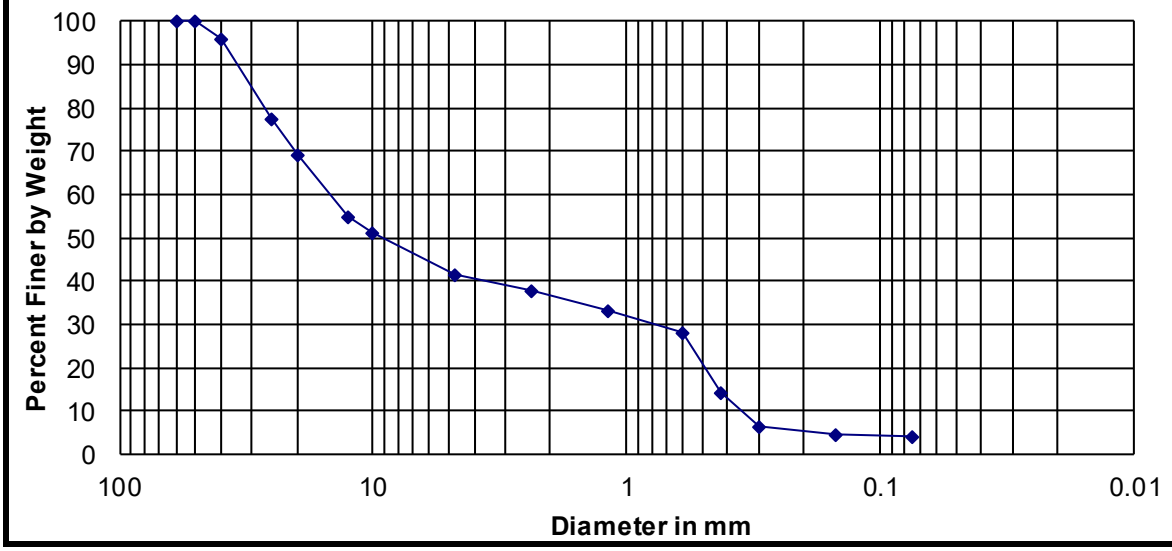
SIEVE ANALYSIS

TP-4

Wt. of Materials : 5509 gms **Date:-2011-8-22**

Sieve Size mm	Wt. Retained gm	Wt. Passing gm	% Passing	Remarks
Total weight	5509.00			
60	0.00	5509.00	100.00	
50	0.00	5509.00	100.00	
40	228.00	5281.00	95.86	
25	1239.00	4270.00	77.51	
20	451.00	3819.00	69.32	
12.5	795.00	3024.00	54.89	Graval 58.65 %
10	200.00	2824.00	51.26	Corse Sand 27.22 %
4.75	546.00	2278.00	41.35	Medium Sand 9.89 %
2.36	201.09	2076.91	37.70	Silt & Clay 4.24 %
1.18	259.74	1817.17	32.99	
0.6	256.95	1560.22	28.32	
0.425	782.02	778.20	14.13	
0.3	418.94	359.27	6.52	
0.15	100.54	258.72	4.70	
0.075	25.14	233.59	4.24	
PAN	5.59			
WASH	0.00			
TOTAL WT.	5509.00			

GRAIN SIZE DISTRIBUTION CURVE



SULPHATE SOUNDNESS OF COARSE AGGREGATES

TP-4				Solution used:	Na_2SO_4
Weight of air dry sample before washing(A ,gm) =				7890.00	
Weight of air dry sample after washing (B ,gm) =				7881.00	
SIEVE TEST BEFORE SOAKING				Date:	2011-09-15
Sieve	C	D	Fraction	E	Sample required
		% Ret	Number		for test
(mm)	Wt. Ret.	= C/Ax100%		Fraction % Ret.	if col. E ≥ 5%
75					3000 K 300
63			5		3000 K 300
50	3030.00	38.4			3000 K 300
37.5	2020.00	25.6	4	64.0	2000 K 200
25	1015.00	12.9			1000 K 50
20	510.00	6.5	3	19.3	500 K 30
12.5	671.00	8.5			670 K 10
10	320.00	4.1	2	12.6	330 K 5
4.75	300.00	3.8	1		300 K 5
<4.75			X	3.8	None Tested
Pan	0.00				
Total	7866.00				
SIEVE TEST AFTER SOAKING					
FRACTION	F	Sieve	G	H	Wt.Loss = (HxE)/100
	TOTAL	(mm)	FR. WT.	% loss (F-G)/Fx100	(%)
	FR. WT.		After Seiving		
5					
4	5050.00	31	4936	2.26	0.663
3	1525.00	16	1486	2.56	0.494
2	991.00	8	981	1.01	0.127
1	300.00	4	293	2.33	0.089
Total	7866.00		7696		1.372
Percentage of loss of weight=			1.37	%	

ALKALI REACTIVITY - CHEMICAL METHOD

Sample: Aggregate		Received: 2011-08-23
		Completed:2011-08-28
Sample	**Observed Values,(%)**	
	Reduction in Alkalinity Rc(mmole/lt)	**Reactive Silica Sc(mmole/lt)**
TP4	47.8	16.50

TP-5

GEOLOGICAL LOG OF TEST PIT

PROJECT: GHUNSA KHOLA HEP, TAPLEJUNG, NEPAL
PIT NO.: TP-5
LOCATION: 582700 E, 3048340 N
ELEVATION:

PIT SIZE 2.0 X 2.0 X 3.0 M
EXPECTED BOUNDARY AREA:
THICKNESS (APPROX.):
LOGGED BY: MALIRAM CHAUDHARY

DEPTH (M)	LITHOLOGY: DESCRIPTION	LOG	TEST RESULTS: LAA %	FI %	EI %	Sp. Gr.	WA %	ACV %	Bulk Density kg/m^3	SSD %	AAR- Chem. mmole/ lt.	AAR- Mortar bar mmole/ lt.	REMARKS
0.0 – 1.0	Sand with gravels.												
1.0 – 2.0	Dominantly sands.		-	-	-	2.549	1.000	-	1146	2.34	-	-	R_c= 45.3 mmole/lt. S_c= 19.9 mmole/lt. R_c= Reduction in Alkanity S_c= Reactive Silica
2.0 – 3.0													

C

Photo 1-5: Test Pit TP-5

1.7 LABORATORY TESTS ON TEST PIT TP-5

UNIT WEIGHT

TP-5

Date :-2011-8-23

Determination Procedure	Compact			Loose			Units
Code Number	1	2	3	1	2	3	
Measure Volume	1316.70	1316.70	1316.70	1316.70	1316.70	1316.70	
Weight of measure + Sample	3380	3400.000	3390.000	3108	3100	3125	gm
Weight of clean dry measure	1602.0	1602.0	1602.0	1602.0	1602.0	1602.0	gm
Net weight Sample	1778.0	1798.0	1788.0	1506.0	1498.0	1523.0	gm
Loose unit wt. of Sample	1.350	1.366	1.358	1.144	1.138	1.157	gm/cc
Loose unit wt. of Sample	1350.35	1365.54	1357.94	1143.77	1137.69	1156.68	kg/m^3
Unit weight Average	1357.940			1146.047			kg/m^3

Compaction Unit Weight= 1357.940 kg/m^3

Loose Unit Weight = 1146.047 kg/m^3

SPECIFIC GRAVITY AND WATER ABSORPTION

TP -5

Date: 2011-08-22

	Determination No.	1	2	Unit
A	Wt. of Oven Dry Sample in Air	1482.3	1506.1	gms
B	Wt. of SSD Sample in Air	1495	1525	gms
C_1	Wt. of Sample + Container in Water	1665.9	1680.9	gms
C_2	Wt. of Container in Water	749.6	749.6	gms
$C=(C_1 - C_2)$	Wt. of SSD Sample in Water	916.3	931.3	gms
A/(B - C)	Bulk Specific Gravity (Oven Dry)	2.561	2.537	
B/(B - C)	Bulk Specific Gravity (SSD)	2.583	2.569	
A/(A - C)	Apparent Specific Gravity	2.619	2.620	
(B-A)/Ax100	Absorption %	0.857	1.255	
Average Bulk Specific Gravity (Oven Dry)		2.549		
Average Bulk Specific Gravity (SSD)		2.576		
Average Bulk Specific Gravity (Appa)		2.620		
Average Absorption %		1.056		

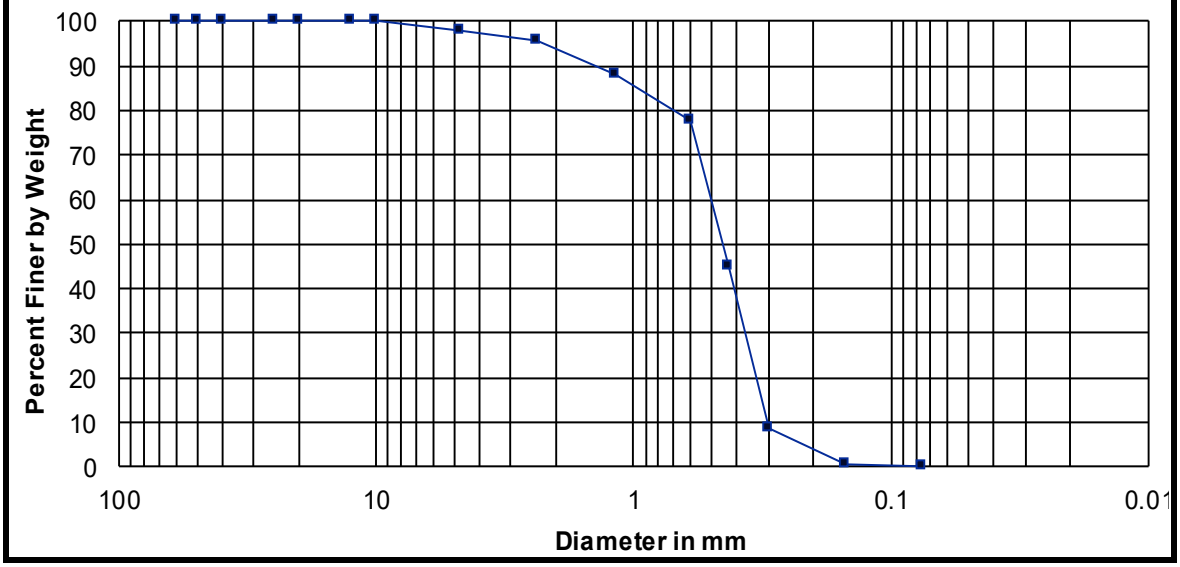
SIEVE ANALYSIS

TP-5

Wt. of Materials : 641 gms **Date:-2011-8-22**

Sieve Size mm	Wt. Retained gm	Wt. Passing gm	% Passing	Remarks
Total weight	641.00			
60	0.00	641.00	100.00	
50	0.00	641.00	100.00	
40	0.00	641.00	100.00	
25	0.00	641.00	100.00	
20	0.00	641.00	100.00	
12.5	0.00	641.00	100.00	Gravel 2.18 %
10	0.00	641.00	100.00	Coarse Sand 52.89 %
4.75	14.00	627.00	97.82	Medium Sand 44.77 %
2.36	15.00	612.00	95.48	Silt & Clay 0.16 %
1.18	48.00	564.00	87.99	
0.6	67.00	497.00	77.54	
0.425	209.00	288.00	44.93	
0.3	231.00	57.00	8.89	
0.15	54.00	3.00	0.47	
0.075	2.00	1.00	0.16	
PAN	1.00			
WASH	0.00			
TOTAL WT.	641.00			

GRAIN SIZE DISTRIBUTION CURVE



SULPHATE SOUNDNESS OF COARSE AGGREGATES

TP-5			Solution used:	Na_2SO_4
Weight of air dry sample before washing, gm (A) =			515	
Weight oven dry sample after washing, gm (B) =			406	
SEIVE TEST BEFORE SOAKING			Date:	2011-08-13
Sieve (mm)	C Wt. Ret.(gms)	D % Ret.= C/Ax100%	Sample required for test if D > 5%	
9.5		0.00		
4.75	100.00	19.42	100 gms	
2.36	101.55	19.72	100 gms	
1.18	104.73	20.34	100 gms	
0.6	100.29	19.47	100 gms	
0.3	102.62	19.93	100 gms	
<0.3		0.00		
Total	509.19	98.87		
SIEVE TEST AFTER SOAKING				
Sieve (mm)	E Sample size (gm)	F Wt.ret (gm)	G % loss = (E-F)/E x 100%	Wt. Loss = (GxD)/100
9.5				
4.75	99.00	97	2.02	0.39
2.36	101.55	99.3	2.22	0.44
1.18	104.73	101.8	2.80	0.57
6	100.29	98.78	1.51	0.29
0.3	102.6	99.26	3.27	0.65
<0.3				
Total	409.19			2.34
Percentage of loss of weight=		2.344 %		

ALKALI REACTIVITY - CHEMICAL METHOD

Sample: Aggregate		Received: 2011-08-23
		Completed:2011-08-28
Sample	**Observed Values,(%)**	
	Reduction in Alkalinity Rc(mmole/lt)	**Reactive Silica Sc(mmole/lt)**
TP5	45.3	19.90

TP-6

GEOLOGICAL LOG OF TEST PIT

PROJECT: GHUNSA KHOLA HEP, TAPLEJUNG, NEPAL
PIT NO.: TP-6
LOCATION: 583912 E, 3049021 N
ELEVATION:
PIT SIZE 2.0 X 2.0 X 1.5 M
EXPECTED BOUNDARY AREA:
THICKNESS (APPROX.):
LOGGED BY: MALIRAM CHAUDHARY

DEPTH (M)	LITHOLOGY: DESCRIPTION	LOG	TEST RESULTS: Optimum moisture %	Max. Dry Density gm/cc	Liquid Limit %	Plastic Limit %	Plasticity Index %	REMARKS
0.0	Mostly clays.							
1.0	Mostly clays with sands and some pebbles and cobbles of gneiss and quartzite.		9.52	2.03	30.80	0.00	30.80	
2.0								
3.0	Mostly clays.							

Photo 1-6 Test Pit TP-6

1.8 LABORATORY TESTS ON TEST PIT TP-6

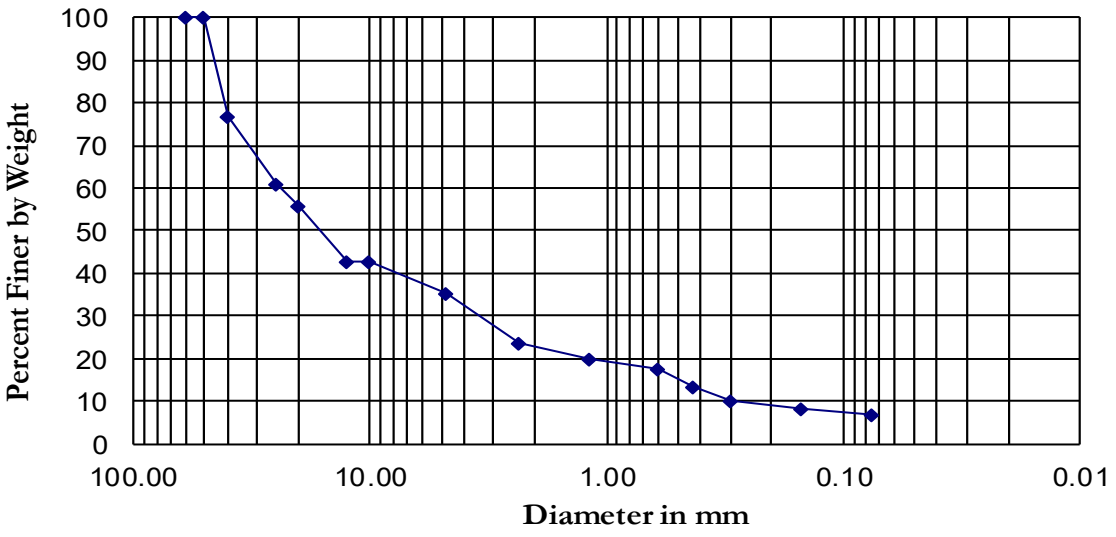
SIEVE ANALYSIS

TP-6

Wt. of Materials 2829 gms **Date:-2011-8-25**

Sieve Size mm	Wt. Retained	Wt. Passing	% Passing	Remarks
Total weight	2829.00			
60	0.00	2829.00	100.00	
50	0.00	2829.00	100.00	
40	660.00	2169.00	76.67	
25	440.00	1729.00	61.12	
20	150.00	1579.00	55.81	
12.5	370.00	1209.00	42.74	Gravel 64.69 %
10	0.00	1209.00	42.74	Corse Sand 22.09 %
4.75	210.00	999.00	35.31	Medium Sand 6.50 %
2.36	325.00	674.00	23.82	Silt & Clay 6.72 %
1.18	110.00	564.00	19.94	
0.6	67.00	497.00	17.57	
0.425	123.00	374.00	13.22	
0.3	85.00	289.00	10.22	
0.15	54.00	235.00	8.31	
0.075	45.00	190.00	6.72	
PAN	190.00			
WASH	0.00			
TOTAL WT.	2829.00			

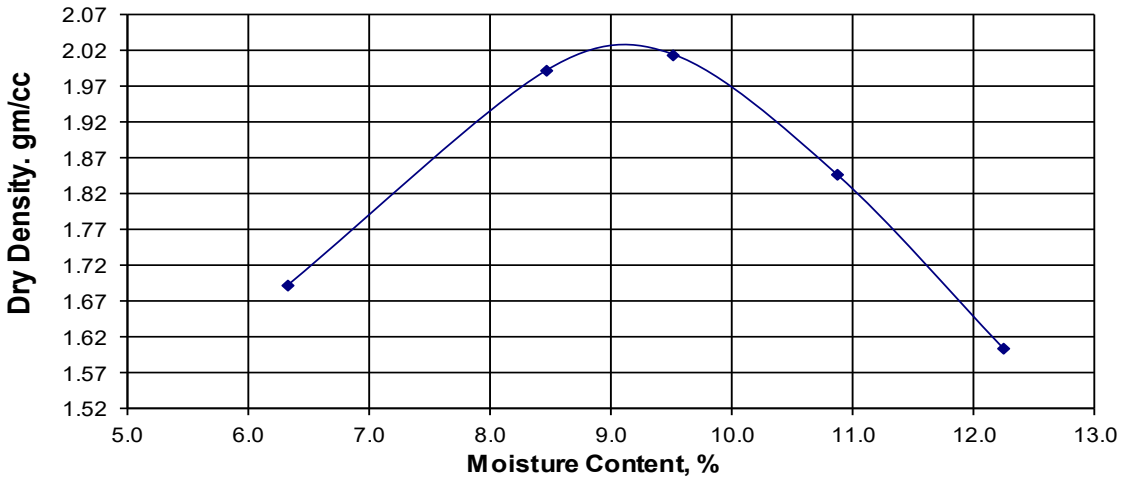
GRAIN SIZE DISTRIBUTION CURVE



MODIFIED PROCTOR COMPACTION TEST

TP-6

No. of layer :	5							
Diameter=	15	cm	No. of blows: 55					
Height=	12.5	cm	Wt. of Hammer : 4.9kg.			Volume :	2209.78 cc	
						Date:	2011-08-22	
Water Content Determination								
Wt. of wet soil+can			290.00	258.00	244.00	309.00	274.00	
Wt. of dry soil+can			277.00	248.00	232.00	285.00	253.80	
Wt. of can			71.80	130.00	106.00	64.30	89.00	
Wt. of dry soil			205.20	118.00	126.00	220.70	164.80	
Wt. of moisture			13.00	10.00	12.00	24.00	20.20	
Water Content,W%			6.34	8.47	9.52	10.87	12.26	
Density Determination								
Water Content,W%			6.34	8.47	9.52	10.87	12.26	
Wt. of Soil +mould			11800	12600	12700	12350	11800	
Wt. of mould			7825	7825	7825	7825	7825	
Wt. of Soil in mould,g			3975	4775	4875	4525	3975	
Wet Density, gm/cm3			1.80	2.16	2.21	2.05	1.80	
Dry Density, gm/cm3			1.692	1.992	2.014	1.847	1.602	



Optium Moisture :9.52% Maximum Dry Density 2.030gm/cc

ATTERBERG'S LIMIT TEST

TP-6						
				Date: 2011-08-25		
	Liquid Limit				Plastic Limit	
Determination No.						
Container No.						
No. of blows	14	21	29	38		
Wt. of wet soil+container (A), gms	28.00	27.00	24.80	25.60		
Wt. of dry soil+container (B), gms	23.00	22.50	20.90	21.70	Non Pl	
Wt. of can (C), gms	7.40	8.30	7.90	8.30		
Wt. of dry soil, w_d = (B-C) gms	15.60	14.20	13.00	13.40		
Wt. of moisture, w_w = (A-B) gms	5.00	4.50	3.90	3.90		
Water content, % =w_w / w_d*100%	32.05	31.69	30.00	29.10		
Plastic Limit, %						

Result :-			Plastic limit :	30.6%
Liquid limit:	30.6 %		Plasticity Index:	0.00

TP-7

GEOLOGICAL LOG OF TEST PIT

PROJECT: GHUNSA KHOLA HEP, TAPLEJUNG, NEPAL
PIT NO.: TP-7
LOCATION: 583668 E, 3048823 N
ELEVATION:
PIT SIZE 2.0 X 2.0 X 1.5 M
EXPECTED BOUNDARY AREA:
THICKNESS (APPROX.):
LOGGED BY: MALIRAM CHAUDHARY

DEPTH (M)	LITHOLOGY: DESCRIPTION	LOG	TEST RESULTS: Optimum moisture %	Max. Dry Density gm/cc	Liquid Limit %	Plastic Limit %	Plasticity Index %	REMARKS
0.0	Dominantly black clays.							
1.0	Mostly black clays with pebbles, cobbles and sands.		9.77	2.109	26.00	0.00	26.00	
2.0	Black clays.							
3.0								

Photo 1-7: Test Pit TP-8

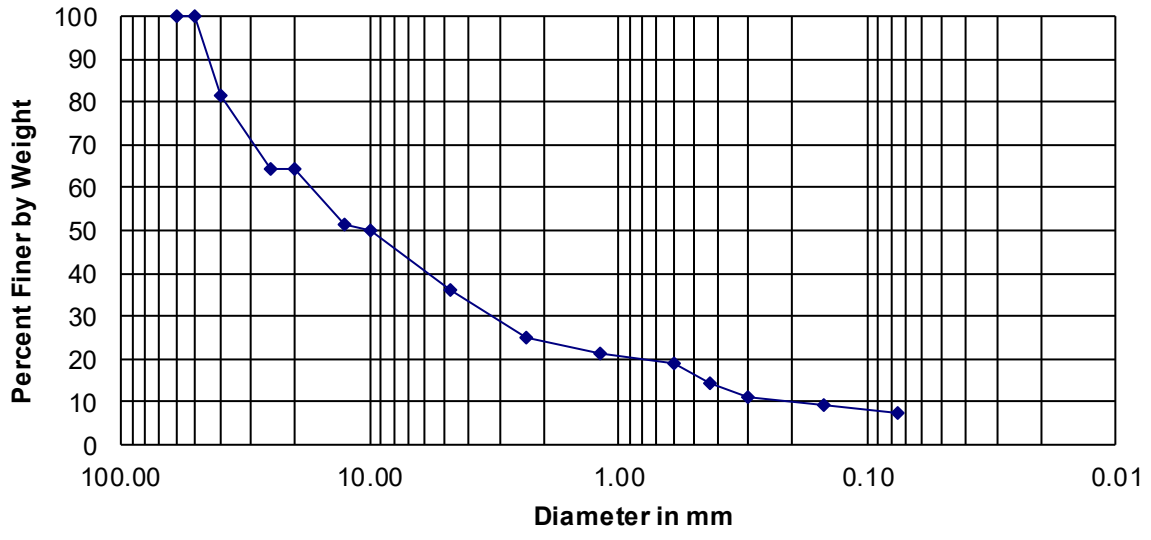
1.9 LABORATORY TESTS ON TEST PIT TP-7

SIEVE ANALYSIS

TP-7

Wt. of Materials : 2867 gms **Date:-2011-8-25**

Sieve Size mm	Wt. Retained gm	Wt. Passing gm	% Passing	Remarks		
Total weight	2867.00					
60	0.00	2867.00	100.00			
50	0.00	2867.00	100.00			
40	530.00	2337.00	81.51			
25	490.00	1847.00	64.42			
20	0.00	1847.00	64.42			
12.5	370.00	1477.00	51.52	Graval	64.14	%
10	44.00	1433.00	49.98	Corse Sand	21.38	%
4.75	405.00	1028.00	35.86	Medium Sand	7.15	%
2.36	308.00	720.00	25.11	Silt & Clay	7.32	%
1.18	105.00	615.00	21.45			
0.6	72.00	543.00	18.94			
0.425	128.00	415.00	14.48			
0.3	99.00	316.00	11.02			
0.15	56.00	260.00	9.07			
0.075	50.00	210.00	7.32			
PAN	210.00					
WASH	0.00					
TOTAL WT.	2867.00					



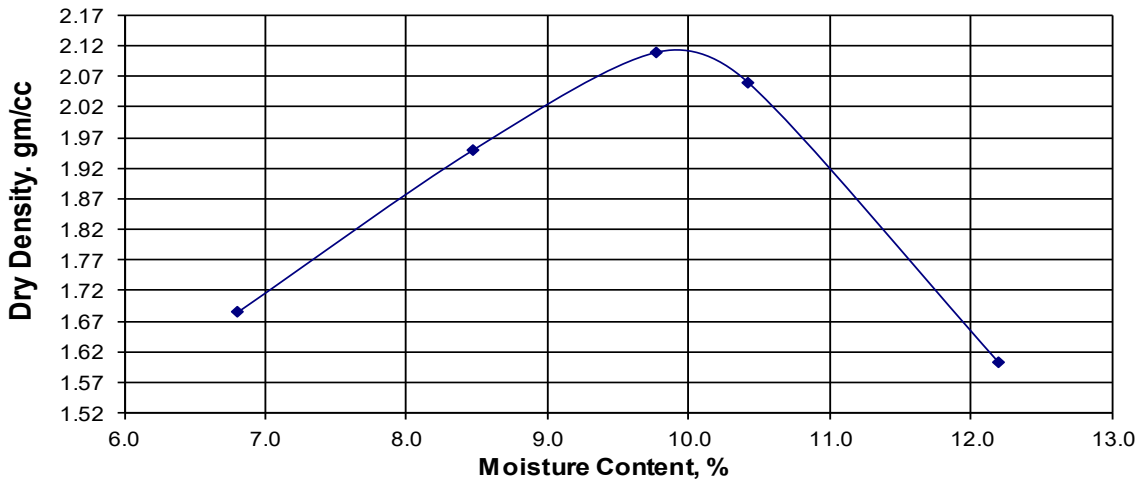
MODIFIED PROCTOR COMPACTION TEST

TP-7

No. of layer :	5						
Diameter=	15	cm	No. of blows: 55				
Height=	12.5	cm	Wt. of Hammer : 4.9kg.			Volume :	2209.78 cc
							Date: 2011-08-22

Water Content Determination						
Wt. of wet soil+can	290.00	257.00	245.50	308.00	274.00	
Wt. of dry soil+can	276.00	247.00	233.00	285.00	254.00	
Wt. of can	70.00	129.00	105.00	64.30	90.00	
Wt. of dry soil	206.00	118.00	128.00	220.70	164.00	
Wt. of moisture	14.00	10.00	12.50	23.00	20.00	
Water Content,W%	6.80	8.47	9.77	10.42	12.20	

Density Determination						
Water Content,W%	6.80	8.47	9.77	10.42	12.20	
Wt. of Soil +mould	11800	12500	12940	12850	11800	
Wt. of mould	7825	7825	7825	7825	7825	
Wt. of Soil in mould.g	3975	4675	5115	5025	3975	
Wet Density, gm/cm3	1.80	2.12	2.31	2.27	1.80	
Dry Density, gm/cm3	1.684	1.950	2.109	2.059	1.603	



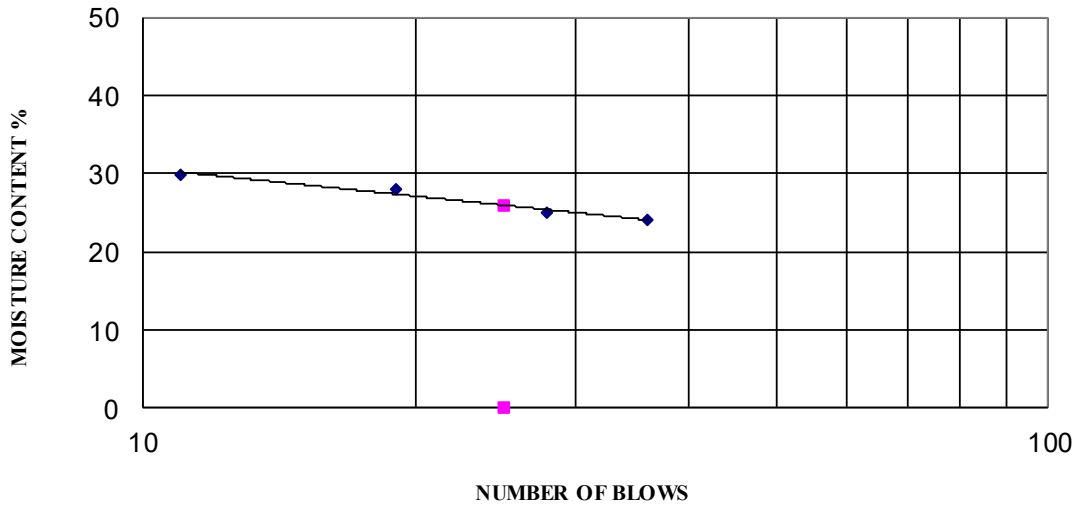
Optimum Moisture : 9.77% Maximum Dry Density 2.109 gm/cc

ATTERBERG'S LIMIT TEST

TP-7

Date: 2011-08-25

	Liquid Limit				Plastic Limit	
Determination No.	1	2	3	4	1	2
Container No.						
No. of blows	11	19	28	36		
Wt. of wet soil+container (A), gms	27.40	26.30	24.80	25.60		
Wt. of dry soil+container (B), gms	22.70	22.40	21.40	22.20	Non PL	
Wt. of can (C), gms	7.00	8.50	7.80	8.10		
Wt. of dry soil, w_d = (B-C) gms	15.70	13.90	13.60	14.10		
Wt. of moisture, w_w = (A-B) gms	4.70	3.90	3.40	3.40		
Water content, % =w_w / w_d*100%	29.94	28.06	25.00	24.11		
Plastic Limit, %						



Result :-		Plastic limit :	26.00%
Liquid limit:	26 %	Plasticity Index:	0.00

TP-8

GEOLOGICAL LOG OF TEST PIT

PROJECT: GHUNSA KHOLA HEP, TAPLEJUNG, NEPAL
PIT NO.: TP-8
LOCATION: 583467 E, 3048636 N
ELEVATION:
PIT SIZE 2.0 X 2.0 X 3.0 M
EXPECTED BOUNDARY AREA:
THICKNESS (APPROX.):
LOGGED BY: MALIRAM CHAUDHARY

DEPTH (M)	LITHOLOGY: DESCRIPTION	LOG	TEST RESULTS: Optimum moisture %	Max. Dry Density gm/cc	Liquid Limit %	Plastic Limit %	Plasticity Index %	REMARKS
0.0	Coarse grained sand with fine gravels and pebbles.							
1.0	Mostly cobbles and boulders of quartzite, amphibolite and gneiss.		8.50	2.12	27.50	0.00	27.50	
2.0								
3.0								

Photo 1-8: Test Pit TP-8

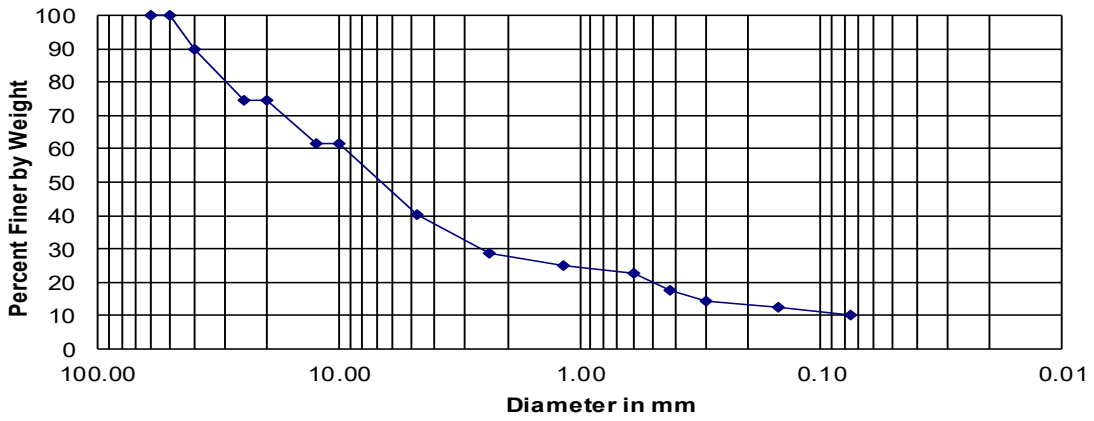
1.10 LABORATORY TESTS ON TEST PIT TP-8

SIEVE ANALYSIS

TP-8

Wt. of Materials : 2986 gms **Date:-2011-8-25**

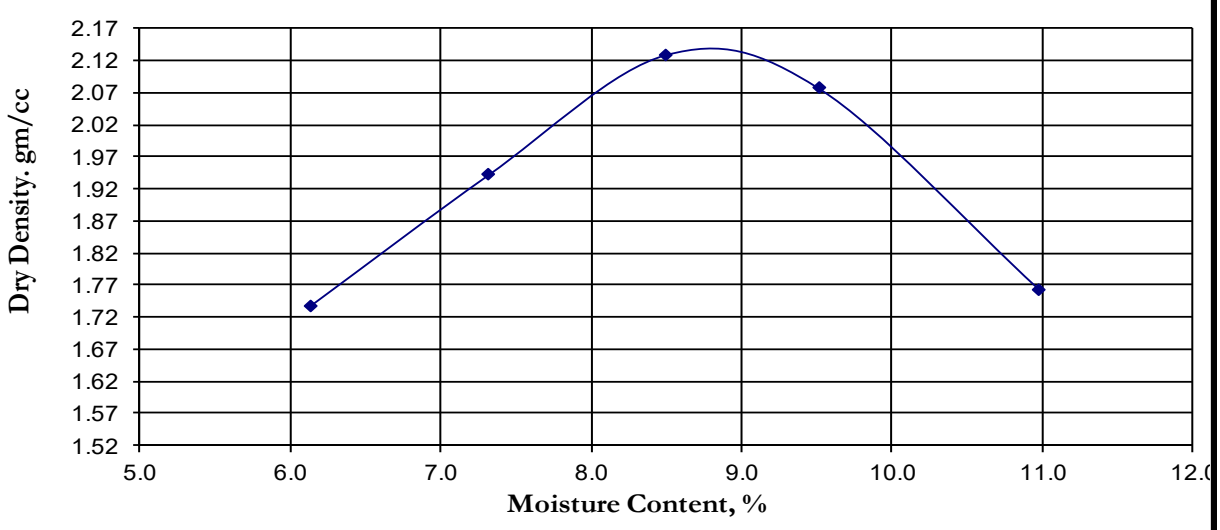
Sieve Size mm	Wt. Retained	Wt. Passing gm	% Passing	Remarks		
Total weight	2986.00					
60	0.00	2986.00	100.00			
50	0.00	2986.00	100.00			
40	310.00	2676.00	89.62			
25	445.00	2231.00	74.72			
20	0.00	2231.00	74.72			
12.5	394.00	1837.00	61.52	Graval	59.75	%
10	0.00	1837.00	61.52	Corse Sand	22.47	%
4.75	635.00	1202.00	40.25	Medium Sand	7.74	%
2.36	347.00	855.00	28.63	Silt & Clay	10.05	%
1.18	111.00	744.00	24.92			
0.6	71.00	673.00	22.54			
0.425	142.00	531.00	17.78			
0.3	108.00	423.00	14.17			
0.15	57.00	366.00	12.26			
0.075	66.00	300.00	10.05			
PAN	300.00					
WASH	0.00					
TOTAL WT.	2986.00					



MODIFIED PROCTOR COMPACTION TEST

TP-8

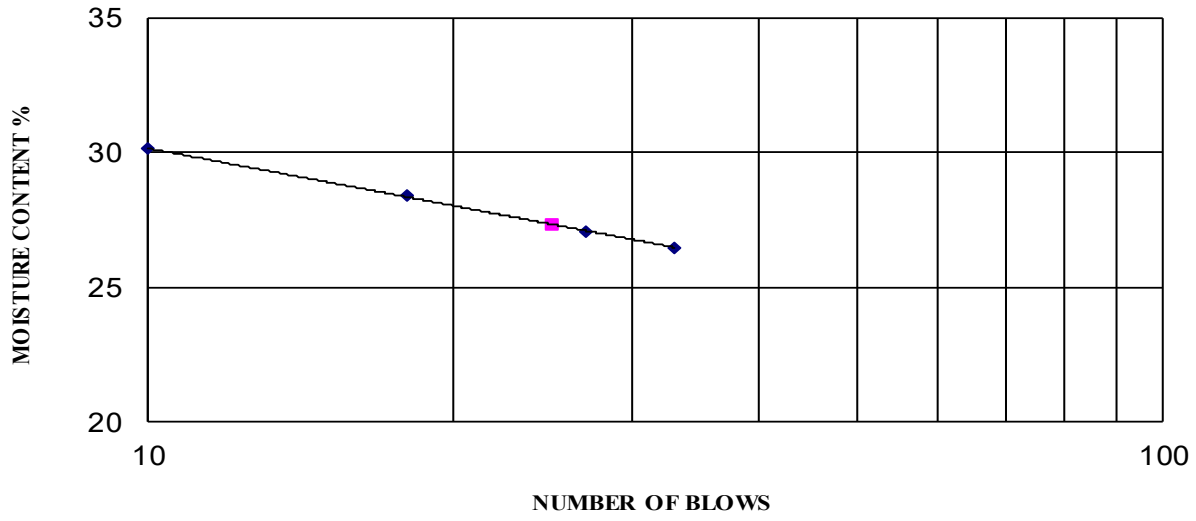
							Source : Taplejung	
No. of layer :	5							
Diameter=	15	cm	No. of blows: 55					
Height=	12.5	cm	Wt. of Hammer : 4.9kg.			Volume :	2209.78 cc	
Sample : **BP-8**							Date:	2011-08-22
Water Content Determination								
Wt. of wet soil+can			297.00	261.00	244.00	306.00	272.00	
Wt. of dry soil+can			284.00	252.00	233.50	285.00	254.00	
Wt. of can			72.00	129.00	110.00	64.30	90.00	
Wt. of dry soil			212.00	123.00	123.50	220.70	164.00	
Wt. of moisture			13.00	9.00	10.50	21.00	18.00	
Water Content,W%			6.13	7.32	8.50	9.52	10.98	
Density Determination								
Water Content,W%			6.13	7.32	8.50	9.52	10.98	
Wt. of Soil +mould			11900	12430	12930	12850	12150	
Wt. of mould			7825	7825	7825	7825	7825	
Wt. of Soil in mould.g			4075	4605	5105	5025	4325	
Wet Density, gm/cm3			1.84	2.08	2.31	2.27	1.96	
Dry Density, gm/cm3			1.738	1.942	2.129	2.076	1.764	



Optium Moisture :8.50% Maximum Dry Density 2.120 gm/cc

ATTERBERG'S LIMIT TEST

TP-8						
				Date:	2011-08-25	
	Liquid Limit				Plastic Limit	
Determination No.	1	2	3	4	1	2
Container No.						
No. of blows	10	18	27	33		
Wt. of wet soil+container (A), gms	27.40	26.30	24.80	25.60		
Wt. of dry soil+container (B), gms	22.70	22.30	21.20	22.00	Non Pl	
Wt. of can (C), gms	7.10	8.20	7.90	8.40		
Wt. of dry soil, w_d = (B-C) gms	15.60	14.10	13.30	13.60		
Wt. of moisture, w_w = (A-B) gms	4.70	4.00	3.60	3.60		
Water content, % =w_w / w_d*100%	30.13	28.37	27.07	26.47		
Plastic Limit, %						



Result :-				Plastic limit :	27.3%
Liquid limit:	27.3	%		Plasticity Index:	0.00

NTP-1

GEOLOGICAL LOG OF TEST PIT

PROJECT: GHUNSA KHOLA HEP, TAPLEJUNG, NEPAL
PIT NO.: NTP-1
LOCATION: 579836 E, 3046433 N
ELEVATION:

PIT SIZE 2.0 X 2.0 X 2.0 M
EXPECTED BOUNDARY AREA:
THICKNESS (APPROX.):
LOGGED BY: MALIRAM CHAUDHARY

DEPTH (M)	LITHOLOGY: DESCRIPTION	LOG	TEST RESULTS: LAA %	FI %	EI %	Sp. Gr.	WA %	ACV %	AIV %	SS %	R_c mmole/ lt.	S_c mmole/ lt.	REMARKS
0.0	Pebbles, cobbles with black soil and sand mix and organic materials.												Expected area mostly contains sand but the test pit consists of 60% pebbles and cobbles and 40% sand.
0.5	Brown coarse grained sand with pebbles, cobbles.		42.78	17.82	54.65	2.60 (C) 2.60 (F)	1.6	29.225	13.75	0.114 (C)	34 (C) 49 (F)	7.6 (C) 4.9 (F)	C= Coarse Aggregates, F= Fine Aggregates, R_c= Reduction in Alkanity S_c= Reactive Silica
1.0	Light brown fine sand.												
1.5	Grey fine sand.												

Photo 1-9: Test Pit NTP-1

1.11 LABORATORY TESTS ON TEST PIT NTP-1

UNIT WEIGHT

NTP-1							
					Date:	06-12-2019	
Measurement of Mould							
Length of Mould =	0.302	m					
Breadth of Mould =	0.302	m					
Height of Mould =	0.064	m					
Date :-							
Descriptions:-							
Sample No:		1	2	3	4	5	6
Sample weight :	Kg	11.724	11.739	11.742	11.659	11.631	11.65
Volume of Mould:	m^3	0.00584	0.00584	0.00584	0.00584	0.00584	0.00584
Density of Soil ϱd	(Kg/m^3)	2008.547	2011.117	2011.631	1997.411	1992.614	1995.869
unit weight $\gamma_d = \varrho_d * g$	KN/m^3	20.085	20.111	20.116	19.974	19.926	19.959
Avg unit wt	KN/m^3	20.029					

SPECIFIC GRAVITY AND WATER ABSORPTION

NTP-1				
			Date: 2019/12/08	
S.N.	**Description**		**No. of Test**	
			Test-1	**Test-2**
1	Location		583408E/3048204N	
2	Wt of saturated aggregate+basket in water	(W1), grms	1876	1870
3	Wt. of basket in water	(W2), grms	1563	1558
4	Wt. of oven dry aggregate	(W3), grms	500	500
5	Wt. of saturated surface dry aggregate	(W4), grms	508	508
6	Specific gravity		2.61	2.59
7	Water absorption (%)		1.6	1.6
8	Average specific gravity		2.60	
9	Average water absorption		1.6	
	Water Absorption(%)= (Weight of water absorbed/ Weight of dry sample)*100			
	Specific gravity=$W_4/(W_4-(W_1-W_2)$			

SPECIFIC GRAVITY (PYCNOMETER METHOD)

NTP-1			
Specific Gravity			
Sampling Numbers		1	2
Weight of Pycnometer + water + soil	(W1)grm	181.83	174.43
Weight of Pycnometer + water	(W2)grm	163.10	162.32
Dry Weight of Sample,	(Ws)grm	30.00	20.00
Temperature of Water, T deg C.		27	27
Specific Gravity of Water, at T deg C.	(Gt)	0.9987	0.9987
Specific Gravity of Soil Sample,	(Gs)	2.66	2.53
Average Sp. Gravity		2.60	
Gs = (Gt* Ws)/(Ws-W1-W2)			

LOS ANGELES ABRASION TEST

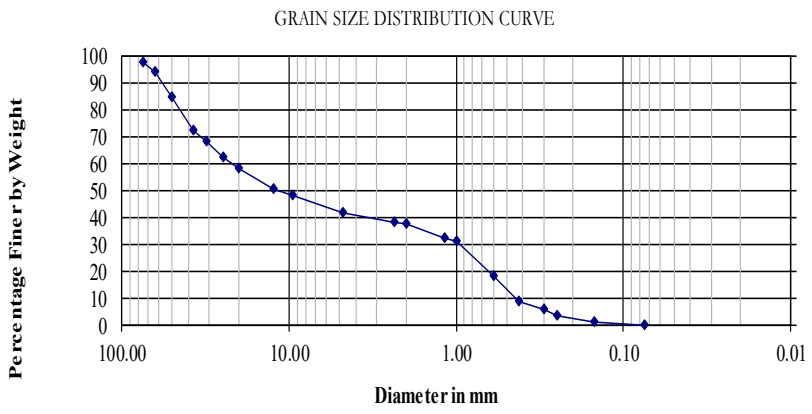
NTP-1					
S.N.	Particulars			Test No. 1	
1	Weight of specimen		W1, grm	5000	5000
2	Weight of specimen after test coarser then 1.7mm		W2, grm	2858	2864
3	Percentage wear =	(W1-W2)*100 / W1		42.84	42.72
4	Average (%)			42.78	
1	Type of aggregate: Construction Material				
2	Standard: ISTM				
3	Number of spheres used: 12				
4	Grading: A				

SIEVE ANALYSIS

NTP-1

Wt. of Sample (gms): 32338.00 Date:- 2019-12-2

Seive Size mm	Wt. of Soil Retained gms	Cumulative Weight Retained gms	Cumulative Percentage Retained %	Percent Passing %	Remarks
75.000	772.00	772.00	2.39	97.61	
63.000	1108.00	1880.00	5.81	94.19	
50.000	3052.00	4932.00	15.25	84.75	
37.500	3986.00	8918.00	27.58	72.42	
31.500	1448.00	10366.00	32.06	67.94	
25.000	1898.00	12264.00	37.92	62.08	Gravel 58.11 %
20.000	1256.00	13520.00	41.81	58.19	Coarse Sand 33.29 %
12.500	2536.00	16056.00	49.65	50.35	Medium Sand 8.50 %
9.500	742.00	16798.00	51.95	48.05	Silt & Clay 0.11 %
4.750	1992.00	18790.00	58.11	41.89	
2.360	1196.00	19986.00	61.80	38.20	
2.000	164.00	20150.00	62.31	37.69	
1.180	1726.00	21876.00	67.65	32.35	
1.000	378.00	22254.00	68.82	31.18	
0.600	4134.00	26388.00	81.60	18.40	
0.425	3166.00	29554.00	91.39	8.61	
0.300	890.00	30444.00	94.14	5.86	
0.250	860.00	31304.00	96.80	3.20	
0.150	738.00	32042.00	99.08	0.92	
0.075	262.00	32304.00	99.89	0.11	
Pan	34.00	32338.00	100.00	0.00	

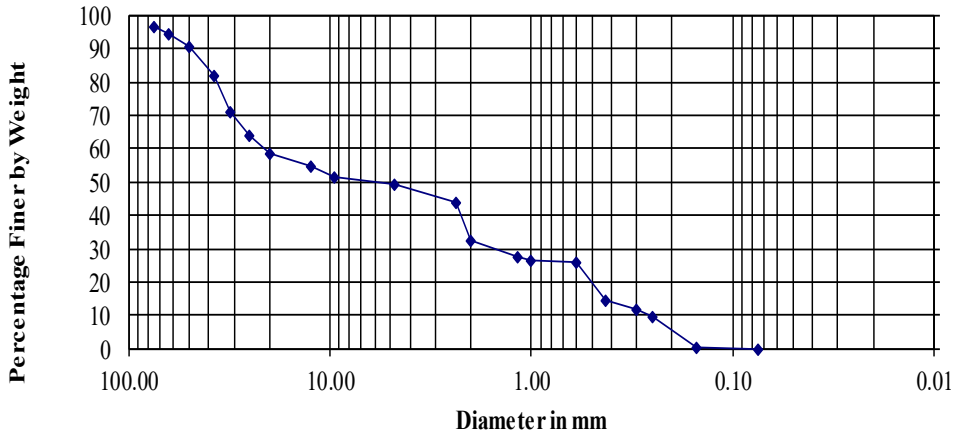


NTP-1

Wt. of Sample (gms): 35700.000 Date:- 2019-12-3

Seive Size mm	Wt. of Soil Retained gms	Cumulative Weight Retained gms	Cumulative Percentage Retained %	Percent Passing %	Remarks
75.000	1108.00	1108.00	3.10	96.90	
63.000	772.00	1880.00	5.27	94.73	
50.000	1448.00	3328.00	9.32	90.68	
37.500	3052.00	6380.00	17.87	82.13	
31.500	3986.00	10366.00	29.04	70.96	
25.000	2536.00	12902.00	36.14	63.86	
20.000	1894.00	14796.00	41.45	58.55	
12.500	1256.00	16052.00	44.96	55.04	Gravel 50.39 %
9.500	1196.00	17248.00	48.31	51.69	Coarse Sand 35.09 %
4.750	742.00	17990.00	50.39	49.61	Medium Sand 14.43 %
2.360	1992.00	19982.00	55.97	44.03	Silt & Claty 0.08 %
2.000	4134.00	24116.00	67.55	32.45	
1.180	1726.00	25842.00	72.39	27.61	
1.000	378.00	26220.00	73.45	26.55	
0.600	164.00	26384.00	73.90	26.10	
0.425	4134.00	30518.00	85.48	14.52	
0.300	860.00	31378.00	87.89	12.11	
0.250	890.00	32268.00	90.39	9.61	
0.150	3166.00	35434.00	99.25	0.75	
0.075	236.00	35670.00	99.92	0.08	
Pan	30.00	35700.00	100.00	0.00	

GRAIN SIZE DISTRIBUTION CURVE



ALKALI REACTIVITY - CHEMICAL METHOD

Sample: Coarse and Fine Aggregate		Received: 22/07/2076
		Completed: 01/08/2076
Particulars	**Observed value, (%)**	
	Reduction in Alkalinity (mmoles/lt)	**Reactive Silica (mmoles/lt)**
NTP-1 (C. Agg)	34	7.6
NTP-1 (F. Agg)	49	49

SULPHATE SOUNDNESS OF COARSE AGGREGATE

Project:		**Ghunsa Khola HEP**				**Date:-2076/09/22**	
Location:		**Taplejung**					
Sample:		**Pit No-1**					
Sieve (mm)	**Wt before test (gm)**	**Wt after soaking & Dry**	**Wt after soaking & Dry**	**Wt after soaking & Dry**	**Wt after soaking & Dry**	**Wt after soaking & Dry**	**Wt after soaking & Dry**
		1	**2**	**3**	**4**	**5**	**6**
25	1001.16	1001.02	1000.88	1000.68	1000.48	1000.37	1000.11
20	502.19	502.08	501.98	501.87	501.71	501.64	501.58
16	503.35	503.23	503.17	503.07	502.96	502.84	502.75
12.5	501.29	501.18	501.08	500.99	500.90	500.78	500.69
Total	**2507.99**						

Sieve (mm)	**Loss in Weight (gm)**	**% loss in Weight**
25	1.05	0.104878341
20	0.61	0.12146797
16	0.60	0.119201351
12.5	0.60	0.119691197
Total	**2.86**	

Percentage of loss of weight =	**0.114**	**%**

AGGREGATE CRUSHING VALUE

NTP-1		Date:2019-12-08			
S.N.	**Description**	**Determition no.**			
		1	**2**	**3**	**4**
1	Weight of sample, (passing 25-20, mm seive), grm	3000	3000	3000	3000
2	Weight retained on (4.75 mm seive), grm	2140	2147	2099	2107
3	Weight passing on ((4.75 mm seive), grm	860	853	901	893
4	Aggregate crushing value,(%)	28.67	28.43	30.03	29.77
5	Average aggregate crushing value,(%)	28.55		29.90	

Aggregate Crushing value= $\frac{\text{Wt. of portion of crushed material passing 4.75mm, seive}}{\text{Total weight of sample}}$

AGGREGATE IMPACT VALUE

NTP-1		Date:2019-12-08			
S.N.	Description	Determion no.			
		1	2	3	4
1	Weight of sample, (passing 12-10, mm seive), grm	500	500	500	500
2	Weight retained on (2.36, mm seive), grm	434	437	428	426
3	Weight passing on ((2.36, mm seive), grm	66	63	72	74
4	Aggregate impact value,(%)	13.20	12.60	14.40	14.80
5	Average aggregate impact value,(%)	12.90		14.60	

Aggregate Impact value= $\frac{\text{Wt. of portion of crushed material passing 2.36mm, seive}}{\text{weight of sample}}$

FLAKINESS AND ELONGATION TEST

NTP-1

Flakiness Index

Passing mm	Retain mm	Wt. of Aggregate in each fraction W1,grm	Passing gms	Flakiness Index
63	50	3668	816	22.25
50	40	5776	1256	21.75
40	31	3190	802	25.14
31	25	3182	384	12.07
25	20	2992	312	10.43
20	16	1668	213	12.77
16	12.5	1008	96	9.52
12.5	10	836	82	9.81
10	6.3	528	98	18.56
Total Weight=		22848	4059	17.77

Flakiness index % =	17.77

Elongation Index:

Passing mm	Retain mm	Wt. of Aggregate in each fraction W1,grm	Retain gms	Elongation Index
63	50	3668	1818	49.56
50	40	5776	2480	42.94
40	31	3190	594	18.62
31	25	3182	2240	70.40
25	20	2992	2092	69.92
20	16	1668	1368	82.01
16	12.5	1008	868	86.11
12.5	10	836	686	82.06
10	6.3	528	294	55.68
Total Weight=		22848	12440	54.45

Elongation index % =	54.45

NTP-1

Flakiness Index

Passing mm	Retain mm	Wt. of Aggregate in each fraction W1,grm	Passing gms	Flakiness Index
63	50	1008	96	9.52
50	40	1608	216	13.43
40	31	2992	322	10.76
31	25	3182	384	12.07
25	20	3190	802	25.14
20	16	5776	1256	21.75
16	12.5	3668	816	22.25
12.5	10	528	82	15.53
10	6.3	836	98	11.72
Total Weight=		22788	4072	17.87

Flakiness index % =	17.87

Elongation Index:

Passing mm	Retain mm	Wt. of Aggregate in each fraction W1,grm	Retain gms	Elongation Index
63	50	1008	868	86.11
50	40	1608	1368	85.07
40	31	2992	2092	69.92
31	25	3182	2240	70.40
25	20	3190	594	18.62
20	16	5776	2480	42.94
16	12.5	3668	1878	51.20
12.5	10	528	294	55.68
10	6.3	836	686	82.06
Total Weight=		22788	12500	54.85

Elongation index % =	54.85

Annex 8: Petrographic Studies

1. PETROGRAPHIC STUDY

The samples were examined in thin section by the polarizing petrographic microscope for texture, structure and fabric, using technique of optical mineralogy. Microphotographs of the thin sections were acquired through the motif 2.0 camera in both plane polarized light (PPL) and under cross nicols (UCN) for identification of the minerals and structures.

The results of the petrographic analyses are presented in the following. The photomicrographs of the samples under PPL and UCN at scale 1 cm = 56 nm, magnification 4×10 are shown in Photo 1-1 to Photo 1-13

1.1 Results

Sample No: 1, NDH-1

Sample Type: Core Sample

Sample depth: 12-14m

Megascopic Study:

The rock sample is clearly well foliated Augen Gneiss. The augen are due to the presence of Quartz and Plagioclase feldspar. Platy minerals consists of dominantly biotite and muscovite. Recrystallized grains of quartz and feldspar are distinct.

Microscopic Study:

Color:

Almost colorless except some pleoichroism shown by biotite under PPl. The euhedral boundaries of feldspar are distinct due to differential relief with surrounding minerals. Under cross nicol UCN a bright field of vision are observed. Where white to milky white grain fairly alternate with multicolored micaceous minerals with distinct biotite at ream.

Texture

- Grain size: Inequigranular distribution of minerals. Plagioclase feldspar is about 1 cm (Augen case), 1.5 to 1 cm in average; biotite up to 0.5 cm; muscovite 0.25 to 0.5 cm; quartz 0.1 to 0.5 cm. there is excessive amount of crystals of garnet with maximum diameter of 0.5 cm.
- Mutual relationship: Phanaritic, inequigranular, layered.

Structure:

- No of foliation: Two sets of foliation intersecting in acute angle. Foliation is well defined by the presence of preferred orientation of platy minerals, the foliation plane is itself folded.
- Deformation structures: Micaceous minerals are highly deformed wrapping the competent feldspar crystals and deformation lamellae are present within the quartz grain. Both the indented and sutured contact are developed in the feldspar and quartz.
- Microfractures: micro fractures are observed.
- Others: No remarks

Mineralogy:

- Mineral Constituents:

Table 1-1: Mineral Constituents of Sample No. 1

Plagioclase feldspar %	Orthoclase %	Quartz %	Platy minerals (Muscovite and biotite)%	Garnet %	Opaque and unidentified%
20	10	20	30	10	10

- Diagnostic Properties:

Plagioclase feldspar shows polysynthetic twinning with subhedral shape; orthoclase is identified by Carlsbad twinning and alteration to quartz and sericite/chlorite; quartz is distinct by anhedral shape and wavy extinction; biotite is highly pleochoric with 1 set of basal cleavage and parallel extinction while muscovite is identified by its multicolored elongated grain under cross nicols; garnet is identified translucent or slightly opaque, leading to variable lusters with high relief in thin

section and is often more lightly colored than spinel. Opaque are identified by its dark appearance in PPL. *Petrological classifications:* It is well foliated rock.

Identification: **Garnet-Mica Rich Augen Gneiss**

Index mineral/ grade of metamorphism: Garnet grade

Engineering Evaluation:

- Microcracks: present
- Weathering: fresh

Discussions:
The rock belongs to hard class due to significant percentage of quartz and feldspar. The presence of lepidoblastic mineral are also significant. In unweather condition rocks may possess high strength and load perpendicular to gneissic structure is recommended where is possible.

Conclusions:

The rock is suitable for foundations as well as other types of engineering construction however it is not suitable for crushed construction aggregates like sand and gravel.

Remarks: Ortho-gneiss

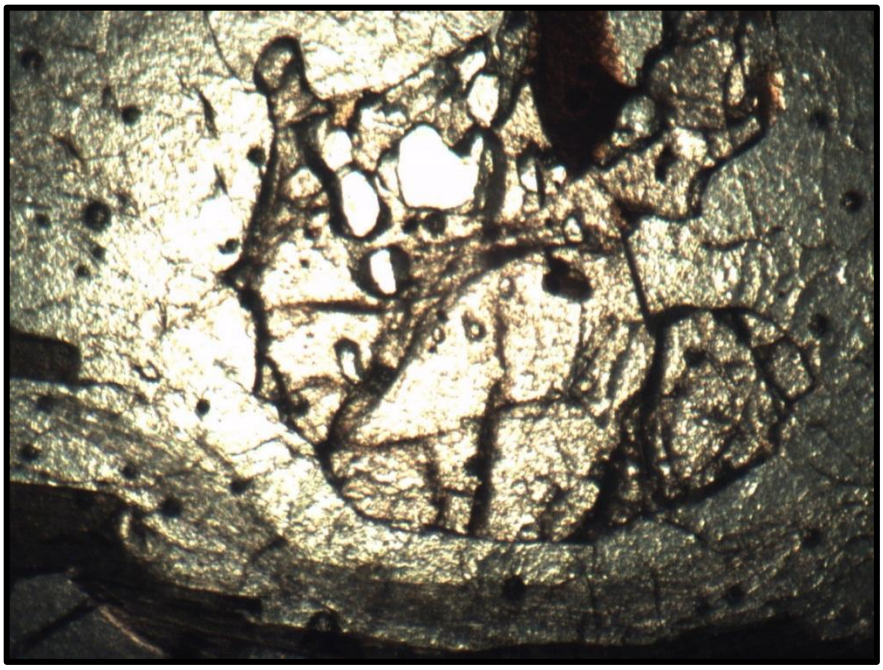


Photo 1-1: Photomicrograph of Augen Gneiss from NDH-1 at 12 – 14 m depth (PPL).

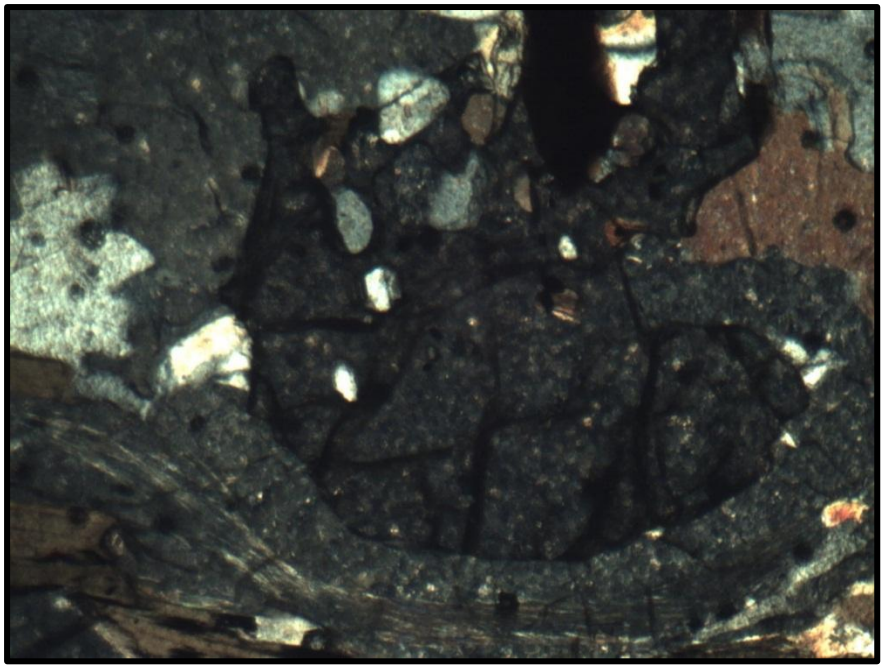


Photo 1-2: Photomicrograph of Augen Gneiss from NDH-1 at 12 – 14 m depth (UCN).

Sample No: 2, NDH-1

Sample Type: Core Sample

Sample depth: 26-28m

Megascopic Study:

The rock sample is clearly well foliated para-gneiss. Quartz and feldspar are major psammatice minerals present. Platy minerals consists of dominantly biotite and muscovite. Recrystallized grains of quartz and feldspar are distinct. There is excessive amount of crystals of reddish garnet and metallic mineralization is also present in the crystalline nature. Mostly the ores of lead, zinc and poly-metallic association is remarkable by its metallic luster and high specific gravity in general.

Microscopic Study:

Color:

Almost colorless except some pleoichroism shown by biotite under PPl. The euhedral boundaries of feldspar are distinct due to differential relief with surrounding minerals. Under cross nicol UCN a bright field of vision are observed. Where white to milky white grain fairly alternate with multicolored micaceous minerals with distinct biotite at ream. The dark field of vision in both PPL and UCN is due to presence of metallic minerals.

Texture

- Grain size: Inequigranular distribution of minerals. Plagioclase feldspar is about 4.5 to 1.0 cm in average; biotite upto 0.5 cm; muscovite 0.25 to 0.5 cm; quartz 0.1 to 0.5 cm. there is excessive amount of crystals of garnet with maximum diameter of 0.7 cm.
- Mutual relationship: Phanaritic, inequigranular, layered.

Structure:

- No of foliation: Two sets of foliation intersecting in acute angle. Foliation is well defined by the presence of preferred orientation of platy minerals the foliation plane is itself folded the feldspar itself represent foliation by its semi-elongation.
- Deformation structures: Micaceous minerals are highly deformed wrapping the competent feldspar crystals and deformation lamellae are present within the quartz grain. Both the indented and sutured contact is developed in the feldspar and quartz. Quartz crystals have shown the pressure shadow structures abundantly.
- Micro-fractures: absent

- Others: Excessive amount of garnets are observed.

Mineralogy:

- Mineral Constituents:

Table 1-2: Mineral Constituents of Sample No. 2

Plagioclase feldspar %	Orthoclase %	Quartz %	Platy minerals (Muscovite and biotite) %	Garnet %	Opaque and unidentified %
30	20	15	15	15	5

Diagnostic Properties:

Plagioclase feldspar shows polysynthetic twinning with subhedral shape; orthoclase is identified by Carlsbad twinning and alteration to quartz and sericite and other inclusion of opaques; quartz is distinct by anhedral shape and wavy extinction; biotite is highly pleochoric with 1 set of basal cleavage and parallel extinction while muscovite is identified by its multicolored elongated grain under cross nicols; garnet is identified translucent or slightly opaque, leading to variable lusters with high relief in thin section and is often more lightly colored than spinel. Metallic minerals are identified by its dark appearance both in UCN and PPL.

Petrological classifications: It is well foliated rock.

Identification: **Garnet rich Para-gneiss**

Index mineral/ grade of metamorphism: Garnet grade

Engineering Evaluation:

- Microcracks: absent
- Weathering: fresh

Discussions:
The rock belongs to hard class due to significant percentage of quartz and feldspar. The presence of lepidoblastic mineral are also significant. In non-weathered condition rocks may possess high strength and load perpendicular to gneissic structure is recommended where is possible.

Conclusions:

The rock is suitable for foundations as well as other types of engineering construction and coarse aggregates.

Remarks: Paragneiss

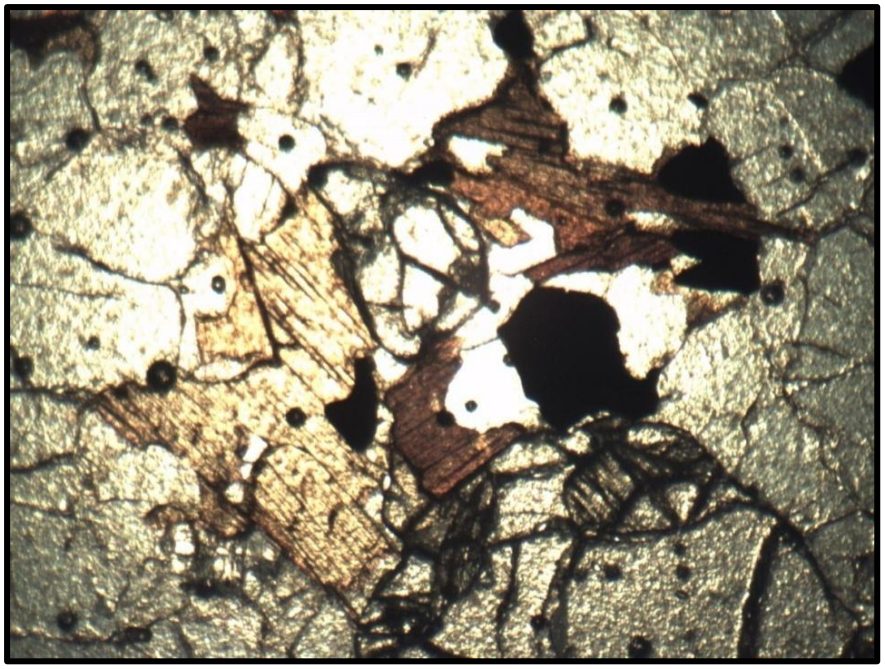


Photo 1-3: Photomicrograph of Augen Gneiss from NDH-1 at 26 – 28 m depth (PPL).

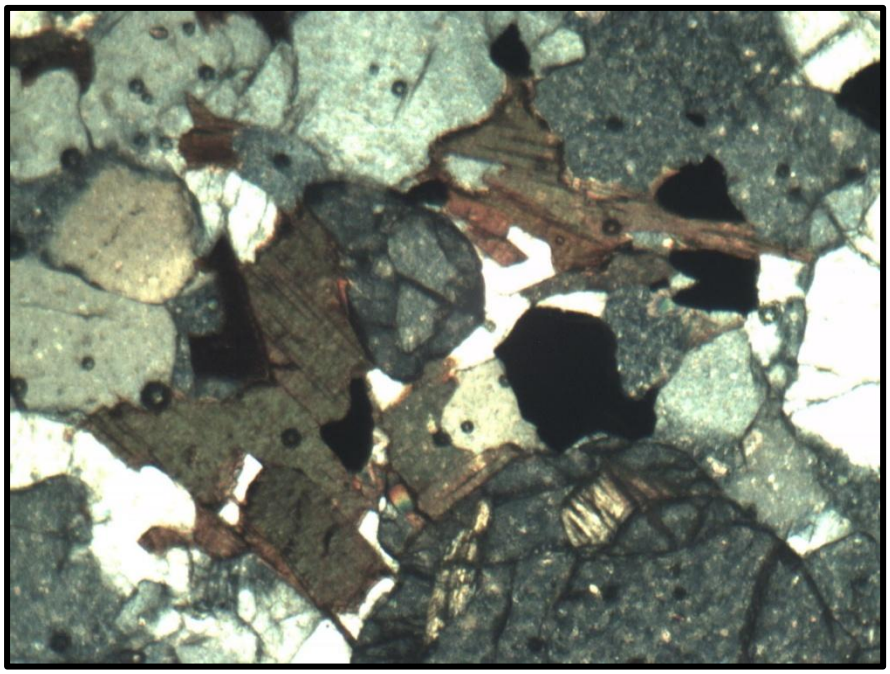


Photo 1-4: Photomicrograph of Augen Gneiss from NDH-1 at 26 – 28 m depth (UCN).

Sample No: 3, NDH-3

Sample Type: Core Sample

Sample depth: 8.00 - 9.15 m

Megascopic Study:

The rock sample is well-crystalline white marble with the presence of crystals of calcite locked together like pieces of a jigsaw puzzle and having sugary texture in the hand sample. The hand sample shows the absence of platy minerals but there is presence of other minerals, including quartz, pyrite, and iron oxides.

Microscopic Study:

Color:

Colorless, but may exhibit pale pastels High relief Visible polysynthetic “calcite” twinning in coarse grains; under cross nicols (UCN) extremely high birefringence.

Texture

- Grain size: Equigranular distribution of crystals of calcite and quartz.
- Mutual relationship: Phanaritic.

Structure:

- No of foliation: Foliation plane is not distinct. It is poorly defined based on slightly elongated grains of calcite.
- Deformation structures: Both the indented and sutured contact is developed in the quartz.
- Micro-fractures: Absent
- Others: Gneissic structure is poor. However, preferred orientation of biotite and muscovite defined gneissic structures.

Mineralogy:

- Mineral Constituents:

Table 1-3: Mineral Constituents of Sample No. 3

Plagioclase feldspar %	Orthoclase %	Quartz %	Platy minerals (Muscovite and biotite) %	Garnet %	Opaque and unidentified %
15	40	15	10	10	10

- Diagnostic Properties:

High order colors under crossed polars, very high relief that changes with rotation Calcite showing simple and lamellar twinning in plane polarized light. Quartz is distinct by anhedral shape and wavy extinction and by its absence of cleavage; biotite is highly pleochoric with 1 set of basal cleavage and parallel extinction while muscovite is identified by its multicolored elongated grain under cross nicols; Metallic minerals are identified by its dark appearance both in UCN and PPL.

Petrological classification: It is crystalline rock.
Identification: **Calcareous – Quartz - Gneiss**

Index mineral/ grade of metamorphism: medium

Engineering Evaluation:

- Microcracks: absent
- Weathering: fresh

Conclusions:

The rock is suitable for any type of construction material based on the physical property. But its chemical analysis is recommended before use. In hand sample it looks like micaceous quartzite. It is fairly calcareous with distinct calcite crystals. Interestingly, the rare layers of mica especially biotite shows the appearance of gneiss. Therefore, its name is calcareous-quartz-gneiss.

Remarks: Galena, Sphelarite , Pyrite, Chalcopyrite are present.

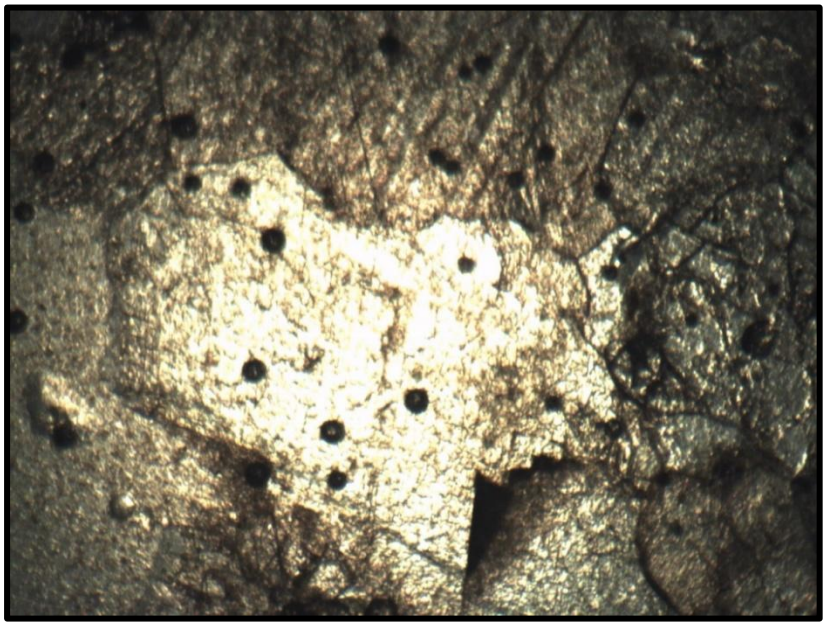


Photo 1-5: Photomicrograph of calcareous Quartz Gneiss from NDH-3 at 8 – 9.15 m depth (PPL)

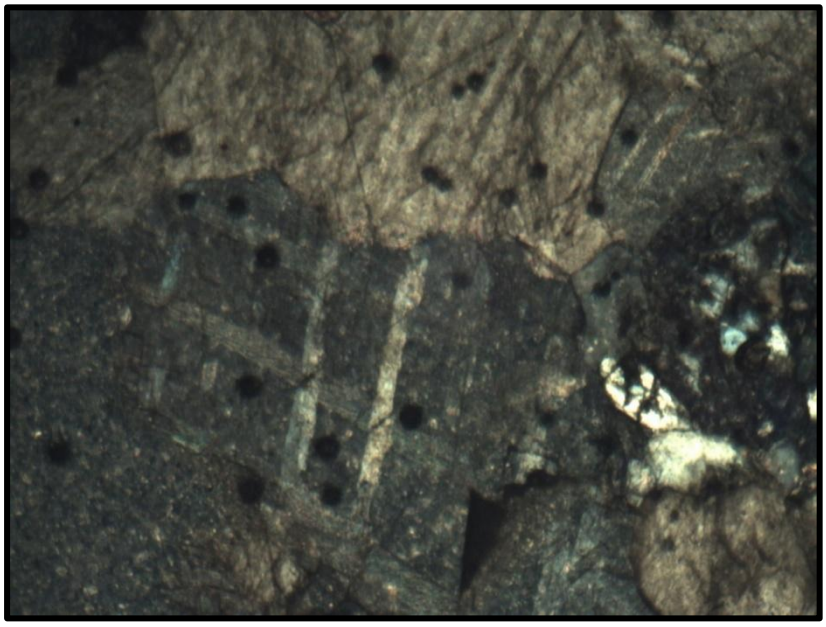


Photo 1-6: Photomicrograph of calcareous Quartz Gneiss from NDH-3 at 8 – 9.15 m depth (UCN)

Sample No: 4, NDH-3

Sample Type: Core Sample

Sample depth: 10.3-11.5 m

Megascopic Study:

The rock sample is crystalline white Calcareous Gneiss with the presence of crystals of calcite locked together like pieces of a jigsaw puzzle and having sugary texture in the hand sample. The hand sample absence of platy minerals but there is presence of other minerals, including plagioclase feldspar, orthoclase feldspar, quartz, pyrite, and iron oxides.

Microscopic Study:

Color:
Colorless, but may exhibit pale pastels High relief Visible polysynthetic "calcite" twinning in coarse grains; under cross nicols (UCN) Extremely high birefringence. Almost colorless except some pleoichroism shown by biotite under PPL. The euhedral boundaries of feldspar are distinct due to differential relief with surrounding minerals

Texture:
Grain size: Equigranular distribution of crystals of calcite and quartz. Mutual relationship: Phanaritic.

Structure:

- No of foliation: Foliation plane is not distinct.
- Deformation structures: Both the indented and sutured contact is developed in the quartz. Some quartz grains have shown core-and-mantle structures.
- Microfractures: Absent
- Others: Gneissic structure is poor due to higher percentage of psammatic minerals like calcite, feldspar and quartz. However, preferred orientation of biotite and muscovite defined gneissic structures.

Mineralogy:

- Mineral Constituents:

Table 1-4: Mineral Constituents of Sample No. 4

Plagioclase feldspar %	Orthoclase %	Quartz %	Muscovite and biotite %	Calcite %	Opaque and unidentified %
20	15	25	5	30	5

- Diagnostic properties:

High order colors under crossed polars, very high relief those changes with rotation Calcite showing simple and lamellar twinning in plane polarized light. Rhombus shape is distinct in calcite. Quartz is distinct by is anhedral shape and wavy extinction; biotite is highly pleochoric with 1 set of basal cleavage and parallel extinction while muscovite is identified by its multicolored elongated grain under cross nicols; Metallic minerals are identified by its dark appearance both in UCN and PPL.

Petrological classification: It is crystalline rock.

Identification: **Calcarious Quartz Gneiss**
Index mineral/ grade of metamorphism: Garnet grade

Engineering Evaluation:

- Microcracks: absent
- Weathering: fresh

Discussions:
The rock belongs to hard class due to significant percentage of quartz and feldspar and the presence of lepidoblastic (having high mica minerals) mineral are also significant. In unweather

condition rocks may possess high strength. The rock is suitable for construction in acid free environment.

Conclusions:
The rock is suitable for any type of construction material based on the physical property. But its chemical analysis is recommended before use to avoid from acidic environment.

Remarks: Galena, splerite, pyrite, chalcopyrite, are present. It is high grade metamorphic rock.

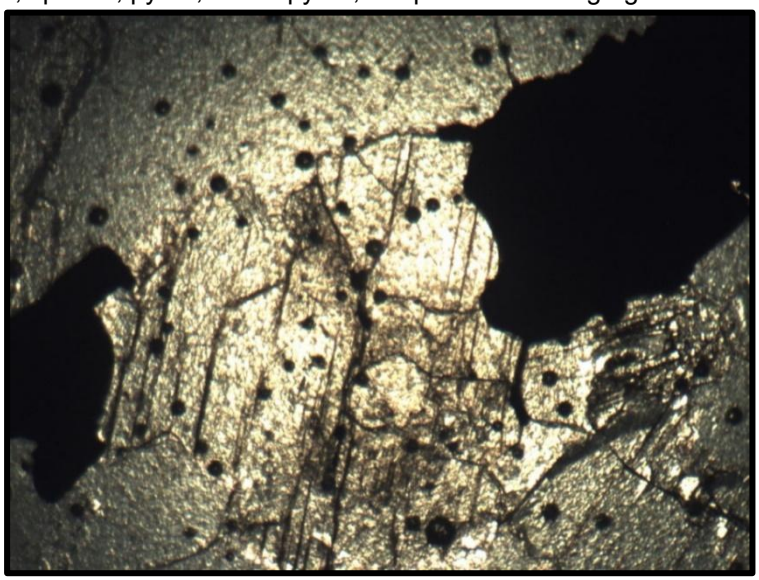


Photo 1-7: Photomicrograph of Calcareous Gneiss from NDH-3 at 10.3 - 11.5 m depth (PPL)

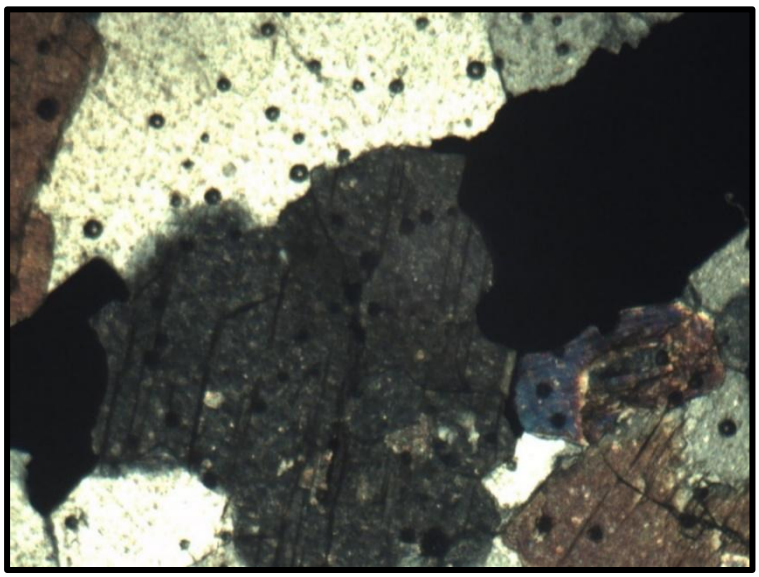


Photo 1-8: Photomicrograph of calcareous from NDH-3 at 10.3 - 11.5 m depth (UCN)

Sample No: 5, NDH-3

Sample Type: Core Sample

Sample depth: 42-43.5 m

Megascopic Study:

The rock sample is crystalline meta-quartzite with presence of crystals of metallic minerals. Recrystallized grain of quartz and opaque are distinct. There is excessive amount of metallic mineralization and act as ore in hand sample.

Microscopic Study:

Color:

Almost all of the field of vision is colorless because of the presence of quartz and feldspar in PPL. The euhedral boundaries of feldspar are distinct due to differential relief with surrounding minerals. Under cross nicol UCN, quartz grains show wonderful wavy extinction.

Texture:

- Grain size: Inequigranular distribution of minerals
- Mutual relationship: Phanaritic, equigranular

Structure:

- No of foliation: No foliation plane is distinct both in hand sample and in thin section.
- Deformation structures: Both the indented and sutured contact is developed in the feldspar and quartz. Slightly elongated grins of quartz define one set of poor foliation.
- Microfractures: present (mostly two sets intersecting)
- Others: metallic minerals are excessively present.

Mineralogy:

- Mineral Constituents:

Table 1-5: Mineral Constituents of Sample No. 5

Plagioclase feldspar %	Orthoclase %	Quartz %	Calcite %	Opaque and unidentified %
4	1	90	3	2

- Diagnostic properties:

Quartz is identified by its anhedral shape and absence of cleavage. First order gray interference color with low-relief is the strong evidence of presence of quartz which imparts it different from other minerals.

Petrological classifications: it is crystalline rock.

Identification: **Green Quartzite**

Index mineral/ grade of metamorphism: Chlorite-grade

Engineering Evaluation:

- Micro-cracks: distinct
- Weathering: Medium as imparted by the bleaching effect of other inclusions.

Discussions:

The rock belongs to hard class due to significant percentage of quartz.

Conclusions:

The rock is suitable for foundations as well as other types of engineering construction materials like fine and coarse aggregates.

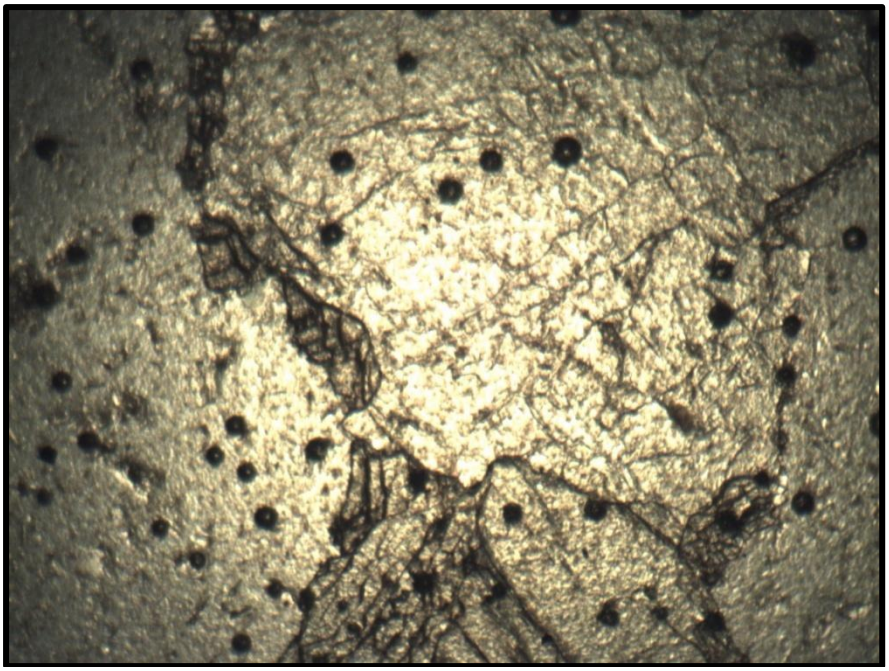


Photo 1-9: Photomicrograph of Green Quartzite from NDH-3 at 42 - 43.5 m depth (PPL).

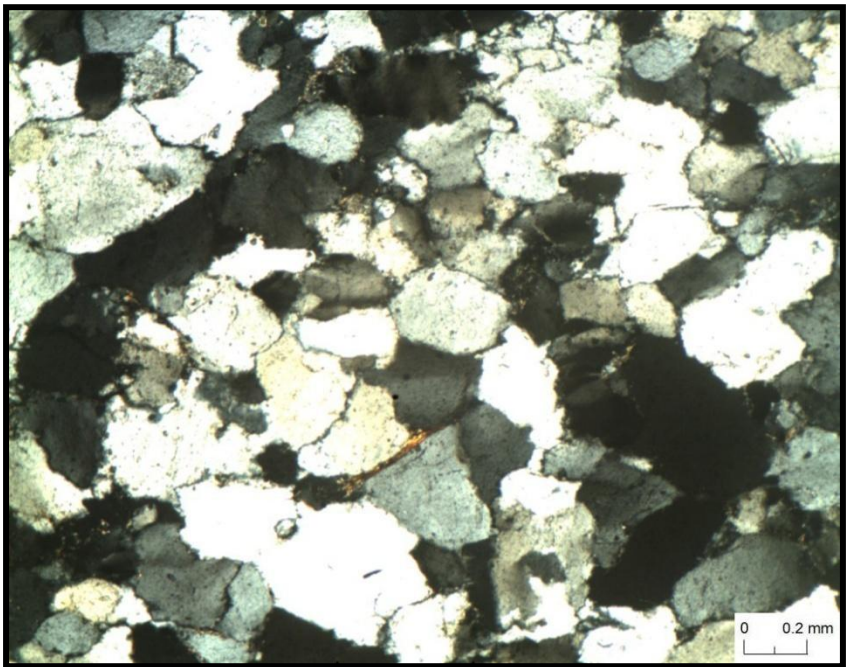


Photo 1-10: Photomicrograph of Quartzite from NDH-3 at 42 - 43.5 m depth (UCN). Grain fracturing is distinct in the grains of quartz and the mineralization is remarkable along the cracks.

Sample No: 6, NDH-4

Sample Type: Core Sample

Sample depth: 30.8-32.5 m

Megascopic Study:

The rock sample is crystalline quartzite with presence of crystals of metallic minerals. Recrystallized grain of quartz and feldspar are distinct. There is excessive amount of metallic mineralization is present.

Microscopic Study:

Color:

Almost all of the field of vision is colorless because of the presence of quartz and feldspar in PPL. The euhedral boundaries of feldspar are distinct due to differential relief with surrounding minerals. Under cross nicol UCN a bright field of vision are observed. The dark field of vision is due to the presence of the metallic minerals and other opaque minerals.

Texture:

- Grain size: Inequigranular distribution of minerals
- Mutual relationship: Phanaritic, inequigranular

Structure:

- No of foliation: No foliation plane is distinct both in hand sample and in thin section.
- Deformation structures: Both the indented and sutured contact are developed in the feldspar and quartz.
- Microfractures: present
- Others: metallic minerals are excessively present.

Mineralogy:

- Mineral Constituents:

Table 1-6: Mineral Constituents of Sample No. 6

Plagioclase feldspar %	Quartz %	Muscovite/ Biotite/ Chlorite %	Opaque and unidentified %
15	45	3	25

- Diagnostic properties:

Plagioclase feldspar shows polysynthetic twinning with subhedral shape; quartz is distinct by anhedral shape and wavy extinction; Metallic minerals are identified by its dark appearance both in UCN and PPL. Some lithic fragments are also observed.

Petrological classifications: It is crystalline rock.

Identification: **Quartzite with Mica partings and metallic minerals.**

Index mineral/ grade of metamorphism: Biotite

Engineering Evaluation:

- Microcracks: Present
- Weathering: moderately weathered

Discussions:

The rock belongs to hard class due to significant percentage of quartz. *Conclusions:* the rock is suitable for foundations as well as other types of engineering construction materials like fine and coarse aggregates.

Remarks: Metallic minerals are abundant.

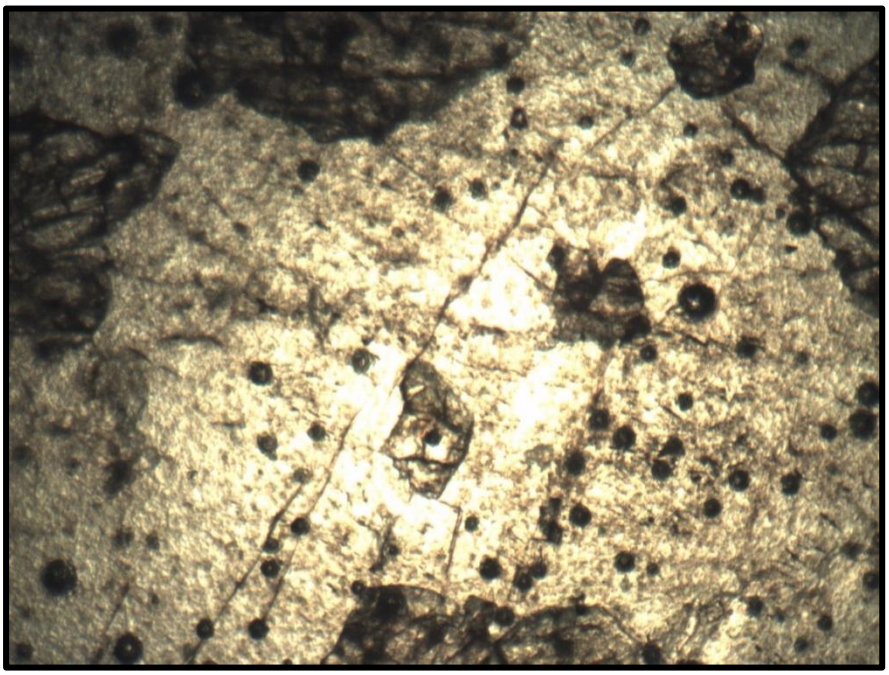


Photo 1-11: Photomicrograph of Meta sandstone with mica partings from NDH-4 at 30.8 – 32.5 m depth (PPL)

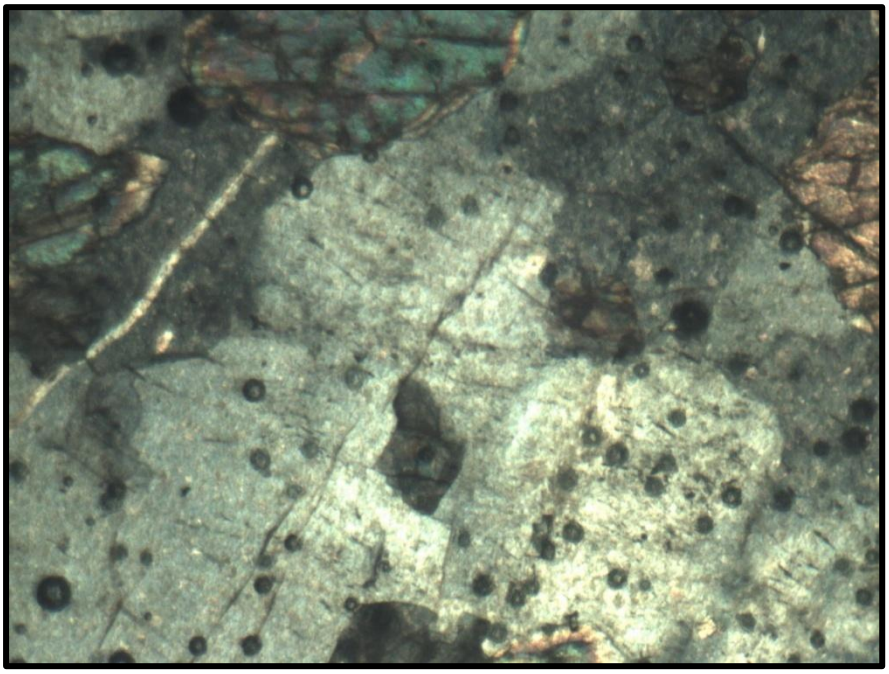


Photo 1-12: Photomicrograph of Metasandstone with mica partings from NDH-4 at 30.8 – 32.5 m (UCN).

Sample No: 7, NDH-4

Sample Type: Core Sample

Sample depth: 19.5-20.5 m

Result:

Table 1-7: Mineral Constituents of Sample No. 7

Field of Vision	Total no. of grains count	Percentage of Minerals					
		Quartz (%)	Plagioclase Feldspar (%)	Muscovite (%)	Biotite (%)	Ferruginous material (%)	Opaque and Unidentified (%)
1	200	32	16	13	19	10	10
2	200	30	15	25	15	10	5
3	200	35	9	16	29	6	5
5	200	29	12	14	25	7	13
4	200	26	20	11	18	15	10
Total No. of Grains %		**30.4**	**14.4**	**15.8**	**21.2**	**9.6**	**8.6**

Table 1-8: Visual determination of roundness

Pit No	Total no. of grain count	Percentage					
		Very angular (%)	Angular (%)	Sub Angular (%)	Sub rounded (%)	Rounded (%)	Well rounded (%)
1	240	27	40	28	4	1	0

Remarks: Most of the grain are angular and very angular. The given sample is fault gauge.

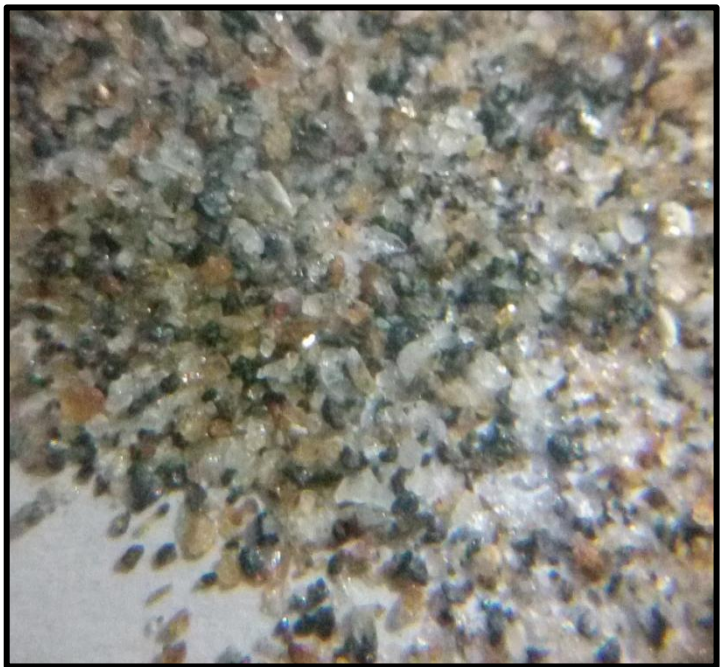


Photo 1-13: Magnified view of carbonaceous material after well-wash out in clean water several times (× 20)

1.1.1 Conclusion

Most of the tested rocks belong to the high-grade metamorphic rocks up to garnet grade and some are with biotite grade. Many samples consist of metallic minerals. Almost all rocks are calcareous in nature.

Table 1-9: Rock types and their features in drill holes

S.N.	Drill hole	Depth in m	Type
1	NDH-1	12.0 - 14.0	Garnet mica rich Augen Gneiss
2	NDH-1	26.0 – 28.0	Garnet rich Paragneiss
3	NDH-3	8.0 - 9.15	Calcareous Quartz Gneiss
4	NDH-3	10.3 - 11.5	Calcareous Quartz Gneiss
5	NDH-3	42.0- 43.5	Green-Quartzite
6	NDH-4	19.5 – 20.5	Fault Gauge
7	NDH 4	30.8 - 32.5	Quartzite with mica partings and metallic minerals

VOLUME IV: PROJECT INFORMATION

SECTION 4: PHYSICAL HYDRAULIC MODEL OF HEADWORKS

JV of Indo Canadian Consultancy Services Limited & Shah Consult International (P.) Ltd.

PHYSICAL HYDRAULIC MODEL STUDY OF THE HEADWORKS OF GHUNSA KHOLA HYDROELECTRIC PROJECT (77.5 MW)

Final Report

December 2020

Hydro Lab

IN COOPERATION WITH NTNU, NORWAY

CERTIFIED ISO 9001:2015 COMPANY DNV·GL

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REMIT HYDRO रेमिट हाइड्रो लिमिटेड

JV of Indo Canadian Consultancy Services Limited & Shah Consult International (P.) Ltd.

Physical Hydraulic Model Study of the Headworks of Ghunsa Khola Hydroelectric Project (77.5 MW)

Final Report

	Name	Signature	Date
Prepared by	Mr. Kunjan Lal Pradhan		7/12/2020
Checked by	Dr. Umesh Singh		11/12/2020
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EXECUTIVE SUMMARY

Ghunsa Khola Hydroelectric Project (hereinafter called "GKHEP") is located in Taplejung District of Province 1, Nepal. This is a Run-of-the-River (ROR) type of hydropower project with installed capacity of 77.5 MW. The project is being developed by Remit Hydro Ltd. The detailed engineering design of the project is being carried out by JV of Indo Canadian Consultancy Services Limited and Shah Consult International Pvt. Ltd. (Hereinafter referred to as "The Client"). Headworks site of the project is located at Lamabagar in Phaktanglung Rural Municipality of Taplejung District, about 7 km upstream of the confluence of Ghunsa Khola with Tamor River. The Client intends to finalize the design of the headworks through physical hydraulic model study. In this regard, an agreement was signed between Hydro Lab Pvt. Ltd. (The Consultant) and JV of Indo Canadian Consultancy Services Limited and Shah Consult International Pvt. Ltd. on 10 June 2019 for conducting the Physical Hydraulic Model Study of the headworks of GKHEP.

As outlined in the scope of works of the contract agreement for the physical hydraulic model study, the following activities were carried out by the Consultant:

- Input data processing, model planning and model design
- River model construction and validation.
- Testing of existing situation of the river
- Construction of the headworks model
- Testing of the given design of the headworks and its minor modifications
- Testing of the selected design and documentation
- Data analysis and Final reporting

The model was built as an undistorted fixed bed model (with sediment supply) in a scale of 1:30 using the Froude's Model Law. A 400 m long river reach of Ghunsa Khola at the headworks starting 250 m upstream to 150 m downstream of the weir axis was reproduced in the river model. The river model was validated against the water level data derived from the 1-D numerical model, provided by the Client, as the water level measurement data in the river during high flow period were not available. Existing situation tests, at 7 different steady flows, were carried out in the validated river model to understand the behaviour of the river mainly during flood situations and to collect data which would allow making comparison of the river behaviour before and after placement of the headworks structures. The tests mainly focused on the hydraulics, sediment deposition patterns and velocities particularly around the headworks area.

Headworks structures were fabricated based on the drawings provided by the Client and were placed in the river model for testing. The tests revealed the following shortcomings in the performance of the base case design:

- Inability of the bedload flushing gates to control the bed deposits in front of the intake
- Unsatisfactory performance of gravel trap flushing system
- Clogging of trash racks by floating debris due to lack of trash passage facility
- Unfavourable hydraulics in front of the intakes due to strong secondary flow from the weir towards the intakes

- The impact of concentrated flow during the floods threatened the structural stability of gabion wall at the left bank, upstream of the weir
- Stilling basin was unable to dissipate the design flood at scenario in which one bedload flushing gates was in inoperable condition. (n-1 gate operation)
- Supercritical flow downstream of the end sill of stilling basin was deemed unfavourable for the stability of boulder ripraps.

The shortcomings of the base case design was discussed with the Client during a meeting held on 17th Oct 2019 at Hydro Lab premises. After the discussion on the test observations, both Parties agreed to carry out modifications in the base case design to improve the performance. Various concepts for modifications were formulated and were first conceptually tested in the model. Based on the test observations 'Modified Design Arrangement I' was developed and agreed for the testing. The modifications mainly involved introduction of undersluices, trash passage structures and gravel traps; change in upstream shape of the weir and changes in the orientation of the intake. In the new design sluicing gates provided in the base case design shall be only used for flushing and flood passage, hence here after they are referred as bedload flushing gates. The design modifications greatly improved hydraulics and bed control in front of the intake. However, some shortcomings were still observed with respect to passage of floating debris, secondary flow in front of intake, gravel traps flushing and energy dissipation in stilling basin.

The performance of the "Modified Design Arrangement I" was discussed with the Client in a meeting held on 20th Dec 2020 at Hydro Lab premises. Both parties agreed to carry out further modifications in the design and "Modified Design Arrangement II" was formulated. The modifications mainly involved introduction of the hopper type gravel traps and over flow trash passage structure just upstream of the bedload flushing gates; extension of the divide wall and constriction of the outlet of the stilling basins. Both parties agreed that hopper type gravel trap with individual flushing channel would significantly improve the gravel flushing performance. As per the Client's request to expedite the model study due to time constrain hopper type gravel trap design performance was not tested in the model.

The tests of the "Modified Design Arrangements-II" revealed that the introduction of overflow trash passage structure in front of the bedload flushing gates, significantly reduced the intensity of the secondary flow. The structure also improved the passage of floating debris. Similarly, after constricting the outlet of the stilling basin by 4 m, the hydraulic jump at the design flood (1 in 100 year's return period) was contained within the stilling basin even in the scenario when one of the bedload flushing gates was not in operation. So the headworks layout, excluding settling basins, after "Modified Arrangement-II" was deemed to perform satisfactorily. The super critical flow downstream of the stilling basins, however, was not favourable for the stability of the boulder rip-rap structure.

Settling basins were then implemented in the "Modified Arrangement-II" for the testing. The test results revealed the following shortcomings in the performance of the settling basin:

- Uneven flow distribution between the settling basins
- Flow at the end of the upstream transition zone was concentrated near the bottom of the basin resulting in reverse flow at the crown of the settling basin.
- Effective length of the settling basin reduced to 80 % of the total available length due to non-uniform distribution of flow in vertical direction.
- Haunting oscillation of flow was observed as a result of non-uniform velocity profile exiting from the inlet transition of the Settling Basin.
- Stagnant recirculating zone observed on both side of the settling basin due to abrupt change in flow area at the basin outlet.

A meeting was held on 5th Feb 2020, between the Client and the Consultant at Hydro Lab premises, to discuss the model observations. The Consultant recommended the following modifications for improving the performance of the settling basin.

- The outlet gates of Settling Basin #2 and Settling Basin #1 shall be operated at 3 m and 3.9 m openings respectively to obtain even flow distribution between the basins. Furthermore, the layout of the outlet tunnel from the settling basins will be adjusted for obtaining the equal head loss and equal withdrawal of discharge from the settling basin.
- A straight and flat identical approach tunnels (with 0 % slope) will be introduced at the entrance of the settling basins to reduce concentration of flow near the bottom of the transition zone
- Angle of inlet transition will be reduced or the size of the approach tunnel will be increased for improving hydraulics at the entrance of the basin.
- An exit transition will be implemented for smooth flow exit path at the end of the basin.

The implementation of the proposed modifications in the settling basins and verifying the performance after modifications would require time extension. As per the Client's request to expedite the model study due to time constrain the recommended modifications will be implemented in the final design of the settling basin, but will not be tested in the physical model. Hence the "Modified Arrangement II" with the implementation of the recommended modifications in the settling basins was considered as the final recommended hydraulic design.

Model tests of the Final Recommended Hydraulic Design (FRHD) were carried out for the final documentation purpose. The recommended modifications were not implemented in the gravel trap and settling basins, so their performance could not be verified in the model. Based on the results of the model study, the following recommendations are made:

- The "Modified Arrangement II" with the implementation of the recommended modifications in the settling basins shall be adopted as the recommended hydraulic design of the headworks.
- The operation regime recommended by this study shall be updated / modified based on the actual operation experience at site.
- Undersluices are prone to clogging when operated at small gate opening. Hence, it is strongly recommended to operate the upstream and downstream undersluices at an opening of 0.6 m and 0.4 m respectively.
- The openings of undersluices shall not be altered during recession period of flood to avoid their clogging and excessive sediment deposition in front of the intake.
- It is strongly recommended that gravel trap flushing gate should be operated at fully open or fully closed position to avoid clogging.
- Once water level in front of the intake rises above 1948.00 masl the trash passage gate shall be closed to avoid overloading of the flushing channel and submergence of the gravel trap and undersluice outlets.
- Proper sediment deposition monitoring system is recommended to be installed in front of the intake while flood motoring system is recommended to be installed upstream of the headworks to collect lead information on flood magnitude required for operating the gates properly.
- Bedload flushing Gates (formerly named as Bedload sluice in Base Case Design) is meant to be operated for gravity flushing of the sediment deposited in front of the intake, passage of extreme flood event (higher than 50 years floods) and for

lowering the water level for routine inspection and maintenance of the intake and undersluices. Hence, it shall be closed during rest of the operation period.

- The deposits in front of the intake shall be flushed towards the end of the monsoon season to avoid longer period of shutdown required for flushing the deposits during low flow season, for regular inspection and maintenance.
- To maintain favourable upstream approach flow hydraulics and to reduce cross current secondary compensation flow, the sediment deposits at right bank upstream of intake shall be kept lower than 1946.00 masl. If the deposit is higher and no flow is observed over the deposit then it shall be flushed or removed manually.
- Before starting the flushing sequence, a stoplog of minimum 1 m height shall be installed at the intake to avoid sediment entering the intake.
- This model study was mainly focused on improving the performance of the headworks structure and finalizing the hydraulic design. Thus, it is recommended to carry out necessary structural analysis of the Final Recommended Hydraulic Design from this study before implementing it for construction. It is worth mentioning that the apron downstream of the stilling basin is prone to damage by super critical flow during major floods. Hence, design of a suitable protection structure is recommended during structural detailing.

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1 INTRODUCTION

1.1 Background

Ghunsa Khola Hydroelectric Project (hereinafter called “GKHEP”) is a Run-off-the-River (ROR) project located in Taplejung District of Province 1, Nepal. The project has an installed capacity of 77.5 MW generated through a gross head of 432 m and a design discharge of 21.28 m^3/s at a 43 % probability of flow exceedence. Headworks site of the project is located at Lamabagar in Phaktanglung Rural Municipality of Taplejung District, about 7 km upstream of the confluence of Ghunsa Khola with Tamor River.

GKHEP is being developed by Remit Hydro Ltd. (Hereinafter referred to as "The Developer"). The detailed engineering design of the project is being carried out by JV of Indo Canadian Consultancy Services Limited and Shah Consult International Pvt. Ltd. (Hereinafter referred to as “The Client”). The Client intends to finalize the design of the headworks through physical hydraulic model study. In this regard, an agreement was signed between Hydro Lab (The Consultant) and JV of Indo Canadian Consultancy Services Limited and Shah Consult International Pvt. Ltd. on 10 June, 2019 for conducting the Physical Hydraulic Model Study of the headworks of GKHEP. The model study incorporates all parts of the headworks structures including the settling basins at a geometric scale ratio of 1:30. Only hydraulic phenomena will be studied in the settling basins i.e. no suspended sediments will be simulated.

1.2 Salient features of Ghunsa Khola HEP

The brief salient features of the base case design of GKHEP are presented in Table 1-1.

Table 1-1: Salient features of the project for Base Case Design

Hydrology	
Design Discharge, Q_{43}	21.28 m^3/s
Mean annual flow	32.21 m^3/s
Design Flood at intake site, Q_{100}	880 m^3/s
1 in 10,000 Flood	1,442 m^3/s
General features	
Type	Run-of-River (ROR)
Gross Head	432 m
Installed Capacity	77.50 MW
Diversion weir	
Type	Concrete Weir
Length	35 m
Crest elevation	1947.00 masl
Full supply level	1947.00 masl
Design flood level	1952.00 masl

Bedload sluice	
Number of openings	2 nos. (with radial gates)
Size (W X H)	4.5 m x 4.5 m
Crest elevation	1940.25 masl
Intake Structures	
Type	Side Orifice
Number of openings	4 nos.
Size of opening (W X H)	3.5 m x 3.1m
Invert level of intake	1941.00 masl
Number of trash racks	4 nos.
Size (W X H)	3.5 m x 3.5 m
Gravel trap	
Numbers	1 no.
Size (W X H X L)	0.5 m X 0.5 m X 17.5 m
Size of flushing channel (L X W X H)	40.7 m X 0.5 m X 0.5 m
Connecting approach tunnel	
Number	2 Nos.
Type	Inverted D-shaped, Pressurized
Size (W X H)	3.1 m X 3.1 m
Length and Slope (Tunnel 1)	155.60 m, 1 in 518
Length and Slope (Tunnel 2)	122.60 m, 1 in 376
Settling basins	
Type	Underground, pressurized
Number of bays (Caverns)	2 Nos.
Size of Caverns (L X W X H)	62 m X 11.5 m X 14.65 m
Flushing operation model	Continuous

1.3 Objective of the study

The main objective of this model study is to provide input in finalising detailed design of the headworks of the GKHEP. The study mainly focused on the hydraulic performances of the headworks structures (including the settling basins) as well as it's ability to exclude bed load and floating debris in the flow to the intakes. The main objectives of the model study as outlined in the contract are as follows:

- To study the natural flow regime and sedimentation pattern near the headworks during different flow conditions.
- To study the hydraulic performance of intake, weir, stilling basin, undersluices and gravel flushing structure considering different flow conditions.
- To study the performance of the headworks structures with respect to handling of floating debris and bed load.

- To verify the structural arrangement to obtain a favourable hydraulic design for proper flood passage and energy dissipation
- To evaluate hydraulic performances of the settling basins using dye tests.

1.4 Scope of Works and Methodology

The scope of works under the physical hydraulic model study of the headworks of GKHEP and their methodology is explained in the following sections.

1.4.1 Input data processing, model planning and preparations

All data required for the planning, design, construction, validation and testing of the physical model was acquired from the Client and processed as necessary. River cross-sections and topography; geometric features within river in the model area such as size, shape and location of large boulders; rock surfaces, vegetation and river bed and river bank materials, were analysed for the river model construction. Hydrological data, sediment data and bed load feeding rates were analysed for determining the model operating parameters. Similarly, water level at corresponding high discharge were analysed for the river model validation. Furthermore, headworks design drawings were scaled and construction drawings were prepared for the structure fabrication. A suitable site in the laboratory premises was selected for the physical model construction. The model plan was prepared and laid out at the site. The site clearance and preparation activities were then carried out for the model construction.

1.4.2 Construction of the river model and fabrication of the headworks

The physical hydraulic model was scaled according to Froude's model law with the length scale ratio as 1:30. The model should be able to simulate the resulting flow patterns, sediment load transport patterns as well as the transport pattern of the floating debris in the river within an engineering context.

The topography features of the river in the modelled area were reproduced using the cement mortar. Similarly, the large boulders and other geometric features within the river were also represented in the model. Water supply to the model and return of the flow to the laboratory reservoir were also arranged. Sediment feeding at the upstream end of the model was arranged and a sediment trapping facility at the downstream end of the model was also constructed. Finally, the headworks structures were fabricated in the laboratory based on the construction drawings.

1.4.3 Validation of the river model

Model validation was performed by simulating the moderate and high flows with and comparing measured water levels in the model with those observed in the prototype. However, the crest gauges were not installed at the model study area. Therefore, after consultation with the Client, it was agreed to adopt the water levels at the desired flow derived from HEC-RAS (Numerical model) simulations provided by the Client

The river bed resistance was fine-tuned and the topography was adjusted in the model to reach an acceptable level of accuracy with water level derived from HEC-RAS (Numerical model) simulations. When a model is capable to simulate desired flow derived from HEC-RAS (Numerical model), it is verified that the model can be used to

study other events like floods or the hydraulic performance and the effects of introducing weir and other headworks structures in the river.

1.4.4 Testing of existing situation in the river model

The flow patterns, water levels, bed load deposition and transport patterns in the river model have to be observed under various flood conditions, before implementation of the headworks. Knowledge on the behaviour of the river during floods is valuable input to the design process of the headworks and to assess the change in behaviour of the river after the headworks structures are introduced and tested in the model.

So, 7 steady flows, including the design flood, were simulated in the river model. Various parameters characterising the behaviour of the river in the modelled area were also measured. The same flow situations were used for testing the performance of the proposed structures. The data measured in this activity were used for comparing the river characteristics after the headworks structure was built.

1.4.5 Testing of the given design and modifications

The headworks structures were then introduced in the river model for testing their performance. A series of tests were carried out for documenting the performance of hydraulic design to explore the possible arrangement of the intake/headworks for satisfactory performance of the different headworks components. The performance of the headworks were evaluated against the following criteria:

- Approach flow hydraulics and hydraulics within the headworks structures
- Safe passage of bed load
- Safe passage of the floating debris
- Safe passage of the design flood
- Abstraction of design flow through the intake
- Acceptable performance of stilling basin
- Acceptable hydraulic performance of settling basins

If the proposed arrangement did not perform as intended, the headworks components were modified and tested until the desired performance was achieved. Finally, the arrangement of the headworks, which performs satisfactorily is recommended.

1.4.6 Detailed testing and documentation of the selected arrangement

A comprehensive test programme, including 6 steady flows, was carried out for the selected headworks arrangement. Hydraulics as well as sediment related data were collected for the documentation. Photographs and videos of tests were also recorded for the documentation. The rating curves of the weir, gates and openings of the headworks were also derived. Finally, the operational principles of the headworks were also recommended to be implemented at least during the initial years of operation.

1.4.7 Final reporting

Results of the model study with relevant data were produced including the description of operational aspects of the recommended design based on the assumptions made during the design and model testing.

2 INPUT DATA

A number of data and information are required for constructing, validating and reproducing the physical features of a river model and headworks structures in the laboratory and testing their hydraulic performance.

The basic data used for the construction of the model are briefly summarized below. The list of input data made available to the Consultant for the model study is presented in Table C-1; Annex C. The available data were then processed for using it in model study.

2.1 Topographical data

Topographic survey of the project site was carried out by the Client and contour map covering the study area with contour interval of 1 m was provided. Similarly, cross-section survey of the river was carried out by the Client at about 15 m interval and made available for the model study. Photographs taken at the headworks area were also made available as complementary information for topography shaping of the river model.

2.2 Boulder and surface material information

Boulder survey was carried out by the Client to identify location, shape, size and orientation of the large boulders which would influence the river hydraulics. A guideline for boulder survey was provided by the Consultant. Accordingly, the Client conducted survey of boulders larger than 1 m in diameter located within the model boundary. Dimensions, coordinates and highest elevation of the surveyed boulders were provided based on which they were individually placed in the river model.

Information on surface bed material, required to simulate bed roughness in the river model, was not available for this study. Therefore, the surface roughness was developed, based on the available visual information (photographs and videos).

2.3 Water level data

Water level measurements at various locations for a known flow in the prototype are required for validation of the river model. High flow water levels are measured with the help of crest gauges installed at various locations in the river banks at the headworks area.

However, the crest gauges were not installed at the model study area. Therefore, after consultation with the Client, it was agreed to adopt the water levels at the desired flow derived from HEC-RAS (Numerical model) simulations provided by the Client.

2.4 Hydrological data

Compilation and analyses of hydrological data, required for the model study, were provided by the Client. The hydrological data referred in the model study are as follows:

- Mean monthly flows at the dam site, presented in Table 2-1.
- Floods with different return periods, presented in Table 2-2.

- The average monsoon at the headworks site was adopted to be 75 m^3/s

Table 2-1: Mean monthly flows at the intake site (m^3/s)

Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
6.70	5.22	5.09	8.43	16.07	42.56	86.33	94.52	67.70	30.35	13.60	9.95

Table 2-2: Flood flow at the Intake site (m^3/s)

Return period in years	Annual flood	2	5	10	20	50	100
Flood flow	164	364	502	593	681	794	880

2.5 Sediment data and bed load feeding rates

The nature of sediment transport in the steep mountainous rivers depends largely on the supply from the catchment and geomorphology of the river channel. Bed load transport rates at high flows is difficult to measure in steep rivers. So, suspended sediment measurements (which can be sampled rather easily) are often used to estimate the bed load transport rates.

Suspended sediment sampling in the Ghunsa Khola, at the vicinity of the headworks, was carried out by the Client during the monsoon of 2009 and 2019. However, bed load was not measured. Based on the field measurements, discharge vs suspended sediment rating curve was provided by the Client which is presented in the Figure B-1, Appendix B. The bed load feeding rates in the model for different discharges were estimated based on the provided rating curve.

Table 2-3: Adopted bed load transport rates

Return Period	Flow (m^3/s)	Measured suspended sediment discharge (kg/s)	Total suspended sediment discharge (kg/s)	Estimated bed load Discharge (kg/sec)	Adopted bed load in model (g/min)	D critical, model (mm)
Design discharge	25.25	6.60	7.92	2.77	33.72	18.9
Low Monsoon Flow	45	16.35	19.63	6.87	83.60	24.2
Avg. Monsoon flow	75	36.50	43.79	15.33	186.57	31.5
1	164	124.79	149.75	52.41	637.93	46.7
2	364	436.80	524.16	183.46	2,232.97	68.2
5	502	723.86	868.64	304.02	3,700.45	74.5
10	593	940.48	1128.58	395.00	4,807.80	79.6
20	681	1168.90	1402.68	490.94	5,975.52	83.6
50	794	1487.82	1785.38	624.88	7,605.83	90.2
100	880	1748.76	2098.52	734.48	8,939.82	93.9

Note:
Total suspended Sediment = 1.2 X Measured suspended sediment
Estimated bed load = 35 % of Total suspended Sediment

In order to take account of the unmeasured suspended sediment concentration and as a safety factor for other unexpected sediment load during monsoon period, the values were increased by 20 % for total suspended sediment load estimates. The bed load is commonly estimated as 10 to 20 % of the total sediment load. However, the proportion ranges from 20 % to higher than 40 % for the mountain streams (Turowski et al., 2010 cited in Ziegler et al., 2014). Since the headworks of the project is located in a steep mountain reach, the bed load is assumed as 35 % of the total suspended sediment load for this study. The applied bed load feeding rates and the largest size of the bed load material supplied in the model are given in Table 2-3.

The Particle Size Distribution (PSD) of bed load feeding was computed based on the pit samples data at the headworks site (shown in Figure B-2, Appendix B), made available by the Client.

2.6 Headworks design and drawings

A list of design drawings of the headworks provided by the Client and adopted for the model study purpose is presented in Table 2-4.

Table 2-4: Key Reference Drawings

S.N	Drawing No.	Description	Remarks
1	A-1	Headworks model layout	Drawings prepared at Hydro Lab
2	A-2	Layout Plan of Headworks	Drawings provided by the Client
3	A-3	Plan view of Headworks	
4	A-4	Weir Section view	
5	A-5	Intake Section Details	
6	A-6	Settling Basin Section View	

3 MODEL CONSTRUCTION

3.1 Model laws and scales

Modelling strategy requires due consideration of similitude, appropriate or practicable model scales and model layout for its construction.

Full model-prototype similitude requires satisfaction of the following conditions:

- Geometric similitude, whereby the ratio of all length dimensions are same and where only similarity in form is involved;
- Kinematic similitude (time and velocity), whereby at all geometrically homologous points in model and prototype, velocities and accelerations are in a constant ratio;
- Dynamic similitude (force), whereby, in addition to kinematic similitude, the force polygons are similar at geometrically equivalent points for model and prototype.

It is known that for a river model where flow is driven by gravity (open channel flow), the inertial forces are predominant and the similitude between model and prototype is governed by Froude Number. Hence, the representation of the flow system in the river is governed by the law of gravity flow or the Froude law. According to this law, Froude number (F) in the prototype has to be same at the corresponding location of the model:

$$F_r = \frac{v_r}{\sqrt{gD_r}} = 1$$

Where, v= velocity of flow (m/s)
g = acceleration due to gravity (m/s^2)
D = hydraulic depth (m)
Subscript 'r' indicates the model to prototype ratio

The influences of surface tension and viscosity are not considered in Froude's Law. The flow being mostly turbulent in case of river flows, the influence of these forces are negligible and the Froude's Law holds true for the case. However, the influence of surface tension could be significant in calm and shallow areas of the river flow.

The model scale of 1:30 was chosen considering the nature of the problems to be studied, the length of the river stretch to be modelled and amount of the maximum flood discharge to be simulated in the model. The scale ratios shown in Table 3-1 are generated according to Froude scaling law for the model. The available input data were then processed based on these scale ratios and used for model studies.

Table 3-1: Model scale ratios

Parameter	Unit	Relative scale	Scale ratio
Length	m	Lr	1:30
Velocity	m/s	$Lr^{1/2}$	1:5.48
Time	sec	$Lr^{1/2}$	1:5.48
Discharge	m^3/s	$Lr^{5/2}$	1: 4,929
Area	m^2	Lr^2	1:900
Volume	m^3	Lr^3	1:27,000
Mannings roughness	$s/m^{1/3}$	$Lr^{1/6}$	1:1.76

3.2 Model Construction

After acquiring all necessary data from the Client, river model was constructed with the highest achievable standard and quality by using standard construction techniques and precise tools and equipment. The model construction process is briefly described in subsequent sections.

3.2.1 Model layout and site preparation

The model comprises of the main river channel of Ghunsa Khola at the headworks. The model boundary covers the river reach extending 250 m upstream and 150 m downstream of the weir axis. Hence, plan of physical model including a 400 m long river reach was laid out at a suitable space in the laboratory for the model construction. The model layout plan is shown in Drawing A-1, Appendix A.

After selecting area for the model construction within the laboratory premises and finalising the model layout plan, the construction site was cleared and model boundary was delineated for the construction of the river model. Base of the model was constructed in several steps, to accommodate the longitudinal slope of the river. The base was made impermeable to prevent leakages from the river model. Grid (reference) lines were established on the base and at least three permanent bench marks were installed in the grid lines. All of the construction works were controlled by giving exact coordinates and levels with reference to established bench marks and grid lines. Construction of the boundary wall was then carried out based on the model layout. The height of the boundary walls were built slightly higher than the top of the topography representing the area.

3.2.2 Construction of river model

The cross-section data were processed and prepared in the form of wooden templates to construct the river topography. Altogether 27 cross-sections were prepared of which 19 represented the upstream reach and 8 represented the downstream reach of the river from the proposed weir axis. Based on the established bench marks, the cross-sections were then placed on the model base at correct level and orientation with the help of theodolite and level machine (Photograph D-1, Appendix D).

After the erection of cross-section templates, the space between two consecutive sections was filled with bricks up to the required level. Since the river model is built as a fixed bed model, the shaping of the topography was done using cement mortar. Topographic map and photographs of the modelled area were referred while shaping the river topography between consecutive cross sections.

Existing boulders have significant influence on the hydraulics of the river flow. So, most of the boulders of size 1 m and larger were individually placed in the river model referring to their location obtained from survey data, photographs and topographic maps. Theodolite and level machine were used to place these boulders at correct location, orientation and level. The boulders smaller than 1 m were abundant in the river so, they were also taken into account while providing surface roughness in the model. Photograph D-2 in Appendix D shows the final view of the river model.

3.2.3 Inflow and outflow arrangements

In order to simulate required discharge in the model, a discharge measurement box fitted with a calibrated V-notch was placed at the inlet tank of the model. Piping arrangements from the pump house to the discharge measuring box were assembled to supply the discharge to the model. Similarly, the discharge to the power plant, diverted through the intakes, was also measured with the help of discharge measuring box fitted with a calibrated V-notch installed after the settling basins.

Sediment was supplied at the model inlet with the help of a vibrating type sediment feeder. Sediments were supplied at the section 0+250 m upstream of the weir axis, sufficiently far from the headworks area. This would allow the river channel to regulate the sediment transport such that the sediment transport can be modelled more accurately in the vicinity of the headworks area. The calculated bed load feeding rates (given in

Table 2-3) were supplied for respective flows.

4 MODEL VALIDATION

The constructed river model should reproduce relevant hydraulic processes of the prototype according to the model law applied for the study. This process of ensuring the achievement of prototype flow situation in the model, such that it could be used for studying other events under varied conditions is known as model validation. Model validation involves verifying the representation of topography and flow resistance of the prototype in the model by simulating the historic events with known discharge and water levels. The topography and/ or the flow resistance in the model is fine-tuned until an acceptable level of accuracy is achieved. The river model ready for validation is shown in Photograph D-2 in Appendix D, whereas the progress of validation test at model can be seen in Photograph D-3, Appendix D.

4.1 Input data

As per the Client's request, water levels obtained from HEC-RAS model simulations were used for model validation. The water levels were compared at different measurement sections around the headworks area. Along each cross-section, measurements were carried out at three different locations (at left, center, right) and were averaged to obtain water levels at different flows. The measurement sections are shown in Drawing A-1, Appendix A. Flow of 75 m^3/s, 164 m^3/s and 593 m^3/s - which represent average monsoon flow, average annual flood and 10 years return period flood respectively - were selected for the validation.

A turbulent and non-uniform flow was observed along the river bend upstream of the headworks. The large boulder located in the middle of the channel at cross section #33 also obstructed the flow causing backwater influence upstream of it. These influences in the flow characteristics is however, not solved by the HEC-RAS model. So, sections #3, #5, #6 and #7 which did not have such influence were selected for the validation.

4.2 Validation

The selected flows were simulated in the river model and the corresponding water levels were measured at the designated locations along the left bank, right bank and the centre of the river. The average value of the water levels was used for the comparison.

The river topography was checked based on available topographic maps and photographs while fine tuning the model. Finally, trial and error method was applied in fine tuning the roughness of the river model until the required water levels were achieved within the chosen tolerance limit. The finally accepted water levels in the river model for the selected discharges are shown in Table 4-1. Detail measurement of water level is presented in Table C-2 to C-4 in Appendix C.

Table 4-1: Measured water levels in the validated river model compared to the prototype (numerically simulated)

Chainage from Weir Axis (m)		70	7	-29	-66
HEC RAS Section		375	300	275	225
Measurement section in model		XS-3	XS-5	XS-6	XS-7
Avg. Monsoon Flow (75 m^3/s)	Measured Water level in the model, (m)	1946.42	1942.78	1941.71	1938.71
	HEC RAS Water level data (m)	1946.5	1942.71	1942.05	1938.6
	Deviation (m)	0.08	-0.07	0.34	-0.11

Chainage from Weir Axis (m)		70	7	-29	-66
HEC RAS Section		375	300	275	225
Measurement section in model		XS-3	XS-5	XS-6	XS-7
Annual Flood (164 m^3/s)	Measured Water level in the model, (m)	1947.025	1943.51	1942.23	1939.125
	HEC RAS Water level Data (m)	1947.32	1943.3	1942.57	1939.07
	Deviation (m)	0.295	-0.21	0.34	-0.055
10 yr Flood (593 m^3/s)	Measured Water level in the model, (m)	1948.07	1945.44	1943.545	1940.54
	HEC RAS Water level Data (m)	1947.73	1945.2	1943.85	1940.53
	Deviation (m)	-0.34	-0.24	0.305	-0.01

The maximum deviation of 0.34 m was observed in the water level at Section #6. Such deviation is acceptable considering the uncertainty and limitation of the numerical model in capturing the phenomenon of the steep rivers.

4.3 Discussion

During validation of the river model, flow resistance and representation of the topography were mainly adjusted. Adjustment in the roughness of the river was necessary in fine tuning of the model. After few trials, the water levels measured in the model compared to the numerical simulation results were found to be well within the tolerance range. The river model was therefore considered ready to simulate different hydraulic processes to study hydraulic and sedimentation phenomenon in the river at different floods.

5 EXISTING SITUATION TEST

The main objective of the existing situation test is to compare river behaviour before and after placing the headworks structures in the river model, at different flow and flood conditions. This test includes recording of flow patterns, water levels, velocities, bed load depositions and floating debris transport patterns. Eventually, these records are used to identify the changes in the river behaviour after the construction of the headworks structures.

5.1 Selected flow situations and sediment supply

In order to document the above-mentioned behaviour of the river, a series of flows, as shown in Table 5-1, were selected.

Table 5-1: Selected flow situations

	Return Period	Flow (m^3/s)
Monsoon flow	Low Monsoon flow	45
	Average Monsoon flow	75
Small Floods	1	164
	2	364
Large Floods	5	502
	10	593
Very Large Floods	20	681
	50	794
Extreme Floods	100	880

Details of the bed load feeding rates is mentioned in Section 2.5. The applied bed load feeding rates and the largest size of the bed load material fed in the model are given in Table 2-3. The Particle Size Distribution (PSD) curve of the feed material is shown in Figure B-2, Appendix B.

5.2 Data collection

Based on the river morphology and the location of the different components of the headworks structures, 8 different measurement sections (data as shown in Drawing A-1, Appendix A) were selected to record. The chainage of the measurement sections were calculated along the thalweg starting from the most upstream measurement section. Measurement Section #5 is located 7 m upstream of the weir axis. Water levels and the velocities were recorded at three different locations along the measurement section: left bank, centre and right bank. Water level and bed level measurements were carried out with the help of level machine while velocities were measured with the help of a micro propeller current meter. The velocity measurements were carried out at 60 % of total depth from the water surface. It is to be noted that the water levels were measured close to the banks while the velocities were measured at certain distance from the bank where the flow depth was sufficient enough for the micro-propeller readings. The observed bed deposit and boulder movement were captured in photographs and video recordings.

5.3 Existing situation test results

The river model starts from 250 m upstream of the weir axis. The river at the upstream end is narrow, steep and consists of two consecutive river bends. Downstream of these bends, the river gradually widens and has a mild slope. The river in this stretch also has a mild curvature with outer bend located at left bank. The river then abruptly narrows to downstream of the weir axis. Downstream of this constriction the river is very steep and consists of river cascades. Based on these distinct geomorphological features, the river reach at the headworks area was divided into three sub-reaches: upstream reach, middle reach and downstream reach. The river stretch between Section #1 and Section # 3 was defined as upstream reach and the reach includes the two river bends. The river stretch between Section #3 and Section #8 was defined as the middle reach and it covers the stretch of river downstream of the bend with mild slope. Whereas the river stretch downstream of Section #8 is defined as downstream reach and the reach includes the river cascades. The sub-reaches including measurement sections are shown in Photo 5-1.

A detailed description of the flow and bed load transport patterns observed during the existing situation tests are presented in this section. The results are interpreted along the above defined sub-reaches. Comparative graphs showing water level and velocities for all selected steady flows along the river thalweg are shown Figures B-3 and B-4, Appendix B respectively. The water levels measured for the various steady flows are presented in Table C- 5 to Table C-11, Appendix, C. Likewise, the measured velocities are presented in Table C-12 to Table C-17, Appendix C. The videos of flow scenarios and sediment deposition pattern taken during the tests are presented in Appendix E. Colourful boulders with predefined sizes were used to observe the boulder transport direction and capacity of the river at different flow conditions.

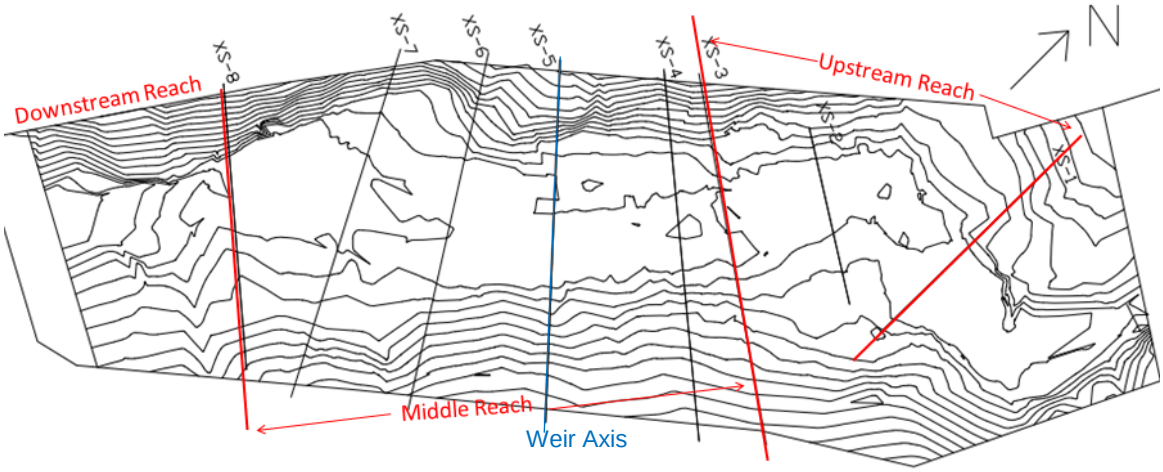

Photo 5-1: A plan view of the River Model showing the river reaches classified to describe the results of the Existing Situation Test

a) Upstream reach (Upstream of the Intake)

This reach consists of two simultaneous river bends located at the upstream end of the model. The river stretch upstream of the first bend (upstream one) is steep and used for flow stabilization, to capture the natural river flow condition. The area of interest for the model study starts just after the first bend (as shown in Photo 5-1). So, the upstream reach is defined as the river stretch from Section #1 to Section #3 as shown in the Photograph D-4, Appendix D

This reach is characterized by a large sharp bend on the left bank between Section #1 and Section #3. During average monsoon flow and annual flood, super critical flow with

standing waves dominated the river reach due to steep river gradient. Most of the boulders at the reach were partially submerged. At two years flood and five years flood, the boulders started to submerge fully and intensity of turbulence increased with some weak jump formation. Whereas at 20 years and higher floods flow with high waves splashed the river banks at the outer bend. Photograph D-5 to Photograph D-11 in Appendix D show the flow condition at the upstream reach for the various steady flows.

The sharp bend and nose cliff at the right bank between Section #1 and #2 caused abrupt change in flow direction and the flow between Section #2 and Section #3 was concentrated towards the left bank at all flows. During average monsoon flow and annual flood, impact of flow surge was observed on the right bank creating a small hydraulic jump just upstream of Section #2. At two years and five years flood, the jump evolved into a prominent skewed hydraulic jump. Whereas at twenty years and higher floods, a high velocity shooting and undular flow impacted the right bank at the bend which pushed the hydraulic jump further downstream of Section #2.

The effect of river bend in the flow was clearly captured by the velocity measurements, up to 5 years flood, along the reach. At Section #1 higher velocity was observed at the left bank which is located at the outer bend. Similarly, at Section #2 flow momentum shifted to the right bank due to the outer bend effect of the second bend, resulting in the higher velocity at the right bank. At floods higher than five years flood, flows along the river bends were highly turbulent because of hydraulic jump formation. So, velocity measurement was difficult due to high intensity of turbulence.

Sediment deposition at the upstream reach was mostly governed by effects of the river bends. At the first bend, upstream of Section #1 sediment deposition was observed on the right bank (inner bend) during average monsoon flow and annual flood. Similarly, at the second bend, between Section #1 and #3, sediment deposition was observed on the left bank (inner bend). The deposition however started to erode at two years flood. Boulders of size 0.5 m and 0.8 m size were transported, along the reach, by average monsoon flow and annual flood respectively. At higher floods, boulder as big as 2 m were easily transported along the reach. The boulder movement was also observed along the left bank, between Section #2 and Section #3, due to the flow deflection caused by the river bend, which can be observed in Photograph D-12, Appendix D.

b) Middle reach

This reach covers the stretch of Ghunsa River from Section #3 to Section #8, which is located downstream of the river bend. The proposed headworks lies within this reach. The river here has a mild curvature and mild slope. As a result a higher terrace deposit is located at the right bank (inner bend) between Section #4 and Section #5. The river gradually widens from Section #3 to Section #4 and is then narrowed from Section #4 to Section #5 by the terrace in the right bank. The proposed intake is located on the terrace. At Section #4 a large boulder (10 m - 16 m diameter) is located in the middle of the river. Moving downstream the river again gradually expands from Section #5 to Section #7 and then starts to constrict from Section #7 to a very narrow at Section #8 (Shown in the Photograph D-13, Appendix D).

The approach flow was directed towards the left bank due to river bends located at the upstream reach. As a result all flows from Section #3 to #4 were concentrated along the left bank. During average monsoon flow and annual flood, super critical flow with standing waves dominated the river channel with higher velocity readings at the right bank. The high velocity flow approaching the reach impacted the large boulder at Section #4 got deflected around the boulder. At the two years flood, the flow started shooting over the boulder. The intensity of the shooting flow increased at higher flows. Eventually, the boulder got fully submerged at fifty years and higher floods. Constriction of the river width by the terrace accelerated the flow from Section # 4 to Section #5, thus creating a

weak hydraulic jump in front of weir axis (at Section #5). Velocity readings here are almost similar at both banks. The intensity of the jump increased with the increase in flow. The high terrace started to submerge from 20 years flood onwards.

The river from Section #5 to Section #7 represent downstream part of the river curvature and the flow at this stretch was directed from left bank to right bank. At average monsoon flow and annual flood, calmer flow with lower standing waves were observed from Section #5 to Section #6, due to gradual widening of the river. However, from Section #6 to #7 flow was accelerated due to a steeper topography, which is reflected in the velocity reading as well. At higher flows the turbulence increased and boiling flows with standing waves were observed from Section #5 to Section #7.

The river topography takes a sharp turn from Section #7 to Section #8 and the river gets narrow downstream of Section #8. As a result, the flow was sharply turned from the right bank to the left bank. This phenomenon is reflected in the velocity readings at both sections showing higher velocity along right bank at Section #7 and higher velocity along left bank at Section #8. The narrowing of the river width and the sharp turning of the flow created a backwater effect up to point upstream of Section #8 at all flows. The interaction between the accelerated flow from Section #7 and back water effect created a hydraulic jump just downstream of Section #7 at all flow scenarios. Photograph D-14 to Photograph D-20 in Appendix D show the flow condition at the upstream reach for the various steady flows.

At average monsoon flow and annual flood, due to back water effect of the large boulder and river flowing on either side of the boulder, sediment deposited both upstream and downstream of the large boulder. The sediment deposition was uniform from Section #5 to Section #6 due to flow retardation. The deposition however started to erode at two years flood. The boulders of size up to 0.5 m and 0.8 m were transported from the upstream reach and got deposited between Section #5 and #6 at average monsoon flow and annual flood respectively. At 5 years and higher floods boulder of 2 m in size were easily transported along the headworks area of the river.

c) Downstream Reach

The downstream reach is located downstream of Section #8. This reach is characterized by a steep fall. So, shooting flow was observed during all flow conditions.

6 CONCEPTUAL STUDIES

Objective of conceptual tests of the headworks model is to achieve a headworks design that would satisfy the general performance standards set for the project in terms of hydraulics and sediment transport. The general headworks performance standards include:

- Safe passage of floods, including design flood situation
- Safe passage of bed load
- Safe passage of trash and floating debris
- Bed control at the intake
- Exclusion of suspended sediments

6.1 Representation of given headworks structure, base case

The headworks model was constructed on the basis of design and drawings provided by the Client. These drawings are presented in Drawings A-2 to A-6, Appendix A. The headworks structures were fabricated and were then placed at the designed locations in the river model, after completion of the existing situation tests.

A view of headworks structures placed in the river model is shown in Photo 6-1 below:

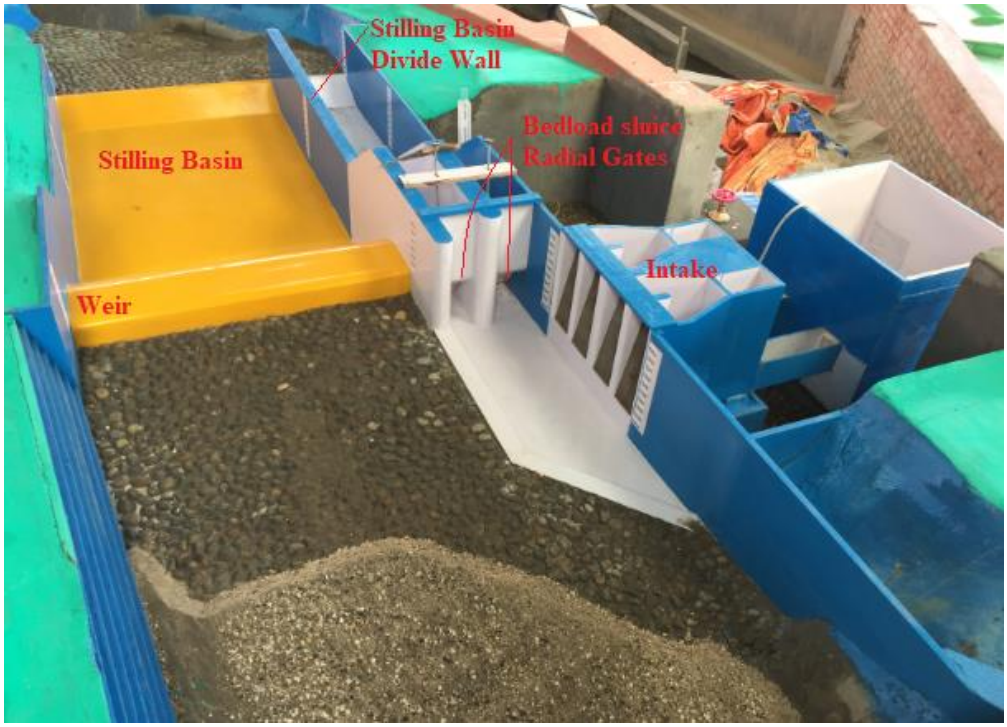

Photo 6-1: A view of the headworks model representing the base case design

6.2 Preliminary testing of base case design

Preliminary tests of the base case design were carried out for various steady flows to assess the overall performance. The selected flows and the corresponding sediment feeding rates applied in the model are presented in Table 2-3. The power plant was considered to be operational till 2 year's return period flood. When the Intakes were at open condition, bedload sluice gate was operated to maintain water level above 1947

masl and for controlling the bedload in front of the intakes, as outlined in the design operation rules.

6.3 Test observation of base case design

The following observations were made during the tests

a) Approach flow

- Approach flow was dominant towards the weir at all flows because of the flow deflection caused by the natural bend and nose cliff at the right bank upstream. However, at average monsoon and lower flows, the flow is again diverting towards the intakes as most of the flow is passing through the intake and bedload sluice gates (as shown in Photo 6-2). This resulted in strong secondary currents in front of the intakes.
- Intakes were closed during the 5 years and higher floods. At these conditions, flow was concentrated towards the weir keeping the intakes in a shadow flow zone. Hence, the larger boulders were transported towards the weir whereas only smaller boulder (below 0.5 m) were transported towards the intakes. Approach flow condition at various floods are shown in Photograph D-23 to Photograph D-26 in Appendix D.

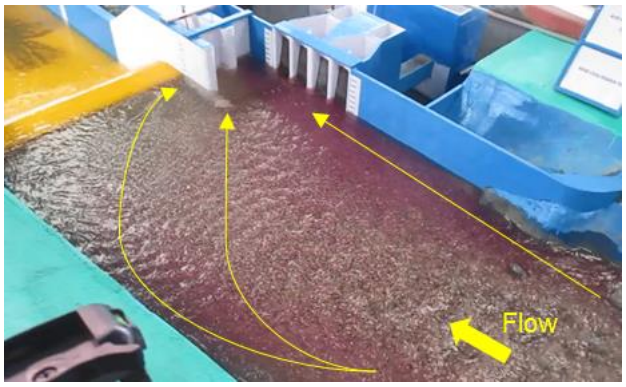

Photo 6-2: Approach flow during average monsoon flow

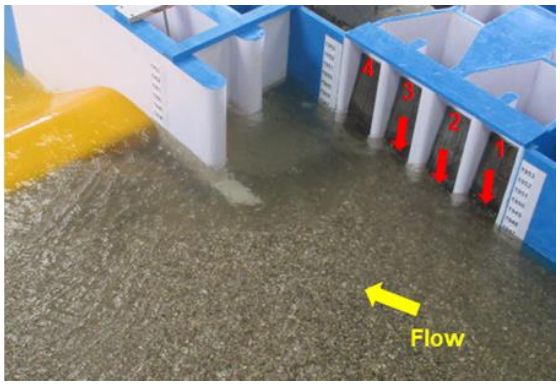

Photo 6-3: First three intakes completely chocked with bed deposition (Avgerage monsoon flow)

b) Bed control in front of the intakes

- Bed control in front of the intakes was not achieved during the operation of the intakes. As a result, the bed built up over time up to the sill level of intake and clogged the trash racks. Sediment also entered through the intakes into the approach canal.
- The left bedload sluice gate was only opened by 0.75 m to maintain the water level at 1647.00~1647.20 masl during average monsoon flow. At this condition, the sediment deposited up to intake gate #4. Hence, the trashracks of three upstream intakes opening (gate number #1, #2, & #3) were completely clogged, drastically reducing water abstraction capacity of the intakes (as shown in Photo 6-3).

c) Flushing of gravel traps

- The vortex tube type flushing system provided at the gravel traps could only flush gravels at intake gate number #4 and downstream half of gate number #3 (as shown in the Photo 6-5). The suction capacity for the remaining length was not sufficient to flush the deposited gravels.

Photo 6-4: Scenario inside the upstream gravel trap during average monsoon flow

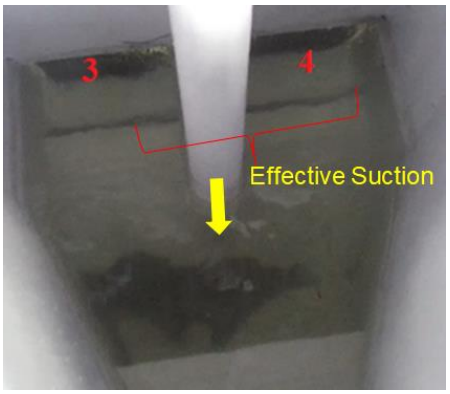

Photo 6-5: Effective suction length of gravel flushing system during average monsoon flow

- The gravel traps were deemed very close to the intake openings for efficient trapping. Lots of gravels, brought into suspension by the turbulence in front of the intakes, continued to be transported in suspension further downstream of the gravel traps and settled in the approach canal.

d) Trash Passage

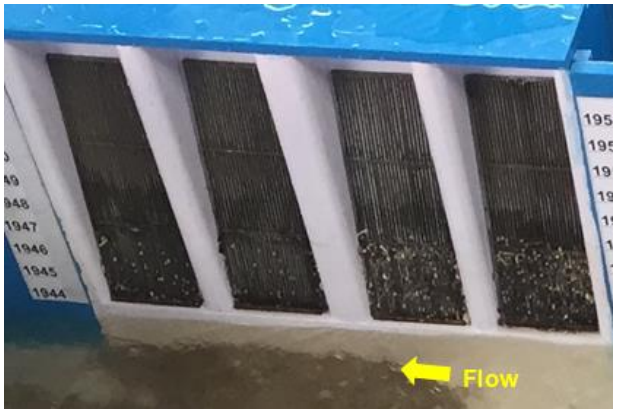

Photo 6-6: Clogging of trash rack opening

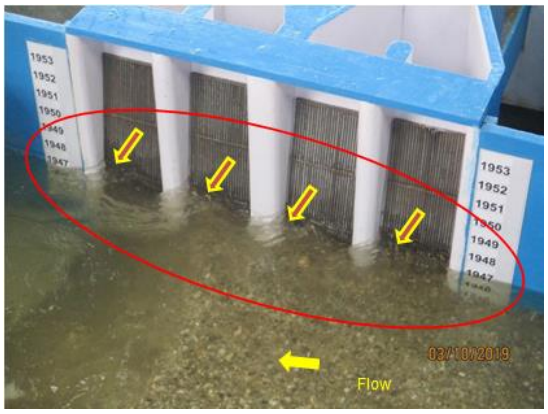

Photo 6-7: Velocity head observed at the piers of the intake

- Since trash passage facility was not provided in this design, the trash racks were frequently clogged by the floating debris. Thus, requiring regular mechanical cleaning.

e) Hydraulics in front of the Intake

- High velocity approach flow, parallel to the intake, struck the upstream face of the intake piers resulting in higher water levels at the areas of impact (as showing in Photo 6-7). This phenomenon resulted in high turbulence around the piers reducing the effective area of the intakes.
- The combination of upstream vertical face of the weir and lower water level in front of the intakes created a secondary current at upstream face of the weir, from the left bank towards the intake on the right bank (as shown in Photo 6-8). An increase in water level could be observed at the left face of the divide wall as a result of impact from the secondary currents (as shown in Photo 6-9). This phenomenon created highly turbulent and rotational flow in front of the intakes, bringing a lot of sediment deposits into suspension.

Photo 6-8: Rotational flow around the divide wall during average monsoon flow

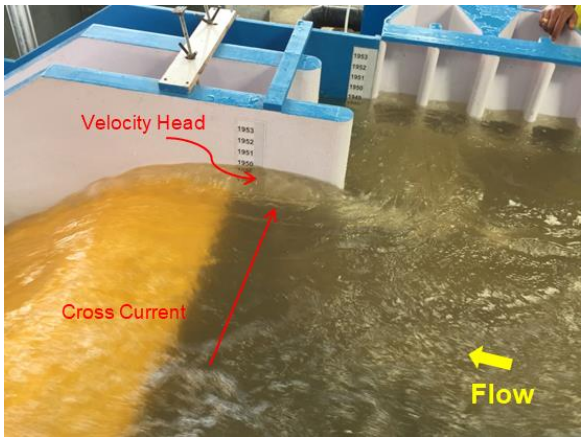

Photo 6-9: Rise in water level at the face of divide wall due to secondary currents during 5 years flood

f) Flood Passage and Energy Dissipation

- During floods, the flow was concentrated towards the weir and the gabion wall at the left bank, due to the flow deflection caused by river bend and a cliff nose located upstream of the headworks.
- Gabion wall may not withstand the impact of such concentrated flow. Hence, a concrete wall is recommended instead.

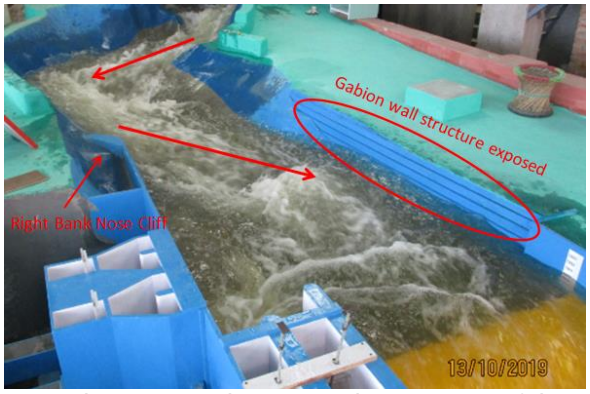

Photo 6-10: Flow scenario upstream of the weir axis during 100 years return period flood with both Radial gates open

Photo 6-11: Flow scenario in the stilling basin at 100 years return period flood with both bedload sluice gates open

- By operating both bedload sluice gates hydraulic jump was formed inside the stilling basin at all floods including the design flood (100 year flood), dissipating the hydraulic energy from the weir. Thus headworks is able to safely pass floods of magnitude at least up to design flood. Floods of 50 years and higher required full opening of both the bedload sluice gates.
- A scenario at which one bedload sluice gate failed to open (n-1 gate open) during floods was also tested. Hydraulic jump formed within the stilling basin at floods upto 50 years return period. However, during 100 years return period flood, a shooting flow was observed in the stilling basin and the hydraulic jump formed further downstream, failing to safely pass the flood.

Photo 6-12: Flow scenario at 100 years return period flood with only one bedload sluice gate open

Photo 6-13: Supercritical flow downstream of end sill during 2 years return period flood

- Supercritical flow occurred downstream of the end sill of stilling basin, at 2 years and higher floods, creating an unfavourable hydraulic conditions for stability of boulder riprap structures (as shown in Photo 6-13).

g) Bed load flushing test

- Bed load flushing test was carried out at low monsoon flow (45 m^3/s) for clearing sediment deposited in front of the intake. Both bedload sluice gates were opened to flush the sediment. When both gates were fully opened, a free gravity flow occurred and the sediment deposits in front of the intake were cleared (as shown in Photo 6-14).
- During flushing the major flow momentum was observed to be in a curved path moving from left bank towards intake.

Photo 6-14: Bedload flushing operation at Low monsoon flow

6.4 Shortcomings of base case design

After the base case design tests, the following shortcomings were observed in the model:

- Inability of the bedload sluice to control the bed deposits in front of the intake, while it is in operation.
- Intakes were clogged by the bed deposit and were unable to extract the design discharge for power generation.
- Unsatisfactory performance of gravel traps flushing system.
- Lacking in trash passage facility resulting in clogging of trash racks by floating debris.
- Unfavourable hydraulics in front of the intake due to secondary current along bottom of the weir moving from left bank towards intake on right bank.

- The impact of concentrated flow during the floods threatened the structural stability of gabion wall at the left bank, upstream of the weir.
- Stilling basin was unable to dissipate the design flood at scenario in which one bedload sluice gate is in inoperable condition. (n-1 gate operation).
- Supercritical flow downstream of the end sill of stilling basin was unfavourable for the stability of boulder ripraps.

6.5 Discussion on the Base Case

After completion of model tests of base case design, a meeting was held on 17th Oct 2019 at HLPL premises, with the Client to discuss the model observations. The Consultant briefed the main concerns and shortcomings in the base case design (outlined in Section 6.4) to the Client. The meeting also focused on some of the possible modifications on the base case design. It was decided to carry out conceptual modifications test with the introduction of undersluices beneath the intake structure for better bed control. It was also decided to implement inclined slope (1:2) at upstream face of the weir to minimize secondary currents and to alleviate sediment transport over the weir.

Hence further modifications were carried out focusing on shortcomings observed in the base case design.

6.6 Conceptual tests with modifications

Several modifications were implemented, at a conceptual level, and tested for determining their ability to improve the shortcomings concerning bed control, hydraulics at intake and energy dissipation. The summary of observations from the conceptual tests is presented as follows:

- Bed control in front of the intakes was improved by introduction of undersluices (Shown in Photograph D-32 in Appendix D).
- Hydraulics in front of the intakes was greatly improved by the implementation of inclined upstream face of the weir (Shown in Photograph D-33 in Appendix D).
- With the introduction of undersluice operation and incline upstream face of the weir no cross current flow across the weir was observed, however rotational flow still present in front of the intake (Shown in Photograph D-34 in Appendix D).
- The reduction of width of the outlet of the stilling basin by 5 m with the introduction of constricting groin wall of 2.5 m width inward on each side contained the hydraulic jump within the stilling basin at design flood, even when both bedload sluice gates were closed (Shown in Photograph D-35 in Appendix D).
- Alternatively, the lowering of stilling basin floor by 1 m also contained the hydraulic jump within the stilling basin at design flood, even when both bedload sluice gates were closed (Shown in Photograph D-36 in Appendix D).

The above discussed conceptual modifications improved the performance of the headworks. So, they were recommended for designing and implementation in the model for more detailed testing.

6.7 Recommendation on Major modification

Despite improvement observed from the implemented modification in Conceptual tests, the following issues were still prevalent:

- Unsatisfactory performance of gravel traps flushing system.
- Lacking in trash passage facility.
- Structural stability of gabion wall at the left bank was questionable due to direct impact of the flow momentum.

Hence, based on the model test observations of Base Case design and conceptual tests with some modifications, following modifications to the headworks design were recommended.

1. The vertical upstream face of the weir should be replaced by inclined slope (1:2)
2. Gabion wall at the left bank should be replaced by concrete wall
3. Introduce undersluice opening of 1 m x 1 m beneath the intake structure to improve the bed control in front of the intake.
4. Two individual gravel trap hoppers should be introduced with two individual outlet channel.
5. Intake orientation should be changed to fit undersluice passages and accommodate gravel trap hopper
6. A free flow channel condition should be provided for outlet discharge of undersluices and gravel traps to ensure a strong suction capacity at the upstream opening.
7. Trash passage channel should be introduced with either flap gate control or slide gate control with a provision of a stoplog control.
8. To contain the design flood for (n-1) one radial gate inoperable condition, two modifications showed satisfactory results. Either of the two modifications mentioned below could be recommended.
    a. Reduce the exit length of the stilling basin sill by 5 m with the introduction of constricting groin wall of 2.5 m width inward on each side of the stilling basin walls.
    b. Lower the stilling basin floor level by 1 m to contain the design flood.

However, performance of these modifications should be verified by physical model testing.

7 MODIFIED DESIGN STUDIES

7.1 Modified Design Arrangement I (MDA-I)

Based on the recommendation from the conceptual tests of the modifications, the Client and the Consultant agreed to formulate a modified design layout. The modified layout is referred as 'Modified Design Arrangement I' (MDA-I). In the MDA-I sluicing gates provided in the base case design shall be only used for flushing and flood passage, hence here after they are referred as bedload flushing gates. The drawings of the New Modified Design Arrangement I are presented in Appendix A Drawing No: A7 to A10. A view of the new design of the headworks can be seen in Photo 7-1.

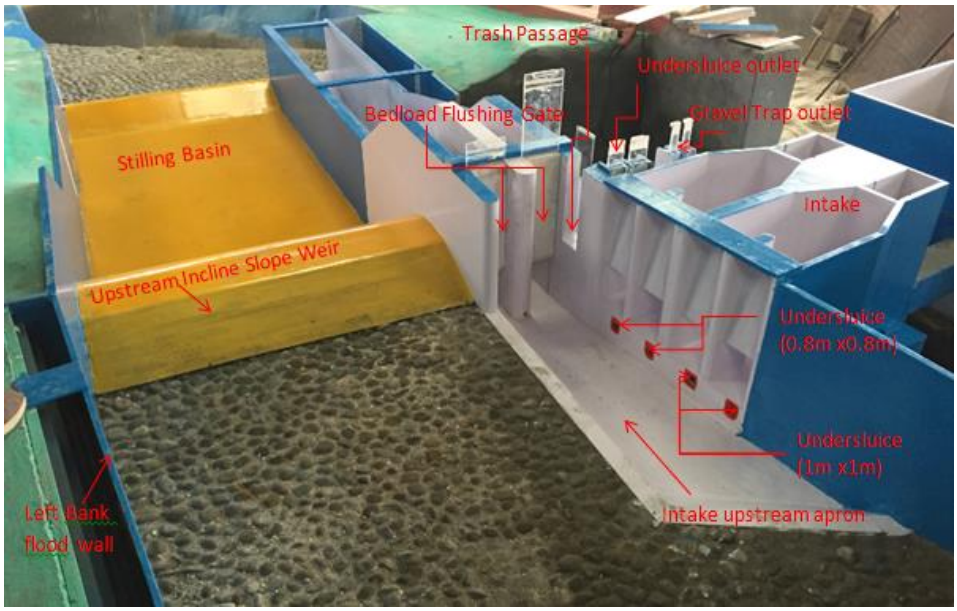

Photo 7-1: A plan view of the headworks structures with Modified Design Arrangement I (MDA-I)

The design modifications incorporated the following changes:

1. The vertical upstream face of the weir was made inclined with slope of 1:2 (H:V)
2. Gabion wall at the left bank, upstream of the weir, was replaced by a concrete wall.
3. Size of bedload flushing gates (formerly named as Bedload sluice in Base Case Design) was changed to 3.5 m X 4.5 m (W X H).
4. Four undersluice openings were introduced beneath the intake structure. Each of the two undersluices at upstream had opening size of 1 m x 1 m (W X H). Both of them discharged into a common conduit of size 1.5 m x 1.5 m (W X H). Similarly, the two undersluices openings at downstream were each of size 0.8 m x 0.8 m (W X H) and then they combined into a common conduit of size 1.2 m x 1.2 m (W X H).
5. Two individual gravel trap hopper was introduced, downstream of the intakes, with two individual outlet channel.
6. The location and orientation of Intakes were changed to accommodate undersluices gravel trap hoppers
7. The weir was shortened, at right bank, to 32 m to accommodate the change in intake orientation and bedload flushing gates.
8. Undersluices and gravel traps were discharged into an outlet channel having bed slope of 1:18, which then joins the channel in which the bedload flushing gates discharge.
9. A two and half meter wide trash passage having crest level at 1946.70 masl was provided on the wall between intake and bedload flushing gates. The flow from the

trash passage was discharged into the outlet channel for undersluice and gravel trap.

10. Invert level of the Intakes were lowered to 1946.60 masl to ensure sufficient submergence at the intakes.
11. Floor level of upstream apron for bedload flushing gates was lowered by 1 m to accommodate undersluice beneath the intakes.

7.1.1 Test observations

Preliminary tests of MDA-I were carried out for various steady flows to assess the overall performance. The following observations were made during the tests.

a) Approach flow and bed control in front of the intakes

- The modified orientation of the intakes created sediment deposition just upstream of the intakes. This deposition deflected the river flow away from the right bank and allowed the flow to approach the intakes from a more perpendicular direction. This improvement in the approach flow increased the effective area of the intakes compared to the base case design. Approach flow condition and bed disposition pattern are shown in Photo 7-2.

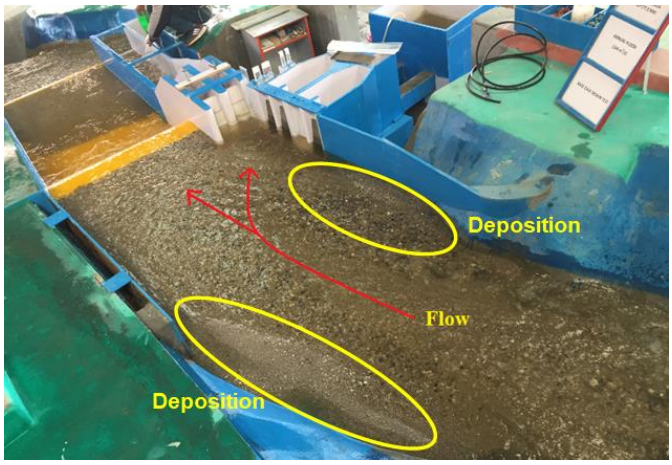

Photo 7-2: Approach flow during average monsoon flow for MDA-I

- Similar to the base case design, with the intake gates closed at 5 years and higher floods, the flow and boulders were discharged over the weir. Only smaller boulders below 0.5 m in diameter were observed at the intake area.
- Undersluices achieved bed control in front of the intake and the intakes remained fully operational. At the average monsoon flow, the right and left undersluice outlet gates were operated at openings of 0.75 m and 0.6 m respectively. Whereas, both undersluice gates were fully opened during annual and 2 years floods.

b) Hydraulics in front of the Intake and Gravel Trap

Photo 7-3: Difference in water level in front of Intake for MDA-I at average monsoon flow

Photo 7-4: Secondary compensation flow at the bottom traveling upstream for MDA-I at average monsoon flow

- Sediment deposition pattern and approach flow directed the momentum of flow towards the downstream intakes. As a result water levels at the downstream of intake were higher than the upstream of intake. So, higher discharge was abstracted through approach Tunnel #2 compared to Tunnel #1. (as shown in Photo 7-3)
- Higher water levels at the downstream gates further created a secondary flow at the bottom which was directed from the Intake Gate 4 towards Intake Gate 1. (as shown in Photo 7-4).
- The secondary flow agitated the sediment deposits at the bottom and entrained them into suspension. As a result significant amount of sediment entered through the intakes overloading the gravel traps at Gate #1 and #2.
- The modified gravel traps were still not able to clear the gravel deposits efficiently. Suction capacity of the gravel traps flushing especially in intake gate 1 and 2 was too low to clear the sediment deposit as highlighted in the Photo 7-5.

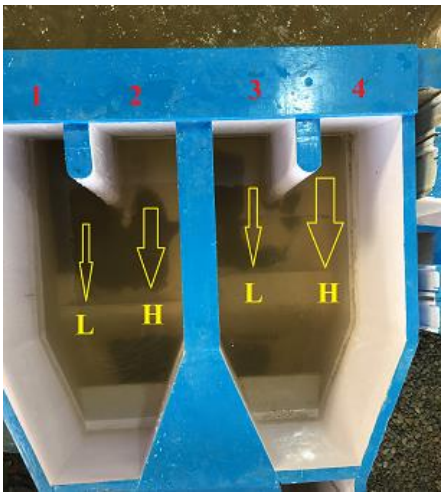

Photo 7-5: Uneven suction at the gravel trap during average monsoon flow operation

c) Sediment flushing channel

- Introduction of undersluices improved the bed control in front of the intakes. A free flow flushing channel provided at the outlet of the undersluices and gravel traps helped in improving the suction capacity of the undersluices. The size and the slope of the flushing channel were adequate to transport the sediments flushed by the undersluices and gravel trap.
- No backwater effect was observed in the flushing channel up to 10 years flood while the bedload flushing gates were closed. Backwater was only observed in the channel at 20 years and higher floods when the bedload flushing gates were opened to pass the flood discharge (Shown in Photograph D-37 and D-38, Appendix D).
- When the water level in front of intake was higher than 1948.00 masl (equal to or higher than the annual flood), discharge trough the trash passage gate was significantly high and started obstructing the flow in the flushing channel resulting in the submergence of the gravel traps outlet. Hence, to ensure free flow condition at gravel traps outlet, at annual flood and higher floods, trash passage gate should be closed when water level goes higher than 1948.00 masl.

d) Trash Passage

- The concentration of approach flow towards trash passage and downstream of intakes allowed removal of about 60 % of the floating debris through the trash passage and the remaining 40 % kept on rotating in front of the intake. (as shown in Photograph D-39, Appendix D)

e) Flood Passage and Energy Dissipation

- Up to 10 years flood the bedload flushing gates were closed and the discharge passing over the weir formed hydraulic jump within the stilling basin. However, above 10 years flood, bedload flushing gates had to be opened to contain the hydraulic jump within the stilling basin.
- For 50 years and higher floods, both bedload flushing gates had to be fully opened for safe flood passage. Stilling basin was incapable of containing hydraulic jumps at these floods when one bedload flushing gates was not in operation (i.e. n-1 scenario). The performance of the stilling basin is summarized in the Table below.

Photo 7-6: Unsatisfactory stilling basin performance at 50 years flood with one bedload flushing gates Open condition

Table 7-1: A summary of performance of stilling basin with respect to safe passage of floods

Operation of bedload flushing gates	Safe passage of flood			
	≤ 10 years flood	20 years flood	50 years flood	100 years flood
All gates closed	Yes	No	No	No
One gate open	Yes	Yes	No	No
Both gates open	Yes	Yes	Yes	Yes

7.2 Discussion and Recommendation

Introduction of undersluices achieved bed control in front of the intakes. The size and the slope of the flushing channel was observed to be adequate to transport the flushed bed load from the undersluices and gravel traps. Introduction of trash passage allowed removal of about 60 % of floating debris. Further modifications are required to improve the performance of passage of floating debris.

The following shortcomings with respect to the hydraulics in front of the intake, gravel flushing system and flood passage were still present:

- Secondary flow in front of the intakes entrained the sediment deposits into suspension which were then transported inside the intakes.
- Suction of the gravel trap flushing at intake 1 and 2 was too low to clear the sediment deposit.
- Stilling basin was unable to contain hydraulic jump of flood above 50 years return period when one bedload flushing gates was not operating (i.e. at n-1 gate operation scenario)

After completion of model tests of "Modified Design Arrangement I" (MDA-I), a joint meeting was held on 20th Dec 2020 with the Client at HLPL premises, to discuss the model observations and further modifications for addressing the observed shortcomings.

It was decided to carry out further studies to improve hydraulics in front of the intake, gravel flushing system, trash passage and stilling basin for energy dissipation. Hence based on the above tests observation the following modifications to the headworks design were recommended.

1. Introduce hopper type gravel trap for individual intake with a provision of individual conduit.
2. Introduce an overflow trash passage channel just upstream of the bedload flushing gates to improve hydraulics in front of the intake
3. Lowering of stilling basin floor level by 1 m or reduction of width of the outlet of the stilling basin.

However, due to time constraint and model reconstruction restrictions, both parties agreed that hopper type gravel trap with individual conduit would improve the gravel flushing system. Hence, it would not be tested in the model. Similarly, conceptual tests of reduction of the stilling basin exit were successful in achieving safe passage of floods. So, they will only be implemented in the model for the final documentation testing. Other recommendations for the modifications will be tested in the model.

7.3 Modified Design Arrangement II

To reduce secondary compensation flow at the bottom traveling upstream; the surface flow momentum energy should be broken and need to ensure significant flow from upstream towards gate 1 and 2 to reduce the rotational compensation flow into the gate 1 and 2. The design modifications included the following changes:

- An overflow trash passage channel just upstream of the bedload flushing gates was introduced to avoid secondary flow and improved trash passage.
- The existing divide wall was extended upstream by 9.5 mwith top elevation at 1946.00 masl.

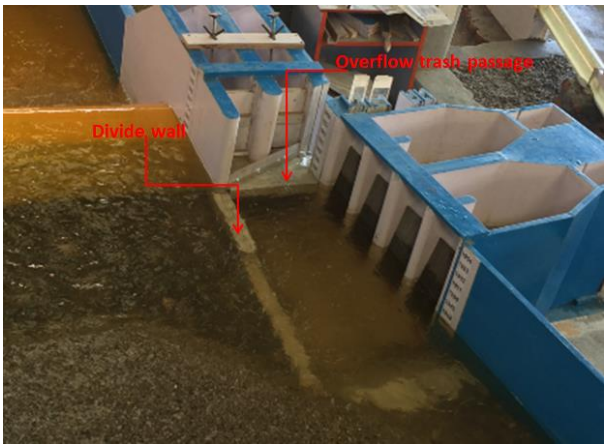

Photo 7-7: Overview of Modified Design Arrangement II

7.3.1 Test observations:

a) Hydraulics in front of the intake and trash passage

- The overflow trash passage channel upstream of the bedload flushing gates reduced the intensity of secondary flow which significantly improved the hydraulics in front of the intake.
- The improved hydraulics in front of the intakes also reduced the magnitude of entrainment of sediment into suspension. So, sediment loading in the upstream gravel trap was significantly reduced.

Photo 7-8: Reduction in the intensity of secondary flow for Modified Design Arrangement II

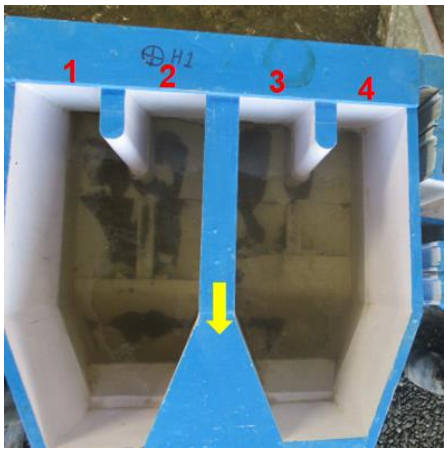

Photo 7-9: Reduction in sediment load at all intakes for Modified Design Arrangement II

- The overflow trash passage structure also significantly improved the removal of floating debris. However, the protruding intake piers created space pocket just in front of the intake where the floating debris started to accumulate. To further improve the trash removal, following modification were tested which showed further improvement.
  - A skirt wall in front of the intake helped to guide the floating debris towards the trash passage channel.
  - Vertical trash rack layout avoided the formation of space pocket thus, preventing accumulation of floating debris.

Photo 7-10: A skirt wall in front of intake for Modified Design Arrangement II

Photo 7-11: Vertical trashrack layout at intake for Modified Design Arrangement II

Discussion

The modifications improved the hydraulics in front of the intakes and the passage of floating debris. So, they are recommended for the implementation in the final layout of the headworks.

7.4 Modified Design Arrangement II with Settling Basins

Testing of the settling basins were carried out only after finalizing the layout of the intakes. The layout approach tunnel to the settling basins had to be adjusted to accommodate the changes in the intake orientation. Overall layout of Modified Design Arrangement II with settling basins is represented in Appendix A Drawing No. 10.

To fit the settling basins within the laboratory's working space, 45 m long portion of the straight approach tunnel was trimmed off for both setting basins. However, the slope of the approach tunnel was maintained same as in the design to simulate the hydraulics at the inlet of the settling basins correctly.

The settling basins were reproduced in the same model scale of 1:30. This scale is too small for testing the settling efficiency of the settling basins. Therefore, only hydraulic performance of the settling basins was tested to confirm that the flow patterns in the settling basins is conducive for settling phenomenon.

Photo 7-12: Layout of intake in Modified Design Arrangement II including settling basins

Test observations:

- Settling Basin #2 received higher flow than Setting Basin #1 due to concentration of approach flow at Intakes #3 and #4. Hence, to obtain equal flow distribution between the basins, the opening of outlet gate of Settling Basin #2 was reduced to 3 m. Meanwhile the outlet of Settling Basin #1 was fully opened.
- Almost uniform horizontal flow was observed throughout the length of the settling basin. (as shown in Photo 7-13)

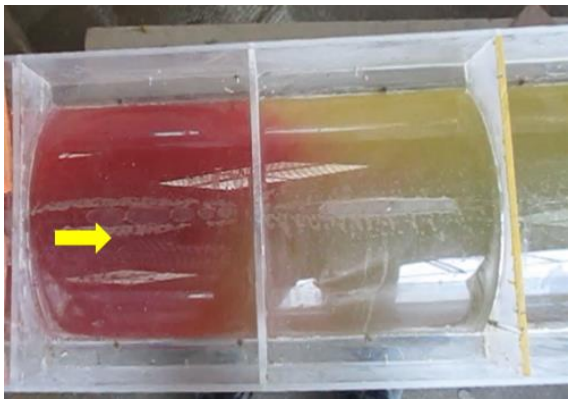

Photo 7-13: Uniform horizontal flow achieved at the end of the inlet transition of the settling basin

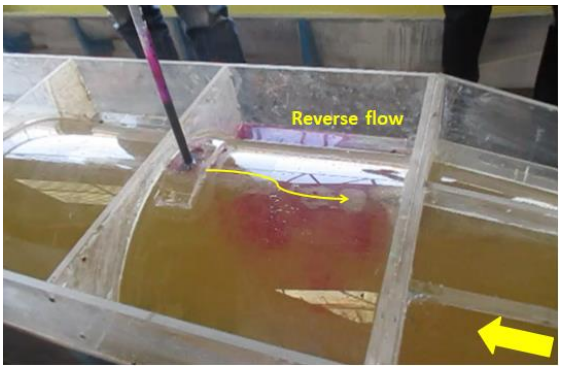

Photo 7-14: Reverse flow at the crown of the transit zone

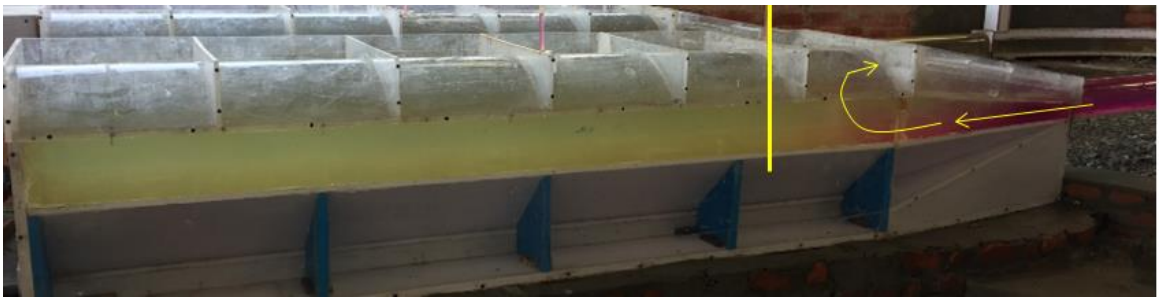

Photo 7-15: Velocity higher at the bottom just downstream ofthe transit zone of Modified Design Arrangement II Settling Basin

- Flow distribution along the depth of the basin was however non-uniform. At the end of the inlet transition, the flow was concentrated at the bottom of the basin (Shown in Photo 7-15)
- A rotational reverse flow was observed at the crown of the inlet transition zone due to non-uniform distribution of flow along the depth (Shown in Photo 7-14).

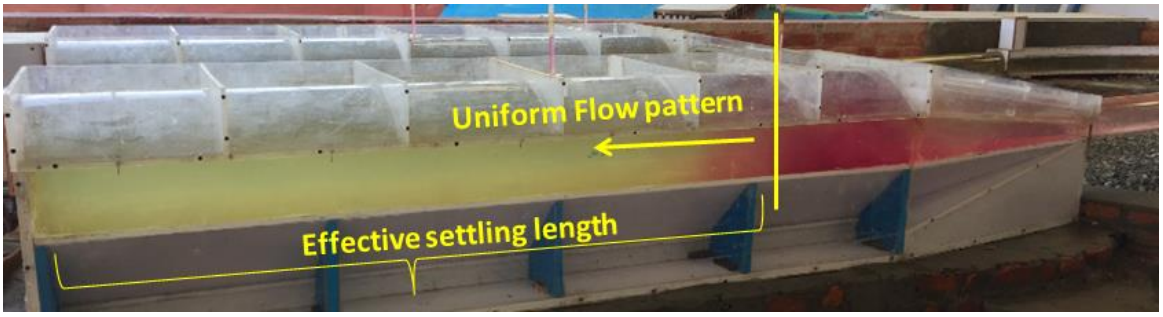

Photo 7-16: Effective settling length of the settling basin having uniform vertical and horizontal flow distribution

- A uniform flow in both vertical and horizontal direction was observed after 1/5 length of the settling basins at the upstream end. Therefore, the effective settling length of the settling basins was observed to be only 4/5 of the total length (Shown in Photo 7-16).
- Haunting oscillation of flow was observed as a result of steep inlet. (Shown in Photograph D-40, Appendix D).
- The abrupt change in flow area created a stagnant recirculating zone on both side of the settling basins at the outlet.

7.5 Discussion and Recommendation

Detailed model observation for Modified Arrangement II with settling basins was carried out. Based on the model observation, the following shortcomings in the settling basins were identified:

- Uneven flow distribution between the settling basins with Settling Basin #2 receiving higher flow
- Flow exiting from transit zone was observed to have varying vertical velocity distribution, resulting in reverse compensation flow at the crown of the settling basins.
- With varying velocity distribution in vertical direction, effective length of the settling basins was reduced to 80 % of the total available length.

- Haunting oscillation of flow was observed as a result of inlet transition of the settling basins.
- Stagnant recirculating zone observed on both side of the settling basins due to abrupt change in flow area at the basin outlet.

After completion of settling basins tests, a joint meeting was held on 5th Feb 2020, with the Client at HLPL premises to discuss the results of model observations.

To address the above shortcomings, the following modifications were recommended in the settling basins.

- To obtain equal flow distribution, outlet gates of Settling Basin #2 and Settling basin #1 shall be opened to 3 m and 3.9 m respectively. Furthermore, the length of flow path between both basins had to be made equal to ensure equal head loss. So, the adjustment of the outlet tunnel layout is recommended as shown in Photo 7-17.

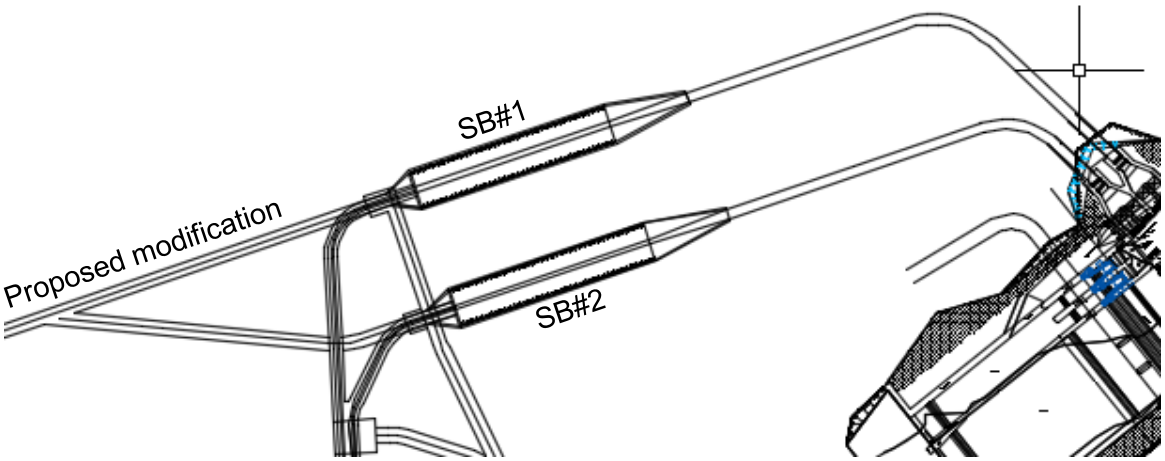

Photo 7-17: Settling basins layout option with increased flow exit path for Settling Basin #2

- Introduce a straight and flat identical approach tunnel (with 0 % slope) at the entrance of the settling basins to reduce concentration of flow near the bottom of the transition zone
- Reduce angle of inlet transition or increase the size of the approach tunnel for improving hydraulics at the entrance of the basins.
- Introduce an exit transition for smooth flow exit path to avoid stagnant recirculation zone at the exit of the basin.

However, due to time constraint, both parties agreed that the recommendation discussed will be implemented in the final design of the settling basins, but will not be tested in the physical model.

8 FINAL HYDRAULIC DESIGN

After completion of model tests of Modified Arrangement II including settling basins, both parties agreed that the performance of the modified designs were satisfactory. Hence, the headworks design labelled “Modified Arrangement II” incorporating the suggestions mentioned in above sections (Section 7.3 and 7.4) will be considered as the Final Recommended Hydraulic Design (FRHD) which is presented in Appendix A, Drawing A11 to A16. The FRHD implemented in the model is shown in Photo below. The components of FRHD are described below.

Photo 8-1: A plan view of the headworks with final recommended hydraulic design (FRHD)

Table 8-1: Structure features of the project Initial Base Case Design and Final Recommended Design

Description	Initial Base Case Design	Changes made in Final Recommended Design
Diversion weir		
Length	35 m	32 m
Crest elevation	1947.00 masl	
Bedload sluice		**Bedload Flushing Gate**
Number of openings	2 nos. (with radial gates)	
Size (W X H)	4.5 m x 4.5 m	3.5 m (W) x 4.5 m (H)
Floor elevation	1940.25 masl	1939.25 masl
Undersluice		
Number of openings		4 nos. (with two control gates)
Size (W X H)		U/S two undersluices → 1 x 1 m D/S two undersluices → 0.8 x 0.8m
Intake Structures		
Type	Side Orifice	
Number of openings	4 nos.	

Description	Initial Base Case Design	Changes made in Final Recommended Design
Size of opening (W X H)	3.5 m x 3.1 m	
Invert level of intake	1941.00 masl	1943.1 masl
Number of trash racks	4 nos.	
Size (W X H)	3.5 m x 3.5 m	
Gravel trap		
Numbers	1 no.	2 Hopper type design
Size (W X H X L)	0.5 m X 0.5 m X 17.5 m	0.8 m x 0.8 m
Size of flushing channel (L X W X H)	40.7 m X 0.5 m X 0.5 m	7.4 m width Open channel
Connecting approach tunnel		
Number	2 Nos.	
Type	Inverted D-shaped, Pressurized	
Size (W X H)	3.1 m X 3.1 m	3.9 m x 3.9 m
Length and Slope (Tunnel 1)	155.60 m, 1 in 36	138 m, 1 in 32
Length and Slope (Tunnel 2)	122.60 m, 1 in 28	118 m, 1 in 27.5
Settling basins		
Type	Underground, pressurized	
Number of bays (Caverns)	2 Nos.	
Size of Caverns (L X W X H)	62 m X 11.5 m X 14.65 m	
Flushing operation model	Continuous	

Diversion weir and stilling basin

The diversion weir is a 32 m long ungated concrete gravity structure with its crest elevation at 1947.00 masl. The upstream and downstream faces of the weir are provided with slopes of 1:2 and 1:1.5, respectively. A boulder rip-rap apron is provided upstream of the weir to protect the riverbed from scouring. The floor level of the boulder rip-rap is provided at an elevation of 1944.00 masl. A concrete flood wall with top elevation at 1955.00 masl is provided at the left bank upstream of the weir.

Downstream of the weir, a 53 m long and 32 m wide stilling basin is provided to dissipate the energy of flow over the weir before releasing into the river downstream. The floor level of the basin is provided at 1935.75 masl and the end sill level at 1938.75 masl. The outlet width of the basin is reduced by 4 m by introducing a constricting wall of 2 m width inward on each side walls of the basin.

Bedload flushing gates

Two 3.5 m wide bedload flushing gates, are provided at the right bank, adjacent to the weir, for flood passage and periodic flushing of sediment deposit from the upstream pool. Each bedload flushing gates is provided with a 3.5 m wide and 4.5 m high radial gates for the operation. A 9.5 m long divide wall with top elevation of 1946.00 masl is provided between weir and bedload flushing gates. The bottom sill of the bedload flushing gates are provided at 1939.25 masl. Downstream of the radial gates, a 53 m long stilling basin is provided for dissipation of energy of flow through the gates before it is released into the river.

Intake, undersluice and gravel trap

Two river intakes, one for each settling basins, are provided at the right bank at an angle of 98.35 degree with respect to weir axis. Each intake has two bays with trashrack

openings, each 3.5 m wide and 3.5 m high, that are separated by intermediate piers. The sill of the intakes is provided at 1943.10 masl and the crown level at 1946.60 masl. Four gravel trap hoppers, each for an intake bay, are provided downstream of the intakes. Two flushing conduits, each serving two gravel traps, is provided to flush the sediment deposited in the gravel traps. The floor of the intake structure slopes down after the gravel trap from 1943.10 masl to 1941.00 masl to ensure adequate water submergence for creating pressure flows in the connecting approach tunnels.

Provision of stop logs are made at each intake bays for accessing the intakes for maintenance. Similarly, emergency and service gates are also provided in each intake. Air vent pipes are provided behind the gates to prevent their vibration.

Four undersluice openings are introduced beneath the intake structure for bed control. The two undersluice orifices at upstream have openings of size 1 m x 1 m (W X H) and both discharge into a common conduit of size 1.5 m x 1.5 m (W X H). Similarly, opening size of two downstream orifices is 0.8 m x 0.8 m (W X H) which discharge into a common conduit of 1.2 m x 1.2 m in size. Gates are provided at the outlets of the conduits from undersluice and gravel traps for regulation. Flow from the undersluices and gravel traps discharges into a 7.4 m wide flushing channel with bed slope of 1:18. The flushing channel is then discharged into the stilling basin of the bedload flushing gates part.

Trash passage and trash racks

Trash racks are provided at openings of each intake bays to prevent floating materials from passing through the intakes. The trash racks are positioned at an angle of 80° to the horizontal and placed in the abutments/piers. A trash rack cleaning machine (TRCM) with a boom is provisioned for cleaning the trash that is accumulated in front of intakes. In addition, an overflow trash passage channel provided just upstream of the bedload flushing gates for safe passage of floating debris accumulated in front of the intakes. The channel is provided in perpendicular direction to the flow and the top elevation of the wall of the channel facing the flow is set at 1947.00 masl. The floor level of the channel is provided at 1946.50 masl. The channel discharges into the undersluice and gravel trap flushing channel through a 2.5 m wide trash passage outlet opening provided on the wall between intake and bedload flushing gates. The sill level of the outlet is provided at 1946.50 masl. The outlet is equipped with a 2.5 m wide vertical slide gate for the regulation.

Settling Basins

Two connecting approach tunnels, one from each river intake, are provided to convey water to the underground settling basins. The lengths of connecting approach tunnels #1 and #2 are about 138 m and 118 m, respectively. The tunnels are D-shaped with a finished size 3.9 m wide and 3.9 m high, and their bed slopes are 1 in 32 and 1 in 28, respectively. At least 35 m length of the approach tunnels upstream of the inlet transition are straight without any longitudinal slope, to avoid concentration of flow at the bottom of the transition zone. The bottom floor level of these tunnels are provided at 1936.70 masl.

Two underground settling basins each 62 m long and 11.5 m wide are provided 25 m apart. A 25 m long inlet transition is provided which expands from 3.9 m at the terminal point of the connecting approach tunnel to 11.5 m at the initial point of the chamber. Similarly, an outlet transition of at least 15 m length is recommended at the end of the basin. The flow from the settling basins is collected by outlet tunnels of size 3.9 m wide and 3.9 m high which then join with a headrace tunnel. The tunnels are D-shaped but their lengths are varied to create same head loss up to the starting point of the headrace tunnel. The length of the outlet tunnels from settling basin #1 and settling basin #2 are 33.5 m and 61 m respectively.

9 FINAL DOCUMENTATION STUDY

Objective of final documentation study is to carry out a comprehensive test program for documentation of the Final Recommended Hydraulic Design (FRHD). The final selected headworks arrangement was tested for the following features during this study:

- Hydraulic performance
- Floating debris transport patterns
- Bedload transport patterns
- Energy dissipation
- Operation regime of the intake and settling basins

The tests were conducted for the flow situations with corresponding bedload feed rates shown in Table 5-1 and Table 2-3 respectively. Additional five measurement sections parallel to the weir axis were introduced near weir and stilling basin for conducting the measurements. These additional sections were marked at XS - #UI (upstream of intake), XS - #I (At Intake), XS -WA (along Weir axis), XS - MSB (middle of stilling basin) and XS - ES (end sill of stilling basin), which are shown in Drawing A-17, Appendix A. For sediment deposition pattern study, cross sectional data of the bed deposition were surveyed after various steady flows on number of sections which are presented in Figure B-13 to B-16, Appendix B

Findings of the final documentation studies are presented in subsequent sections. Average water depths observed along the river for all steady flows are presented in Figure B-5 to B-12, Appendix B and data are tabulated in Table C-18 to C-24, Appendix C. Meanwhile bed deposition levels are presented in Figure B-13 to B-16, Appendix B and data are tabulated in Table C-25 to C-28, Appendix C. In addition 3-dimensional DEM of sediment depositions in the model were reconstructed using Structure from Motion (SfM) technique. The accuracy of the DEM was 6 cm in the prototype. Results of analysis from 3-dimensional DEM is represented in Photograph D-75 to D-76, Appendix D. Videos taken during the tests are presented in Appendix E.

9.1 Average monsoon flows

During average monsoon flow, most of the river flow was directed towards the intake portion at the right bank, whereas only a small amount of flow was discharged over the weir. The operation of different gates at average monsoon flows is presented in Table 9-1. A view of flow pattern at headworks during this flow is presented in Photograph D-41, Appendix D.

The river bend and cliff nose at the right bank located upstream of the intakes deflected flow towards the left bank (i.e. towards weir portion). However, since most of the flow was passing through the intakes and undersluices, the flow was re-directed towards the intake, located at the right bank, creating a curved flow path (as shown in Photograph D-41, Appendix D). Due to the curvature of the flow path, sediments deposited along both left and right bank upstream of the intakes, as shown in Photograph D-41 and Photograph D-42, Appendix D. This sediment deposition pattern is reflected in the bed level measurement data shown in Figure B-13 to 16, Appendix B.

The approach flow was mostly concentrated towards the intakes IG4 and IG3 as shown in Photograph D-41, Appendix D. Most of the floating debris was passed by the overflow trash passage structure. Some of the floating debris were trapped in front of the intake within the pocket created between the intake piers which would require mechanical raking (as shown in Photograph D-43, Appendix D). Due to concentration of flow

towards the downstream intakes, the water level at the intake IG4 was higher than the intake IG1. The difference in water level created a secondary flow at the bottom travelling upstream towards intakes IG2 and IG1 (Shown in Photograph D-44, Appendix D). However, the intensity of secondary flow was lowered by the overflow trash passage.

Sediment movement was uniformly distributed amongst the upstream 3 undersluices located at intakes IG1, IG2 and IG3 whereas there was very less sediment deposition in front of undersluice at intake IG4. Boulders of size up to 0.3 m were observed moving towards intake, which were easily flushed through undersluices.

Submerged hydraulic jump formed in the stilling basin at about 1 m downstream of the toe of the weir. The maximum water level of 1939.32 masl was recorded at middle section of the stilling basin. The flow downstream of the end sill of the stilling basin was super critical. Flow scenario in the stilling basin is shown in Photograph D-45, Appendix D.

The measured bed levels are presented in Table C-25, Appendix C. Similarly, the measured water levels are presented in Table C-18, Appendix C. Whereas, figures comparing the water levels measured during existing situation test and for the FRHD are presented in Figure B-6, Appendix B. Videos of the model tests during average monsoon flow is presented in Appendix-E.

Table 9-1: Details of gate openings maintained for Average Monsoon flow

Gates	Gates position	Remarks
Intake	Fully open	Design discharge and discharge for the flushing units is abstracted.
Undersluice (1 m x 1 m)	0.75 m open	
Undersluice (0.8 m x 0.8 m)	0.5 m open	
Gravel trap	Fully open	
Bedload flushing gates	Fully closed	
Trash Passage	Fully open	
Setting Basin #1 Outlet	Fully open	To ensure almost equal flow distribution amongst the settling basins
Setting Basin #2 Outlet	3 m open	

9.2 Annual flood

During annual flood, most of the river flow was directed towards the weir, whereas the required design flow and the discharge for various flushing was attracted towards intake structure. The operation of different gates at annual flood is presented in Table 9-2. A view of flow pattern at headworks during this flow is presented in Photograph D-46, Appendix D.

The river bend and cliff nose at the right bank located upstream of the intakes deflected flow towards the left bank. However, the sediment deposits along the left bank pushed flow towards the centre of the river channel. The flow further scoured the sediment deposits in the river bed forming a channel at the centre and high sediment deposition pattern emerged on either side of the river bank (as shown in Photograph D-46 to D-48, Appendix D). This sediment scouring and deposition pattern is reflected in the bed level measurement data shown in Figure B-13 to16, Appendix B and Photo D-77, Appendix D.

Bed level measurement clearly shows a significant scouring of sediment deposits in the river bed at the centre from sections #3 to #4. The scouring decreased from Section #4 to Section #UI flow retardation caused by the weir. With the flood kinetic energy reduced, no significant depletion of bed was observed from Section #UI to Section #5. Bed level

measurement clearly shows a significant increase in deposition on either banks especially on the right bank between Section #3 and #4.

At annual flood average water level in front of intakes was 1948.24 masl. Once the water level in front of the intake went above 1948.00 masl (During annual flood and above) the discharge from the trash passage overloaded the flushing channel resulting in a submergence of gravel trap outlet (Shown in Photograph D-50, Appendix D). Submergence of the outlets of undersluices and gravel trap reduce their suction capacity drastically leading to clogging. Hence, to avoid the submergence of the outlets of gravel traps and undersluices, the trash passage was closed once water level went above 1948.00 masl and operated intermittently.

Momentum of the approach flow was mostly directed towards weir and the intake and bedload flushing gates were located in the recirculating flow shadow area as shown in Photograph D-46 and D-47, Appendix D. Similar to average monsoon flow, secondary flow was still present in front of the intakes at the bottom travelling upstream towards Intakes IG2 and IG1. However, the intensity of secondary flow was significantly reduced by a shifting of flow momentum towards the left bank over the weir and insuring sufficient surface flow along the right bank delivering significant required flow to the discharge abstracted through intakes IG1 and IG2 (as shown in Photograph D-49, Appendix D). Most of the floating debris transported over the weir and those transported towards the intakes got accumulated in front of intake as the trash passage was in a closed position.

The scouring of the sediment deposits at the centre of river channel and deposition of sediment on along the river banks significantly increases sediment load to the undersluices. However, at full gate openings the undersluices were able to achieve bed control in front of the intakes. The sediment loading was also more uniformly distributed amongst all undersluices. Boulders upto 0.6 m in size were transported by the river. Most of the boulders including the larger ones, however, were directed towards the weir by the momentum of the flow (shown in Photograph D-46 and 47, Appendix D). The boulders of size up to 0.4 m were transported towards the intake which were flushed downstream by the undersluices.

Similar to average monsoon flow submerged hydraulic jump formed in the stilling basin but at about 2 m downstream of the toe of the weir. The maximum water level of 1940.33 masl was recorded at middle section of the stilling basin. The flow downstream of the end sill of the stilling basin was observed to be super critical. Flow scenario in the stilling basin is shown in Photograph D-51, Appendix D.

The measured bed levels are presented in Table C-26, Appendix C. Similarly, the measured water levels are presented in Table C-19, Appendix C. Whereas, figures comparing the water levels measured during existing situation test and for the FRHD are presented in Figure B-7, Appendix B. Videos of the model tests during annual flood is presented in Appendix-E.

Table 9-2: Details of gate openings maintained for Annual Flood

Gates	Gates position	Remarks
Intake	Fully open	Design discharge with discharge for the flushing units is abstracted.
Undersluices (1 m x 1 m)	Fully open	
Undersluices (0.8 m x 0.8 m)	Fully open	
Gravel trap	Fully open	
Bedload flushing gates	Fully closed	
Trash Passage	Fully closed	
Setting Basin #1 Outlet	Fully open	To ensure almost equal flow distribution amongst the settling basin
Setting Basin #2 Outlet	3m open	

9.3 Two years return period flood

During two years return period flood, similar to annual flood, most of the river flow was directed towards the weir, whereas the required design flow and the discharge for various flushing was only attracted towards the intakes. The operation of different gates adopted during this flood is presented in Table 9-2. A view of flow patterns at headworks during this flow is presented in Photograph D-52, Appendix D.

At two years return period flood with the steep river profile upstream of the river bend super critical flow throughout the river cross section was observed upto Section #3. Higher intensity of turbulence with a centralized flow path of unsteady flow was observed upto the weir axis.

Similar to the annual flood flow approaching the headworks was concentrated at the centre of the river channel and directed slightly towards the left bank while it approaches the weir axis (Section #5). The scouring of the sediment deposits continued towards the centre of the river channel and higher sediment deposition occurred on either side of the river bank especially on the right bank (as shown in Photograph D-52 and D-53). Closer to the weir, the concentration of flow at the left bank further scoured the sediment deposits at the left bank and high sediment deposition pattern emerged on right bank (as shown in Photograph D-52 and D-53). This sediment scouring and deposition pattern is reflected in the bed level measurement data shown in Figure B-13 to B-16, Appendix B and Photo D-76 and Photo D-77, Appendix D.

The average water level in front of the intake was 1949.4 masl. So, the trash passage was not operated to avoid submergence of the outlets of the undersluices and gravel trap in the flushing channel (as shown in Photograph D-54, Appendix D). Most of the floating debris was transported over the weir and those transported towards the intakes got accumulated in front of the intake as the trash passage was in a closed position.

Higher sediment deposition phenomena at the right bank significantly increased transport of bed load towards the undersluices. However, at full gate openings the undersluices were able to achieve bed control in front of the intakes. The sediment loading was also more uniformly distributed amongst all undersluices. Boulders up to 1 m in size were transported by the river. Most of the boulders including the larger ones, however, were directed towards the weir by the momentum of the flow (shown in Photograph D-52 and 53, Appendix D). The boulders of size up to 0.6 m were transported towards the intake which were flushed downstream by the undersluices.

Similar to annual flood submerged hydraulic jump was observed in the stilling basin and the maximum water level of 1941.78 masl was recorded at middle section of the stilling basin. Super critical flow was observed downstream of the end sill of the stilling basin which could risk the stability of the boulder riprap downstream of the basin. Flow scenario in the stilling basin is shown in Photograph D-56, Appendix D.

The measured bed levels are presented in Table C-27, Appendix C. Similarly, the measured water levels are presented in Table C-20, Appendix C. Whereas, figures comparing the water levels measured during existing situation test and for the FRHD are presented in Figure B-8, Appendix B. Videos of the model tests during 2 years flood is presented in Appendix-E.

Table 9-3: Details of gate openings maintained for Two years Flood

Gates	Gates position	Remarks
Intake	Fully open	Design discharge with discharge for the flushing units is abstracted.
Undersluices (1 m x 1 m)	Fully open	
Undersluices (0.8 m x 0.8 m)	Fully open	

Gravel trap	Fully open	
Bedload flushing gates	Fully closed	
Trash Passage	Fully closed	
Setting Basin #1 Outlet	Fully open	To ensure almost equal flow distribution amongst the settling basins
Setting Basin #2 Outlet	3m open	

9.4 Five years return period flood

Highly turbulent and unsteady flow was observed during five years return period flood compared to earlier flows. Due to high turbulence and heavy sediment loading in the river, it is recommended to close down the intake gates and shutdown the power plant. The operation of different gates adopted for five years return period flood is presented in Table 9-4. A view of flow patterns at headworks during this flood is presented in Photograph D-57, Appendix D.

At five years return period flood with the steep river profile upstream of the river bend, super critical flow throughout the river cross section was observed upto Section #3. Flow momentum with high kinetic energy strikes the right bank at outer bend. Most of the flow energy was observed to be released with the impact to the right bank creating a hydraulic jump. The river then turns nearly 90 degrees flowing towards the weir guided by the right bank nose cliff (as shown in Photograph D-57 and D-58). Higher intensity of turbulence with a centralized flow path of unsteady flow was observed upto the weir axis.

Similar to two years flood scenario, flow approaching the headworks was concentrated at the centre of the river channel and directed towards the left bank while it approaches the weir axis (Section #5). Scouring of the sediment deposits continued along the centre of the river channel exposing the boulder packed upstream apron. Higher sediment deposition occurred on either side of the river bank especially on the right bank (as shown in Photograph D-57 and D-59). Closer to the weir, the concentration of flow at the left bank further scoured the sediment deposits at the left bank and high sediment deposition pattern emerged on right bank (as shown in Photograph D-57 and D-59). This sediment scouring and deposition pattern is reflected in the bed level measurement data shown in Figure B-13 to 16, Appendix B.

The average water level in front of the intake was at 1950.00 masl. So, trash passage was not operated to avoid submergence of the outlets gravel of traps and undersluices in the flushing channel. Intake gates were closed as the power plant was not in operation. Higher sediment deposition phenomena at the right bank transported bed load towards undersluices which were operated at full opening to ensure bed control in front of the intake.

Boulders up to 1.2 m in size were transported by the river. Most of the boulders including the larger ones, however, were directed towards the weir by the momentum of the flow (shown in Photograph D-57 to D-59, Appendix D). The boulders of size up to 0.6 m were transported towards the intake which were flushed downstream by the undersluices.

Similar to two years return period flood submerged hydraulic jump was observed in the stilling basin with maximum water level of 1942.77 masl recorded at middle section of the stilling basin. Super critical flow with higher depth was observed downstream of the end sill of the stilling basin which could risk the stability of the boulder riprap downstream of the basin. Flow scenario in the stilling basin is shown in Photograph D-60, Appendix D.

The measured bed levels are presented in Table C-28, Appendix C. Similarly, the measured water levels are presented in Table C-21, Appendix C. Whereas, figures comparing the water levels measured during existing situation test and for the FRHD

are presented in Figure B-9, Appendix B. Videos of the model tests during 5 years flood is presented in Appendix-E.

Table 9-4: Details of gate openings maintained for Five years flood

Gates	Gates position	Remarks
Intake	Closed	Power plant shut down
Undersluices (1 m x 1 m)	Fully open	
Undersluices (0.8 m x 0.8 m)	Fully open	
Gravel trap	Fully open	
Bedload flushing gates	Closed	
Trash Passage	Closed	
Setting Basin #1 Outlet	Closed	Power plant shut down
Setting Basin #2 Outlet	Closed	

9.5 20 years return period flood

Highly turbulent and unsteady flow was observed during 20 years return period flood. So, the power plant is recommended to be shut down and intake gates will be closed. The operation of different gates adopted for 20 years return period flood is presented in Table 9-5. A view of flow pattern at headworks during this flow is presented in Photograph D-61, Appendix D.

Similar to five years flood scenario, flow approaching the headworks was concentrated at the centre of the river channel and directed slightly towards the left bank while it approaches the weir axis (Section #5). Intake gates were closed as the power plant was not in operation and the average water level in front of the intake was 1950.80 masl. Trash passage was not operated to avoid overloading of the flushing channel.

Boulders larger than 1.5 m in size were transported by the river. Most of the boulders including the larger ones, however, were directed towards the weir because all of the river flow was passing over the weir. Undersluices were closed to avoid clogging.

Similar to 5 years return period flood submerged hydraulic jump was observed in the stilling basin with maximum water level of 1943.73 masl recorded at middle section of the stilling basin. Super critical flow with higher depth was observed downstream of the end sill of the stilling basin which could risk the stability of the boulder riprap downstream of the basin. Flow scenario in the stilling basin is shown in Photograph D-62, Appendix D.

The measured water levels are presented in Table C-22, Appendix C. Whereas, figures comparing the water levels measured during existing situation tests and for the FRHD are presented in Figure B-10, Appendix B. Videos of the model tests during 5 years flood is presented in Appendix-E.

Table 9-5: Details of gate openings maintained for 20 years flood

Gates	Gates position	Remarks
Intake	Closed	Power plant shut down
Undersluices (1 m x 1 m)	Closed	
Undersluices (0.8 m x 0.8 m)	Closed	
Gravel trap	Closed	
Bedload flushing gates	Closed	
Trash Passage	Closed	
Setting Basin #1 Outlet	Closed	Power plant shut down
Setting Basin #2 Outlet	Closed	

9.6 50 years return period flood

The power plant is recommended to be shut down during 50 years flood and intake gates will be closed. The operation of different gates adopted at this flood is presented in Table 9-6. A view of flow patterns at headworks during this flow is presented in Photograph D-64, Appendix D.

Intake gates were closed as the power plant was not in operation and the average water level in front of the intake was 1950.96 masl. Trash passage was not operated to avoid overloading of the flushing channel.

Boulders larger than 1.5 m in size were transported by the river. Most of the boulders including the larger ones, however, were directed towards the weir because all of the river flow was passing over the weir. Undersluices were closed to avoid clogging.

Submerged hydraulic jump was formed in the stilling basin with maximum water level of 1943.88 masl recorded at middle section of the stilling basin. Super critical flow with higher depth was observed downstream of the end sill of the stilling basin which could risk the stability of the boulder riprap downstream of the basin. Flow scenario in the stilling basin is shown in Photograph D-65, Appendix D.

The measured water levels are presented in Table C-23, Appendix C. Whereas, figures comparing the water levels measured during existing situation test and for the FRHD are presented in Figure B-11, Appendix B.

Table 9-6 Details of gate openings maintained for 50 years flood

Gates	Gates position	Remarks
Intake	Closed	Power plant shut down
Undersluice (1mx1m)	Closed	
Undersluice (0.8m x 0.8m)	Closed	
Gravel trap	Closed	
Bedload flushing gates	Closed	
Trash Passage	Closed	
Setting Basin #1 Outlet		Power plant shut down
Setting Basin #2 Outlet		

9.7 100 years return period flood (design flood)

The flow was extremely turbulent hence the power plant is recommended to be shut down and intake gates will be closed. The operation of different gates for 100 years flood is presented in Table 9-7. A view of flow patterns at headworks during this flow is presented in Photograph D-67, Appendix D.

Turbulent Flow with waves and surges occurred along the entire stretch upto Section #4. The flow energy was observed to be released with the impact to the right bank creating a hydraulic jump. The right bank nose cliff defected the flow momentum away from the intake. Unsteady flow with high intensity observed all the way upto the weir axis (as shown in Photograph D-67 and D-68).

Bedload flushing gates were fully opened to increase the flood passage area. So, large amount of flow and sediment was passing through the sluice gates. Intake gates were fully closed and the average water level in front of the intake was 1950.96 masl. The trash passage was not operated to avoid overloading of discharge into the flushing channel. Boulders larger than 1.5 m in size were transported by the river. Most of the boulders were directed towards the weir. However, large boulders were also transported though the bedload flushing gates as well. Undersluices were closed to avoid clogging.

In the scenario when only one of the bedload flushing gates was opened, submerged hydraulic jump formed in the stilling basin with maximum water level of 1944.15 masl recorded at middle section of the stilling basin. When two bedload flushing gates were in operation the hydraulic jump formed within the basin. So, energy dissipation was satisfied at both bedload flushing gates opening condition. Super critical flow with higher depth was observed downstream of the end sill of the stilling basin which could risk the stability of the boulder riprap downstream of the basin. Flow scenario in the stilling basin is shown in Photograph D-69, Appendix D.

The measured water levels are presented in Table C-24, Appendix C. Whereas, figures comparing the water levels measured during existing situation tests and for the FRHD are presented in Figure B-12, Appendix B.

Table 9-7 Details of gate openings maintained for 100 years flood

Gates	Gates position	Remarks
Intake	Closed	Power plant shut down
Undersluice (1mx1m)	Closed	
Undersluice (0.8m x 0.8m)	Closed	
Gravel trap	Closed	
Bedload flushing gates	Fully Opened	Both gate to be fully opened
Trash Passage	Closed	
Setting Basin #1 Outlet	Closed	Power plant shut down
Setting Basin #2 Outlet	Closed	

9.8 Flushing of sediment in front of the intakes

A test was carried out at low monsoon flow (45 m^3/s) to flush the sediment deposited in front of the intakes. Both bedload flushing gates were opened to create a gravity flow for required for flushing. The low monsoon flow was sufficient to flush the sediment deposits in front of the intakes (Shown in Photograph D-70 to D-72).

During flushing operation, a hydraulic jump was observed just in front of the Intakes IG1. The hydraulic jump agitated the sediment deposits and started entering to the intakes IG1 and IG2. Hence to avoid this phenomena installation of at least 1 m high stop log at both intakes IG1 and IG2 is recommended before starting the flushing operation.

9.9 Overall Observation

a. Approach flow and hydraulics in front of intakes

The approach flow to the headworks was largely influenced by morphology of the river upstream. The river bend and cliff nose located at the right bank, upstream of the intakes deflect the flow naturally towards the left bank (i.e. towards the weir). Most of the river flow up to average monsoon flow were mostly passing through the intake part. So, these flows were re-directed towards the intakes from the left bank, approaching intakes following a curved flow path. The flow were mostly concentrated towards intakes IG4 and IG3 which created a low intensity secondary flow at the bottom in the direction of upstream intakes IG2 and IG1. For small flood scenario, annual and 2 years flood, most of the river flow was directed towards weir on the left bank and the flow shadow area formed at the right bank. So, less turbulent flow was observed in front of the intakes. The secondary flow, similar to that of average monsoon flow was observed in front of the intakes. For large floods: 5 years and higher floods, the flow was concentrated at the centre of the river channel and most of the flow was discharged over the weir. Intake structures remain in flow shadow area.

b. Floating debris handling

The approach flow for upto average monsoon flow was mostly concentrated towards intake IG4 and IG3. The overflow trash passage structure located downstream of IG4 thus attracted most of the floating debris but some were trapped in the pocket of space between the individual intake piers in front of the intakes. When the water level in front of the intake was higher than 1948.00 masl (i.e at annual flood and above) the discharge through the trash passage overloaded the flushing channel submerging the outlets of the gravel trap. Hence, the trash passage was only intermittently operated when the water level in front of the intakes was higher than 1948.00 masl to avoid the submergence of the outlets of undersluices and gravel traps.

c. Sedimentation and bed control

Higher amount of sediment deposition occurred along both left and right bank, upstream of the intake area, at flows up to average monsoon flow due to curved approach flow path. The bed load transported towards the intake was flushed by the undersluices and the bed control in front of the intakes was achieved. At minor flood scenarios, most of the flow was concentrated at the centre of the river channel scouring the sediment deposits. The scoured sediment were deposited at both banks and sediment bars emerged on either side of the river bank. The sediment transported towards the intakes, by the above phenomena, were flushed by the undersluices and the bed control in front of the intakes was achieved.

At higher floods, as the flood intensity increases the flow direction moved towards the left bank of the weir consequently sediment deposit increased on the right bank. Undersluices were kept open up to 5 years flood to flush the sediments and avoid built up of river bed in front of the intakes. Undersluices were closed at floods higher than 5 years flood to avoid their clogging. Sediment deposits along the centre of the river channel, upstream of the weir, is scoured and transported over the weir. Bedload flushing gates are opened fully to pass the design flood and sediment load partially.

Once large flood event subsides, intake was unable to extract the required design discharge after the flow has lowered due to heavy sediment deposited in front of the intakes. Therefore, it is recommended to flush the sediment deposited in front of intakes during falling limb of the large floods before operating the intakes for power generation.

d. Boulder movements

The concentration of approach flow was towards weir which favoured transport of most of the boulders including the large ones over the weir. Only boulders of smaller sizes were attracted towards the intake, which were flushed downstream by the undersluices. Undersluices were operated upto 5 years flood. At floods higher than 5 years return period, the undersluices were closed to avoid attraction of larger boulders towards the intake and clogging them.

e. Stilling Basin performance

Stilling basin with constricted width at the exit helped increasing water depth to contain hydraulic jump within the basin. The basin performed satisfactorily at all flows including the design flood. However, at floods higher than 50 year's return period, at least one of the bedload flushing gate had to be fully opened for favourable energy dissipation in the stilling basin. Stilling Basin in the weir portion was capable of handling flood up to 800 m^3/s. The super critical flow downstream of the end sill of the stilling basin, at 2 years and higher floods, was not favourable for stability of the boulder riprap structure.

10 OPERATIONAL ASPECTS

In combination to satisfactory design and quality construction, good operation regime of the headworks plays an important role in achieving desired hydraulic performance of the hydropower plants mainly built in Himalayan Rivers, which are known to have complex hydraulics and heavy sediment load. Therefore, operational aspects of the recommended design based on the assumptions made during the design and model testing have been described below. The guidelines should however, be used as a basis for headworks operation and shall be modified if required, based on the experience of first few years of operations.

For the ease of operation, the headworks operation regime has been divided into eight categories as follows, based on possible flow condition of river at the headworks area

- Dry Season flow (River flow < 25 m^3/s)
- Pre/Post monsoon low flow (River flow 25 - 45 m^3/s)
- Low Monsoon flow (River flow 45 – 75 m^3/s)
- High Monsoon flow (River flow 75 – 150 m^3/s)
- Minor floods (River flow 150 – 365 m^3/s)
- Large floods (River flow 365 – 550 m^3/s)
- Very Large Floods (River flow 550 – 790 m^3/s)
- Extreme flood event (River flow >790 m^3/s)

For starting the power plant after shutdown either for maintenance/inspection or due to very large flood events, two operation regimes has been recommended

- Soft restart operation
- Hard restart operation

The above classification is relative and specific to operation of the headworks of Ghunsa Khola Hydroelectric Project only. Suggested operation regimes are set based on the river flow or water levels at the headworks. Since water levels largely depend on the state of gate openings, it may be less reliable. So, a real time discharge monitoring is recommended upstream of the headworks area.

Rating curves of weir, undersluice, bedload flushing gates and the trash passage were generated to have additional information on flow passing over/through them during operation. The rating curves generated for the weir, bedload flushing gates, undersluice and trash passage are presented in Figure B-17 to Figure B-20, Appendix B respectively. For the generation of rating curves, water levels were measured at three different locations at various discharges. It is to be noted that sediment was not supplied in the river during the measurements. The water level measured at weir axis and in front of the intake marked XS – WA and XS - #I was used for creating the final rating curves. Meanwhile water level in front of the intakes marked XS - #I was used for generating operation principal for all gate operations.

It is also to be noted that the bedload flushing gates shall be closed in all the flow scenarios except for extreme flood event (above 50 year's floods) and during gravity flushing of the sediment deposited in front of the intakes.

Table 10-1: Summary of recommended gate operation regime with respect river flow and water level in front of intake

	Discharge	Water level in front of intake during operation	Target water level for operation	Intake	Gravel Trap	Upstream Undersluice (1x1m opening) (Gate size 1.5m)	Downstream Undersluice (0.8x0.8m opening) (Gate size 1.2m)	Trash passage	Bedload flushing gates
	(m^3/s)	(masl)	(masl)	(Open / Close)	(Open / Close)	(m)	(m)	(Open/Close)	(Open / Close)
Dry Season flow	< 25	< 1947.00	1946.5	Open	Closed/(RF)	Closed/(RF)	Closed/(RF)	Open	Closed
Pre/Post monsoon low flow	25-45	1946.50-1947.00	1946.5-1947.0	Open	Open/Close (M)	Closed/(RF)	Closed/(RF)	Open	Closed
Low Monsoon Flow	45-75	1947.00-1947.50	1947.0	Open	Open/Close (M)	0.6 - 0.75	0.4 - 0.5	Open	Closed
High Monsoon flow	75-150	1947.50-1948.00	1947.5	Open	Open	0.75 - 1.5	0.5 – 1.2	Open	Closed
Minor floods	150-365	1948.00-1949.40	1948	Open	Open	1.5 Fully open	1.2 Fully open	Closed	Closed
Large floods	365-550	1949.40-1950.30	<1950.9	Closed	Closed	Fully open	Fully open	Closed	Closed
Very Large floods	550-790	1950.30-1950.90	<1950.9	Closed	Closed	Closed	Closed	Closed	Closed
Extreme flood event	>790	>1950.90	<1950.9	Closed	Closed	Closed	Closed	Closed	Opened

Note: (RF) regular flushing as per the availability of flow

(M) operate the gravel trap gate to fully open or fully closed condition as per the availability of flow

Gravel Trap operation: Fully open / Fully closed
Upstream undersluice operation: Minimum Opening of 0.6m
Downstream undersluice operation: Minimum Opening of 0.4m
Trash passage operation: Fully open / Close once water level in front of intake goes above 1948.00 masl.
Bedload flushing gate: Operate for bedload flushing / Extreme flood event

10.1 Dry Season flow (River flow < 25 m^3/s)

The dry season flow here is defined as all possible river flows below the 25 m^3/s. This flow occurs during dry season normally from November to May.

Bed load transport is expected to be very low and is usually trapped in the pool in front of the intakes, so undersluices and bedload flushing gates shall be closed.

The gravel traps shall be operated based on available water in the river also considering environmental flow release. When water level in front of the intakes is less than 1946.50 masl, gravel traps will be fully closed. Undersluices and gravel traps should be flushed intermittently and alternatively as per availability of water.

Although the amount of trashes and floating debris in the river is expected to be less during non-monsoon flow, occasional trapping and cleaning of the trash rack at the intake might be required as no flow will be spilled through the trash passage during this dry season flow situation. The Table 10-2 below presents the recommended gate operation regime for dry season flow period.

Table 10-2: Recommended gate operation regime for river flow < 25 m^3/s or water level from 1946.5 masl to 1947.00 masl with all flushing gates fully closed

Gates	Opening	Remarks
Intake	All gates open	
Gravel trap	Closed	Gates are intermittently operated as per the availability of flow in the river.
Upstream Undersluices (1 x 1 m opening) (Gate size 1.5 m)	Closed	Gates are intermittently operated as per the availability of flow in the river.
Downstream Undersluices (0.8 x0.8 m opening) (Gate size 1.2 m)	Closed	Gates are intermittently operated as per the availability of flow in the river.
Trash passage	Open	Trash passage gate left open but water level will be low hence expect no flow through the trash passage
Bedload flushing gates	Closed	

When the flow in the river is below 22 m^3/s less than the design discharge, water uptake for power production has to be gradually decreased to maintain water level above 1946.50 masl in front of the intakes.

When the river flow is greater than 25 m^3/s or water level in front of the intakes at the suggested operation goes above 1947.00 masl, Pre/Post monsoon low flow operation regime should be followed which is mentioned in Section 10.2 below.

10.2 Pre/post monsoon low flow (River flow 25 - 45 m^3/s)

The Pre/post monsoon low flow is referred to river flows from 25 m^3/s to 45 m^3/s or water level in front of intakes from 1946.50 masl to 1947.00 masl at gate operation recommended in Table 10-3. This flow generally occurs during the month of June and October.

Bed load transport is expected to be very low at these river flows as it is usually trapped in the pool in front of the intakes. Therefore, gravel traps and undersluices shall be operated intermittently based on flow availability in the river. It should be noted that the gravel traps should be operated either fully open or fully closed for effective flushing. While operating the upstream undersluices the gate opening should be higher than 0.6 m to avoid their clogging. Similarly, while operating the downstream undersluices the gate opening should be higher than 0.4 m.

Hence, for Pre/post monsoon low flow scenario the gravel traps will be in operation with gates fully opened. Both gravel traps gates to be opened or only one gate to be opened at a time will be determined by availability of water to ensure water level is maintained between 1946.5 to 1947.00 masl in front of the intakes.

The floating debris in the river are expected during rainfall events in Pre/post monsoon low flow period. In such cases the gravel trap gates shall be closed to let the water level rise for passing floating debris over the weir. The Table 10-3 below presents the recommended gate operation regime for the pre/post monsoon flow period.

Table 10-3: Recommended gate operation regime for river flow from 25 m^3/s to 45 m^3/s or water level from 1946.50 masl to 1947.00 masl

Gates	Opening	Remarks
Intake	All gates open	
Gravel trap	Fully Open / Closed	Both gravel trap gates to be opened or only one gate to be open at a time will be determined by availability of water to ensure water level is maintained between 1946.5 to 1947.00 masl
Upstream Undersluices (1 x 1 m opening) (Gate size 1.5 m)	Closed	Gates are alternately operated as per the availability of flow in the river.
Downstream Undersluices (0.8 x 0.8 m opening) (Gate size 1.2 m)	Closed	Gates are alternately operated as per the availability of flow in the river.
Trash passage	Open	Trash passage gate left open but water level will be low hence expect no flow through the trash passage unless gravel trap gates are closed to increase the water level to let the trash overflow.
Bedload flushing gates	Closed	

When the river flow is higher than 45 m^3/s or water level in front of the intakes rises above 1947.00 masl at the suggested operation then low monsoon flow operation regime should be followed which is mentioned in Section 10.3 below.

When flow in the river is below 25 m^3/s at the suggested operation then dry season flow operation regime should be followed which is mentioned in the Section 10.1 above.

10.3 Low monsoon flow (River flow 45 m^3/s -75 m^3/s)

The average flow during monsoon period is estimated to be 75 m^3/s. The low monsoon flow here is referred to river flows from 45 m^3/s to 75 m^3/s or water level in front of thw

intake is from 1947.00 masl to 1947.50 masl at gate operation recommended in Table 10-4. This flow occurs during early July and end of October.

River transports sediment during these flows. Therefore, the gravel traps should be operated in full opening. Whereas upstream and downstream undersluices should be operated at least at an opening of 0.6 m and 0.4 m respectively. For maintaining the water level in front of the intakes at 1947.00 masl, the maximum openings at which upstream and downstream undersluices can be operated under this flow conditions is 0.75 m and 0.5 m respectively. Gate openings should be changed gradually to avoid extreme turbulence in front of the intakes.

While transitioning from pre monsoon low flow (45 m^3/s and below) to low monsoon flows (greater than 45 m^3/s or water level greater than 1947.00 masl) with gravel traps fully opened, upstream undersluices gate should be opened by 0.6 m to maintain water level of 1947.00 masl. With the upstream undersluices gates 0.6 m opened and if water level still remains higher than 1947.00 masl, then downstream undersluices gate should also be opened by 0.4 m to maintain water level of 1947 masl. Further, if water level continues to be higher than 1947.00 masl, upstream undersluices gate should be modulated between 0.6 to 0.75 m and downstream gate should be modulated between 0.4 to 0.5 m opening to maintain water level at 1947.00 masl.

The river will also transport significant floating debris during this period. The overflow trash passage structure should be operated at full opening to pass the floating debris safely downstream. However, the floating debris accumulated within the pocket space in front of the intakes should be mechanically removed from time to time.

Occasional pre monsoon floods higher than these flows might occur. So, caution should be taken for safe passage of bed load, floating debris and flood during this period. The headworks operators shall be at high alert so that operation procedure for high floods can be carried out immediately to avoid possible damages to the headworks structures and most importantly to the safety of the operators themselves.

During few pre-monsoon flood events, amount of sediment and floating debris could be even higher than during very large floods. Therefore, it is recommended to close the plant operation if the sediment concentration in river water is very high. The Table 10-4 below presents the recommended gate operation regime for the low monsoon flow period

Table 10-4: Recommended gate operation regime for river flow from 45 m^3/s to 75 m^3/s or water level from 1947.00 masl to 1947.50 masl

Gates	Opening	Remarks
Intake	All gates open	
Gravel trap	Full	
Upstream Undersluices (1 x 1 m opening) (Gate size 1.5 m)	0.6 m - 0.75 m	Opening should be controlled to maintain water level of 1947.0 masl in front of intake
Downstream Undersluices (0.8x0.8m opening) (Gate size 1.2 m)	0.4 m - 0.5 m	Opening should be controlled to maintain water level of 1947.0 masl in front of intake
Trash passage	Open	
Bedload flushing gates	Closed	

When flow in the river increases above 75 m^3/s or water level in front of the intakes rises above 1947.50 masl, with trash passage, gravel traps fully open, upstream and downstream undersluice opened by 0.75m and 0.5m respectively, then the monsoon high flow operation regime should be followed.

If the water level in front of the intakes drops below 1947.00 masl with trash passage, gravel traps fully open, upstream and downstream undersluices opened by 0.6 m and 0.4 m respectively, at first downstream undersluices gate should be closed to maintain water level of 1947.00 masl. With the downstream undersluices fully closed and if water level continues to go below 1947.00 masl, then upstream undersluices gate should be fully closed to maintain water level of 1947.00 masl. With both undersluices gates fully closed and water remains below 1947.00 masl, pre/post monsoon low flow operation regime should be followed which is mentioned in Section 10.2 above.

10.4 High monsoon flow (River flow from 75 m^3/s to 150 m^3/s)

High monsoon flow here is referred to river flows from 75 m^3/s and 150 m^3/s or water level in front of the intakes recorded from 1947.50 masl to 1948.00 masl. This flow occurs from June to September.

The river transports significant amount of sediments during high monsoon flows. Therefore, gravel traps and undersluices shall be operated to manage the sediments. Based on river flow, upstream undersluices gates shall be operated at openings from 0.75 m to 1.5 m. Similarly, downstream undersluices gates shall be operated at opening from 0.5 m to 1.2 m. Gravel trap, however, must be fully opened. The water level in front of the intakes should be maintained at 1947.5 masl.

Significant amount of floating debris are also expected during this period. The overflow trash passage structure should be kept open for safe passage of floating trashes to downstream. However, the trash accumulated at the pocket space in front of the intakes should be manually or mechanically removed from time to time. The recommended gate operation regimes for high monsoon flow periods is presented in Table 10-5 .

Table 10-5: Recommended gate operation regime for river flow from 75 m^3/s to 150 m^3/s or water level from 1947.50 masl to 1948.00 masl

Gates	Opening	Remarks
Intake	All gates open	
Gravel trap	Full	
Upstream Undersluices (1 x 1 m opening) (Gate size 1.5 m)	0.75 m - 1.5 m	Opening should be controlled to maintain water level of 1947.50 masl in front of intakes
Downstream Undersluice (0.8 x 0.8 m opening) (Gate size 1.2 m)	0.5 m - 1.2 m	Opening should be controlled to maintain water level of 1947.50 masl in front of intakes
Trash passage	Open	
Bedload flushing gates	Closed	

When flow in the river increases above 150 m^3/s or water level in front of the intake rises above 1948.00 masl, with trash passage, gravel trap, upstream and downstream undersluice gates fully opened, minor flood operation regime should be followed which is mentioned in Section 10.5 below. However, when the water level falls below 1947.50

masl with trash passage, gravel traps fully open, upstream and downstream undersluice opened by 0.75m and 0.5m respectively, low monsoon flow operation regime should be followed.

10.5 Minor Floods (River flow from 150 m^3/s to 365 m^3/s)

Minor floods frequently occur during monsoon season. However, possibility of such floods during pre-monsoon, post monsoon and even during dry seasons could not be completely ignored. Minor flood is referred to river flows from 150 m^3/s to 365 m^3/s or when water level in front of the intakes is from 1948.00 masl to 1949.40 masl with gates in operation as mentioned in Table 10-6. The upper limit of the minor flood condition is the two year's return period flood.

The power plant shall be in operation at these flows unless extreme sediment loads are transported in the river. If the water level in front of the intakes rises above 1948.00 masl, gravel traps and undersluices should be operated at full opening. However, once water level in front of the intakes goes above 1948.00 masl the trash passage has to be closed to avoid overloading of the undersluices and gravel traps flushing channel. Since the river flow is concentrated towards the weir hence majority of floating debris will pass over the weir. However, the significant floating debris could accumulate over time in the shadow flow area in front of the intakes. Therefore, once the flood subsides and the water level drops below 1948.00 masl trash passage should be opened to safely pass the accumulated floating debris to downstream. The floating debris trapped in the pocket in front of the intakes would require manual or mechanical cleaning.

Table 10-6: Recommended gate operation regime for river flow from 150 m^3/s to 365 m^3/s or water level from 1948.00 masl to 1949.40 masl

Gates	Opening	Remarks
Intake	All gates open	
Gravel trap	Full	
Upstream Undersluice (1x1m opening) (Gate size 1.5m)	Full	
Downstream Undersluice (0.8x0.8m opening) (Gate size 1.2m)	Full	
Trash passage	Closed	
Bedload flushing gates	Closed	

When flow in the river increases above 365 m^3/s or water level in front of the intake rises above 1949.40 masl, at the suggested operation then large flood operation regime should be followed which is mentioned in Section 10.6 below.

However, when flow starts to decrease and water level falls below 1948.00 masl at the suggested operation then high monsoon flow operation regime should be followed.

10.6 Large floods (River flow from 365 m^3/s to 550 m^3/s)

The floods above two year's return period flood and below ten year's return period flood are referred as large floods from operational aspect of this project. At these conditions, the river flow is from 365 m^3/s to 550 m^3/s and the water level in front of the intakes is from 1949.40 masl to 1950.30 masl with gates in operation as mentioned in the Table 10-7.

Such large floods transport high sediment load in the river and due to high turbulence large size gravels transported in suspension. So, undersluices and gravel traps operation cannot ensure bedload free water in the approach tunnels. Hence plants should not be operated at these flow conditions.

Since the power plant will not be operational, the trash passage gate and gravel traps shall be closed as well. However, the undersluices shall be operated at full opening to control bed deposit in the front of the intakes. Table 10-7 presents the recommended gate operation regime for minor flood condition.

Table 10-7: Recommended gate operation regime for river flow from 365 m^3/s to 550 m^3/s or water level from 1949.40 masl to 1950.30 masl

Gates	Opening	Remarks
Intake	All gates Closed	Plant shutdown
Gravel trap	Fully Closed	Intake gates closed condition
Upstream Undersluices (1 x 1 m opening) (Gate size 1.5 m)	Fully Open	
Downstream Undersluices (0.8 x 0.8 m opening) (Gate size 1.2 m)	Fully Open	
Trash passage	Fully Closed	
Bedload flushing gates	Fully Closed	

When flow in the river increases above 550 m^3/s or water level in front of the intake rises above 1950.30 masl, at the suggested operation then very large flood operation regime should be followed which is mentioned in Section 10.7 below.

However, when flow is below 200 m^3/s or water level falls below 1949.00 masl at the suggested operation then power plant can be restarted following soft restart procedure as mentioned in the above section.

10.7 Very large floods (River flow 550 to 790 m^3/s)

The floods above ten year's return period and below fifty year's return period are referred as very large floods from operational aspect of this project. At these conditions, the river flow is from 550 m^3/s to 790 m^3/s and the water level in front of the intakes is from 1950.30 masl to 1950.90 masl with gates in operation as mentioned in the Table 10-8.

Such large floods transport high sediment and bedload in the river and due to high turbulence large size gravels and cobbles will be transported in suspension. Hence, under these flood conditions, undersluices and intakes can be easily clogged and get damaged when operated. Therefore, the power plant shall be shut down during these

floods. Table 10-7 presents the recommended gate operation regime for very large flood condition.

Table 10-8: Recommended gate operation regime for river flow from 550 m^3/s to 790 m^3/s or water level from 1950.30 masl to 1950.90 masl

Gates	Opening	Remarks
Intake	All gates Closed	Plant shutdown
Gravel trap	Fully Closed	Intake gates closed condition
Upstream Undersluices (1 x 1 m opening) (Gate size 1.5 m)	Fully Closed	
Downstream Undersluices (0.8 x 0.8 m opening) (Gate size 1.2 m)	Fully Closed	
Trash passage	Fully Closed	
Bedload flushing gates	Fully Closed	

When flow in the river increases above 790 m^3/s or water level in front of the intakes rises above 1950.90 masl, at the suggested operation then extreme flood event operation regime should be followed which is mentioned in Section 10.8 below.

However, when flow decreases to 200 m^3/s or below and the water level in front of the intake falls below 1949.00 masl at the suggested operation then power plant can be restarted following hard restart procedure as mentioned in Section 10.11 below.

10.8 Extreme flood event (River flow above 790 m^3/s)

The floods higher than 50 years return period are referred as extreme floods from operational aspect of this project. At these conditions the river flow is higher than 790 m^3/s and water level in front of the intakes is above 1950.90 masl with gates in operation as mentioned in Table 10-9.

Such extreme floods transport high sediment and bedload in the river and due to heavy turbulent large size boulders will be in motion. Under these flood conditions safe flood passage will be the main priority. Hence, Intake, gravel traps and undersluices will be in closed position. For safe flood passage at least one or both bedload flushing gates shall be operated at full opening. In case both bedload flushing gates malfunctions and cannot be operated then trash passage gate has to be fully opened for emergency flood passage. Table 10-9 presents the recommended gate operation regime for extreme flood condition.

Table 10-9: Recommended gate operation regime for river flow above 790 m^3/s or water level is above 1950.90 masl

Gates	Opening	Remarks
Intake	All gates Closed	Plant shutdown
Gravel trap	Closed	Intake gates closed condition
Upstream Undersluices (1 x 1 m opening)	Closed	

(Gate size 1.5 m)		
Downstream Undersluices (0.8x0.8m opening) (Gate size 1.2m)	Closed	
Trash passage	Closed	If Bedload flushing gates fails to open, Open Trash passage.
Bedload flushing gates	Fully opened	

When flow starts to decrease below 200 m^3/s, continue following the flushing operation as mentioned in the Section 10.9 below. After the flood event subsides carry out a through damage assessments. Once damage assessment complete and headworks deemed safe for operation, follow hard restart operation procedure as mentioned in Section 10.11 to restart the power plant.

10.9 Flushing with bedload flushing gates

The bedload flushing gates shall be kept closed except at the extreme flood conditions. The bedload flushing gates are also designed to provide access for routine check up and maintenance of the intake gates/undersluices and to clear the deposits in front and upstream of the intakes by gravity flushing.

Flushing operation shall be carried out towards the end of monsoon season when the river flow is around 45-75 m^3/s. This ensures sufficient flow to transport the deposited bed load in front of the intake during flushing operation. This flushing is typically suggested to clear the bed deposited after large flood event or during draw down for routine inspection and maintenance of the intake gates.

The routine inspection and maintenance is generally done during low flow seasons. However, after a large flood event sediment deposited in front of the intakes is required to be cleared before staring the plant operation. Hence, at low flows gravity flushing of the deposits accumulated during entire monsoon period would require very long hours of shut down. Therefore, it is recommended that the flushing operation to be carried at the end of every large flood event. This will also help to maintain favourable approach flow towards the intake during post monsoon and dry flow season.

It was also observed that the intake gate sill could be accessed when the flow in the river was less than 25 m^3/s. However, a small hydraulic jump was formed and the water was splashing near IG1 gate. This scenario would be possible only during dry season period. During higher flow (45-150 m^3/sec) hydraulic jump occurs in front of the Intake IG1 which will transport sediment inside the intakes. Hence, installation of at least 1 m high stoplog at the intakes is recommended before starting the flushing sequence to avoid this phenomenon. To avoid clogging of the intakes, gravel traps and undersluices gates shall be closed before opening the bedload flushing gates either for flood passage or for flushing operation. Steps to be followed for bedload flushing gates flushing are mentioned below;

I. Close the intake gates.
II. Close gravel trap gates.
III. Install stoplog, with minimum height of 1 m, at the intakes. In case of extreme flood event when bedload flushing gates have to be opened for flood passage, installing the stoplog might not be possible. Hence, stoplog installation should be avoided and consider operator's safety first during extreme flood event.
IV. Close undersluice gates
V. Open the bedload flushing gates for gravity flushing.

10.10 Soft restart operation

Soft restart operation is defined as the procedure followed for restarting the power plant without bedload flushing gates flushing operation. Soft restart procedure is generally followed after the power plant has been shut down due to large flood events (Floods less than 550 m^3/s), for maintenance and inspection of power plant or settling basins which does not require water level to be lowered down in front of the intakes. For soft restart operation following steps are recommended

I. Ensure flow in the river is below 200 m^3/s, i.e. corresponding water level should be below 1949.00 masl with all gates in closed position. However, when the trash passage, gravel traps are closed and both undersluices gates are at full open position, the corresponding water level should be below 1948.50 masl.
II. Ensure no excessive floating debris is accumulated in front of the intakes, if present operate the trash passage to clear it first.
III. Close the trash passage.
IV. Open the undersluices outlet gates fully to confirm undersluices are not clogged and to ensure bed control in front of the intakes.
V. Open the gravel trap outlet gates to ensure gravel trap flushing is not clogged.

After ensuring that undersluices and gravel traps operating in good conditions, regulate the undersluices and gravel traps gates to maintain water level in front of the intakes above 1947.50 masl before starting the power plant.

Once water level in front of the intake is above 1947.50 masl, power plant can be restarted. Ensure the gradual increase in water uptake to avoid unfavourable hydraulics in front of the intakes. Water uptake for power production, gravel trap gates and undersluices gates operation should be modulated to maintain water level at 1947.00 masl in front of the intakes. Once the power plant is in full operation with water level in front of the intake is below 1948.00 masl, open the trash passage gate and follow the operation guidelines mentioned in Section 10 as per the flow available in the river.

10.11 Hard Restart operation

Hard restart operation is defined as the procedure followed for restarting the power plant which requires bedload flushing gates flushing operation. Hard restart procedure is generally followed after the power plant has been shut down due to any of the following reasons:

- Very large flood or extreme flood event (Flood larger than 550m^3/s).
- When large sediment deposit is observed in front or upstream of the intakes and especially on the right bank which could potentially restrict the flow towards intake during low flows.
- When the undersluices gets clogged and bed starts to build up in front of the intakes.
- Inspection and maintenance of undersluices or gravel traps.

In these cases water level needs to be drawn down by operating the bedload flushing gates. During very large flood event flushing operation should be carried out when the flood starts to subside. Therefore, ideally drawdown flushing operation should be carried out when flow in the river is between 150 m^3/s to 45 m^3/s.

Hence, for hard restart operation initially bedload flushing gates flushing operation steps have to be followed, which varies according to the river flow and are explained in the Section 10.9. Once bedload flushing gates flushing operation is complete follow the soft restart operation steps as mentioned in the Section 10.10 to start the plant operation.

11 CONCLUSIONS AND RECOMMENDATIONS

Physical hydraulic model tests of the base case design of headworks of Ghunsa Khola HEP was carried out in a geometrically similar Froudian model built at the scale of 1:30. The model represented the reach of Ghunsa Khola 250 m upstream and 150 m downstream of the weir axis. The tests revealed the following shortcomings in the performance of the design:

- Inability of the bedload sluice gates to control the bed deposits in front of the intake
- Unsatisfactory performance of gravel trap flushing system
- Clogging of trash racks by floating debris due to lack of trash passage facility
- Unfavourable hydraulics in front of the intakes due to strong secondary flow from the weir towards the intakes
- The impact of concentrated flow during the floods threatened the structural stability of gabion wall at the left bank, upstream of the weir
- Stilling basin was unable to dissipate the design flood at scenario in which one bedload sluice gate is in inoperable condition (n-1 gate operation)
- Supercritical flow downstream of the end sill of the stilling basin was unfavourable for the stability of boulder ripraps

After completion of the model tests of base case design, a meeting was held on 17th Oct 2019 with the Client at Hydro Lab's premises. After the discussion on the test observations, both parties agreed to carry out modifications in the base case design to improve the performance.

Various concepts for modifications were formulated and were conceptually tested in the model. Based on the test observations 'Modified Design Arrangement I' was developed and agreed for the testing. The modifications mainly involved introduction of undersluices, trash passage structures and gravel traps; change in upstream shape of the weir and changes in the orientation of the intakes. In the new design Bedload sluice radial gates are only used for flushing and flood passage, hence is named as bedload flushing gates. The design modification improved hydraulics and bed control in front of the intakes. However, some shortcomings were still observed with respect to passage of floating debris, secondary flow in front of the intakes, gravel trap flushing and energy dissipation in stilling basin.

The performance of the Modified Design Arrangement I was discussed with the Client in a meeting held on 20th Dec 2020. Both parties agreed to carry out further modifications in the design and "Modified Design Arrangement II" was formulated. The modifications mainly involved introduction of the hopper type gravel traps and over flow trash passage structure just upstream of the bedload flushing gates openings; extension of the divide wall and constriction of the outlet of the stilling basin. Both parties agreed that hopper type gravel trap with individual flushing channel would significantly improve the gravel flushing performance. As per the Client's request to expedite the model study due to time constrain hopper type gravel trap design performance was not tested in the model.

The tests of the Modified Design Arrangements-II revealed that the introduction of overflow trash passage structure in front of the bedload flushing gate significantly reduced the intensity of the secondary flow intensity. The structure also improved the passage of floating debris. Similarly, after constricting the outlet of the stilling basin by 4 m, the hydraulic jump at the design flood (1 in 100 year's return period) was contained within the stilling basin even when one of the bedload flushing gates was not in

operation. So, the headworks layout, excluding settling basins, after Modified Arrangement-II was deemed to perform satisfactorily. The super critical flow downstream of the stilling basin, however, was not favourable for the stability of the boulder rip-rap structure.

Settling basins were then implemented in the Modified Arrangement-II for the testing. The test results revealed the following shortcomings in the performance of the settling basin:

- Uneven flow distribution between the settling basins
- Flow at the end of the upstream transition zone was concentrated near the bottom of the basin resulting in reverse flow at the crown of the settling basins.
- Effective length of the settling basins was observed to be 80 % of the total available length due to non-uniform distribution of flow in vertical direction.
- Haunting oscillation of flow was observed as a result of non-uniform velocity profile exiting from the inlet transition of the settling basins.
- Stagnant recirculating zone observed on both side of the settling basins due to abrupt change in flow area at the basin outlet.

A meeting was held on 5th Feb 2020 with the Client to discuss the model observations. The Consultant recommended the following modifications for improving the performance of the settling basins.

- The outlet gates of Settling Basin #2 and Settling Basin #1 shall be operated at 3 m and 3.9 m openings respectively to obtain even flow distribution between the basins. Furthermore, the layout of the outlet tunnel from the settling basins will be adjusted for obtaining the equal head loss and equal withdrawal of discharge from the settling basins.
- A straight and flat identical approach tunnel (with 0 % slope) will be introduced at the entrance of the settling basins to reduce concentration of flow near the bottom of the transition zone
- Angle of inlet transition will be reduced or the size of the approach tunnel will be increased for improving hydraulics at the entrance of the basin.
- An exit transition will be implemented for smooth flow exit path at the end of the basins.

The implementation of the proposed modifications in the settling basins and verifying the performance after modifications would require some time. As per the Client's request to expedite the model study due to time constrain the recommended modifications will be implemented in the final design of the settling basins, but will not be tested in the physical model. Hence, the Modified Arrangement II with the implementation of the recommended modifications in the settling basins was considered as the final recommended hydraulic design.

Model tests of the Final Recommended Hydraulic Design (FRHD) were carried out for the final documentation purpose. The recommended modifications were not implemented in the gravel trap and settling basins so their performance could not be verified in the model. Based on the results of the model study, the following recommendations are made:

- The Modified Arrangement II with the implementation of the recommended modifications in the settling basins shall be adopted as the final recommended hydraulic design.
- The operation regime recommended by this study shall be updated /modified based on the actual operation experience at site.

- Undersluices are prone to clogging when operated at small gate opening. Hence, it is strongly recommended to operate the upstream and downstream undersluices at minimum openings of 0.6 m and 0.4 m respectively.
- The openings of undersluices shall not be altered during recession period of flood to avoid their clogging and excessive sediment deposition in front of the intakes.
- It is strongly recommended that gravel trap flushing gates should be operated at fully open or fully closed position to avoid clogging.
- Once water level in front of the intakes rises above 1948.00 masl the trash passage gate shall be closed to avoid overloading of the flushing channel and submergence of the gravel trap and undersluices outlets.
- Proper sediment deposition monitoring system is recommended to be installed in front of the intake while flood motoring system is recommended to be installed upstream of the headworks to collect lead information on flood magnitude required for operating the gates properly
- Bedload sluice is meant to be operated for gravity flushing of the sediment deposited in front of the intake, passage of extreme flood event (higher than 50 years floods) and for lowering the water level for routine inspection and maintenance of the intake and undersluices. Hence, it shall be closed during rest of the operation period.
- The deposits in front of the intakes shall be flushed towards the end of the monsoon season to avoid longer period of shut down required for flushing the deposits during low flow season, for regular inspection and maintenance
- To maintain favourable upstream approach flow hydraulics and to reduce cross current secondary compensation flow, the sediment deposits at right bank upstream of intake shall be kept lower than 1946.00 masl. If the deposit is higher and no flow is observed over the deposit then it shall be flushed or removed manually.
- Before starting the flushing sequence, a stoplog of minimum 1 m height shall be installed at the intake to avoid sediment entering the intake.
- This model study was mainly focused on improving the performance of the headworks structure and finalizing the hydraulic design. Thus, it is recommended to carry out necessary structural analysis of the Final Recommended Hydraulic Design from this study before implementing it for construction. It is worth mentioning that the apron downstream of the stilling basin is prone to damage by super critical flow during major floods. Hence design of a suitable protection structure is recommended during structural detailing.

APPENDIX A
Drawings

Physical Hydraulic Model Study of the Headworks of Ghunsa Khola HEP (78 MW)

Final Report

December 2020

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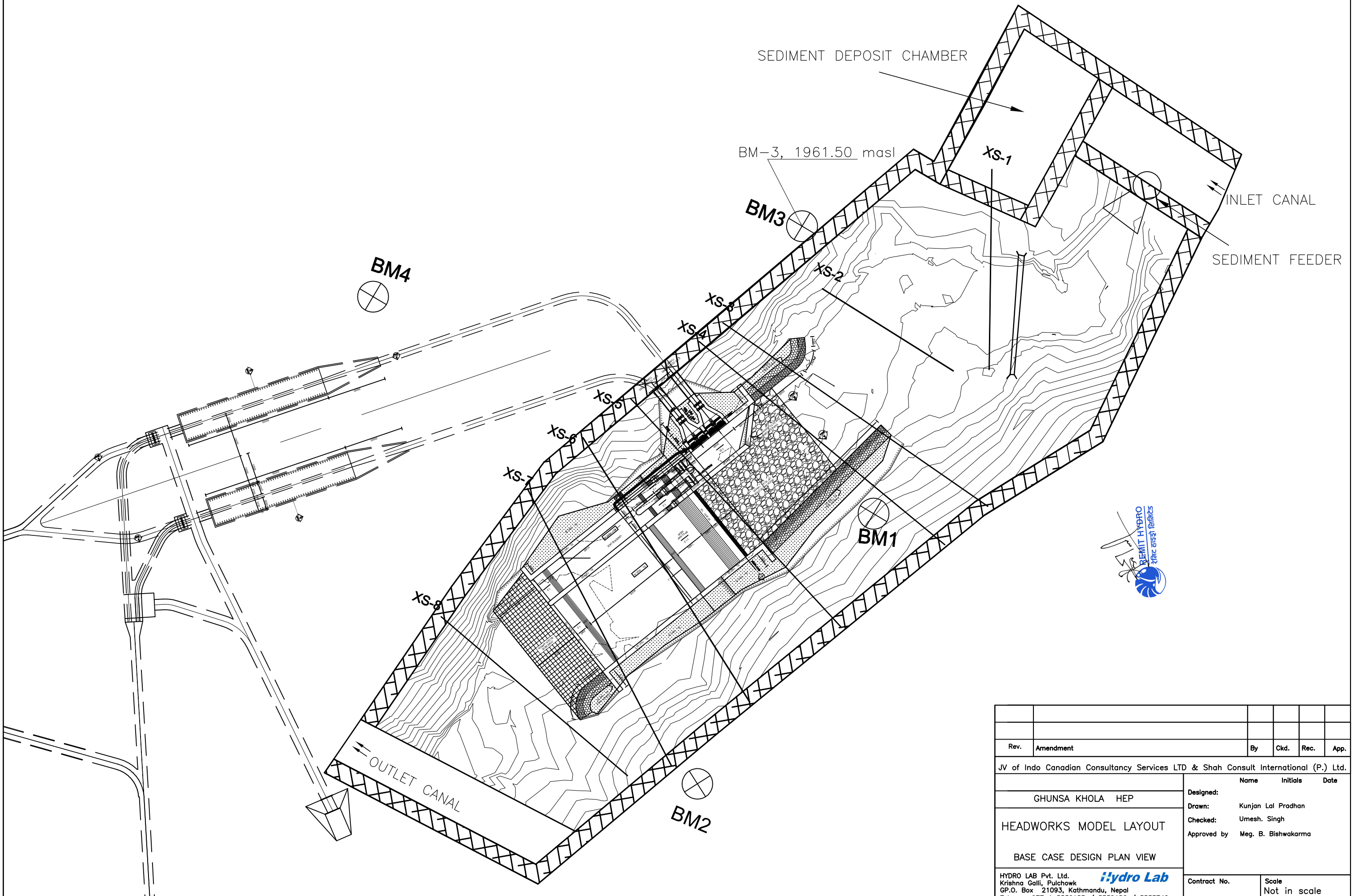

Rev.	Amendment	By	Ckd.	Rec.	App.

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GHUNSA KHOLA HEP

HEADWORKS MODEL LAYOUT

BASE CASE DESIGN PLAN VIEW

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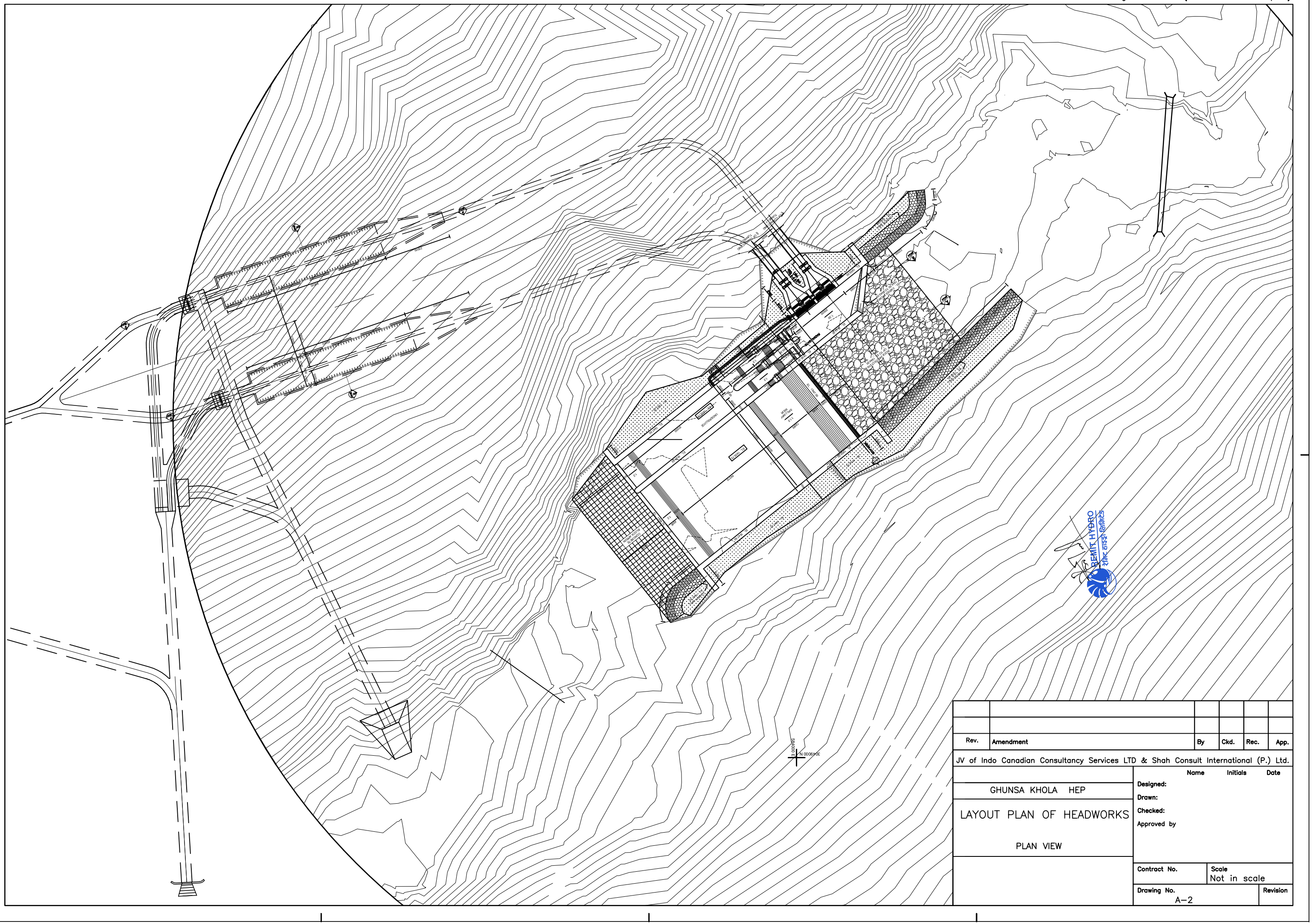
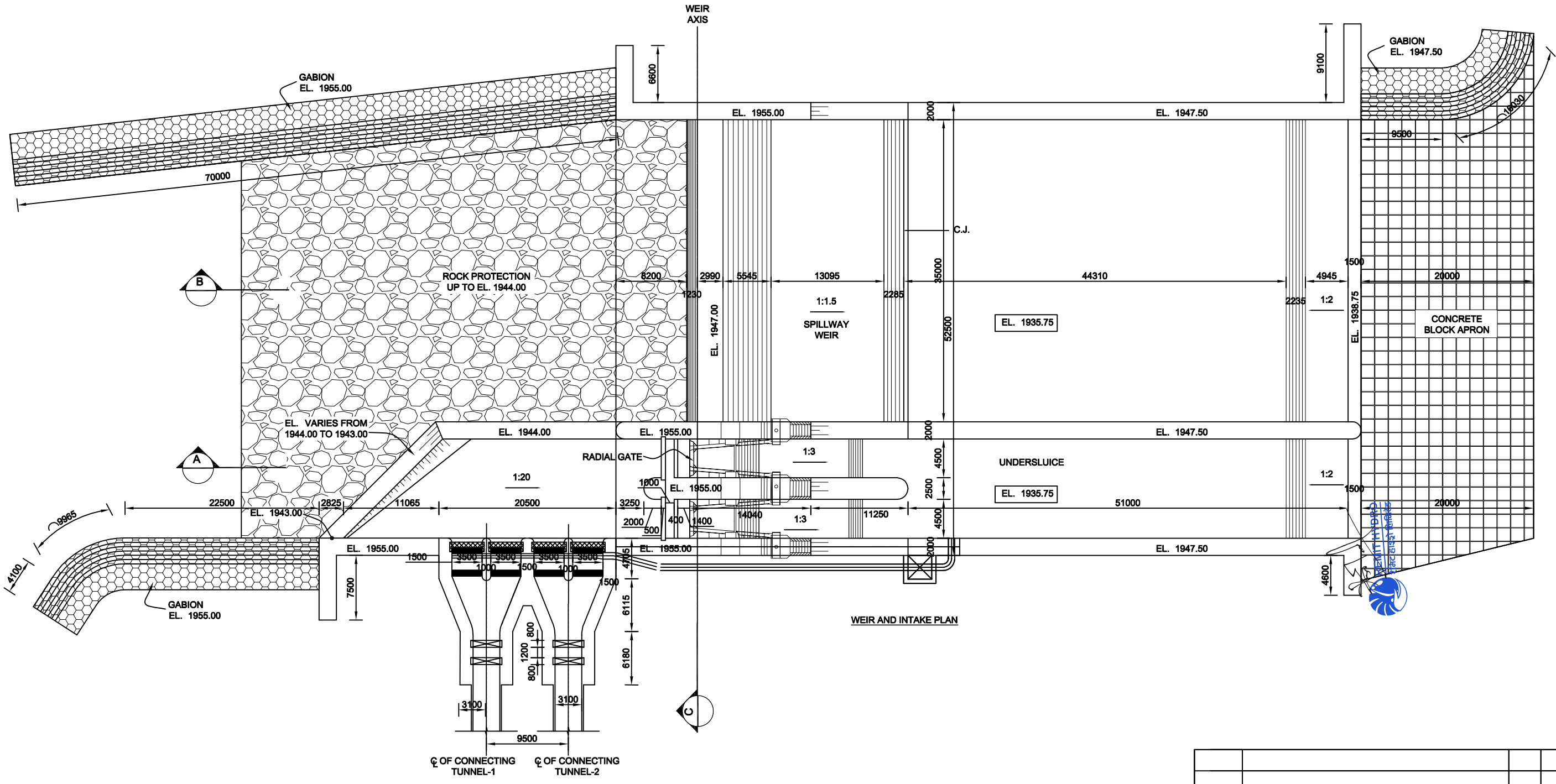
Drawing No. : A–2 (As refer to the report)





Rev.	Amendment	By	Ckd.	Rec.	App.

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GHUNSA KHOLA HEP

PLAN VIEW OF HEADWORKS

BASE CASE DESIGN PLAN VIEW

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Approved by	Client		

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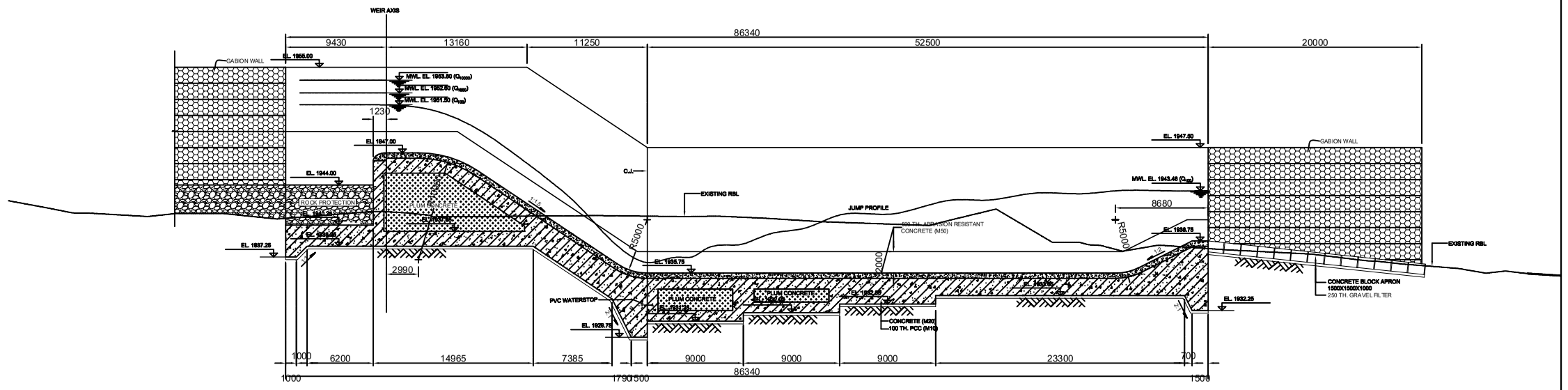
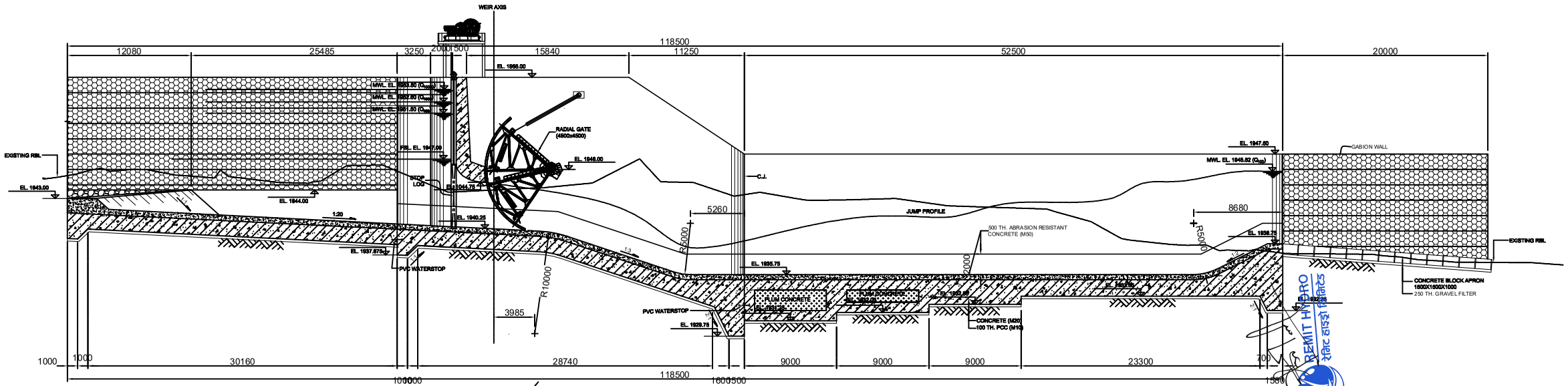
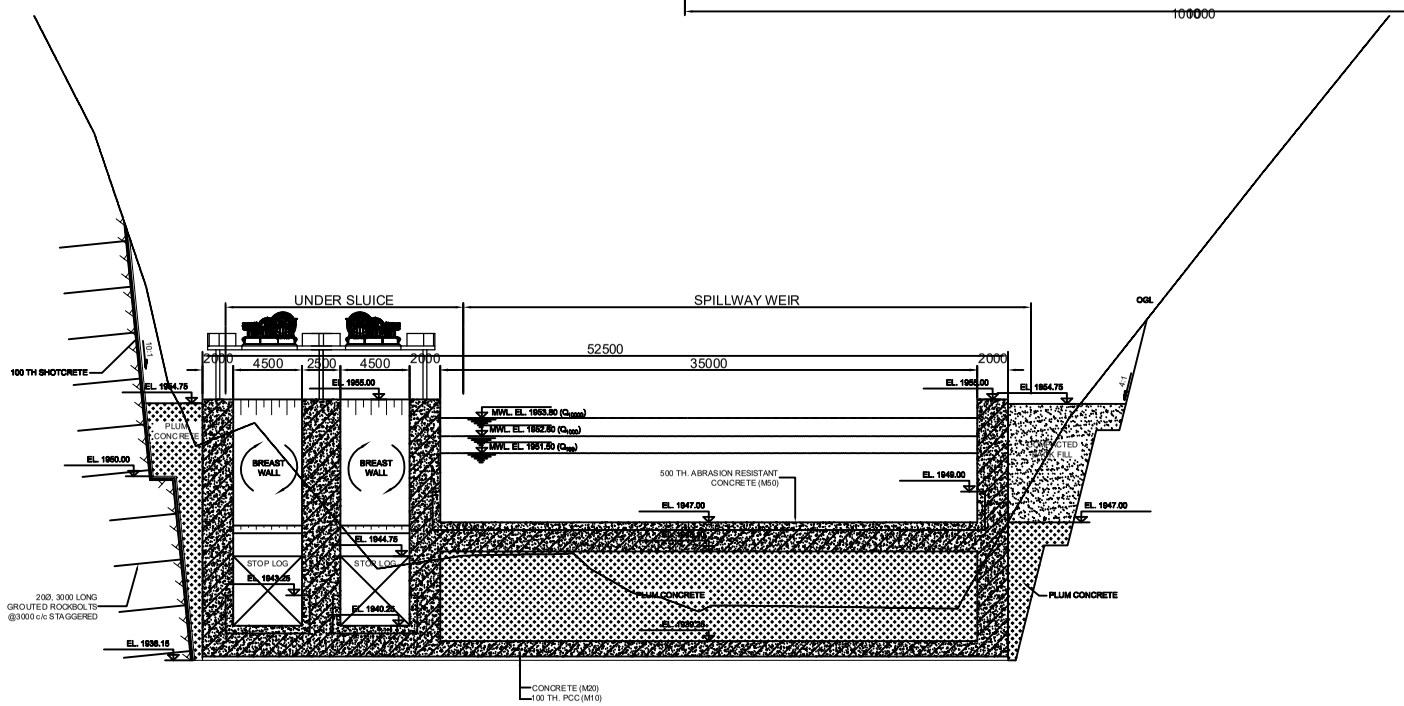


L–Section View B–B



L–Section View A–A



Section View C–C

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GHUNSA KHOLA HEP

WEIR SECTION VIEW

BASE CASE DESIGN SECTION

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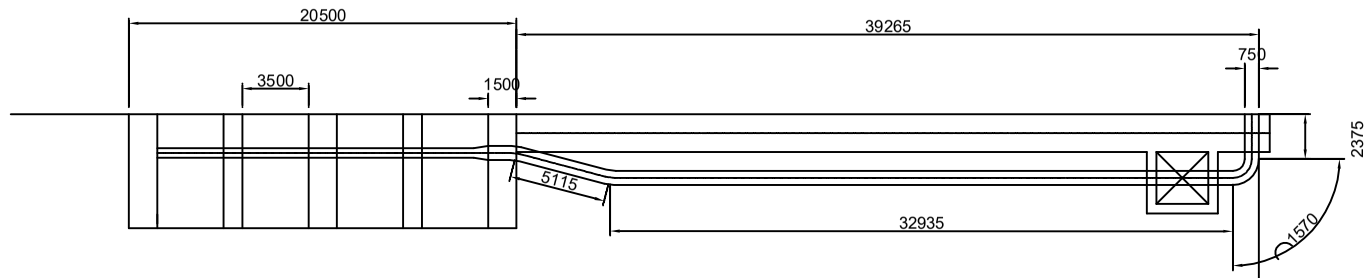
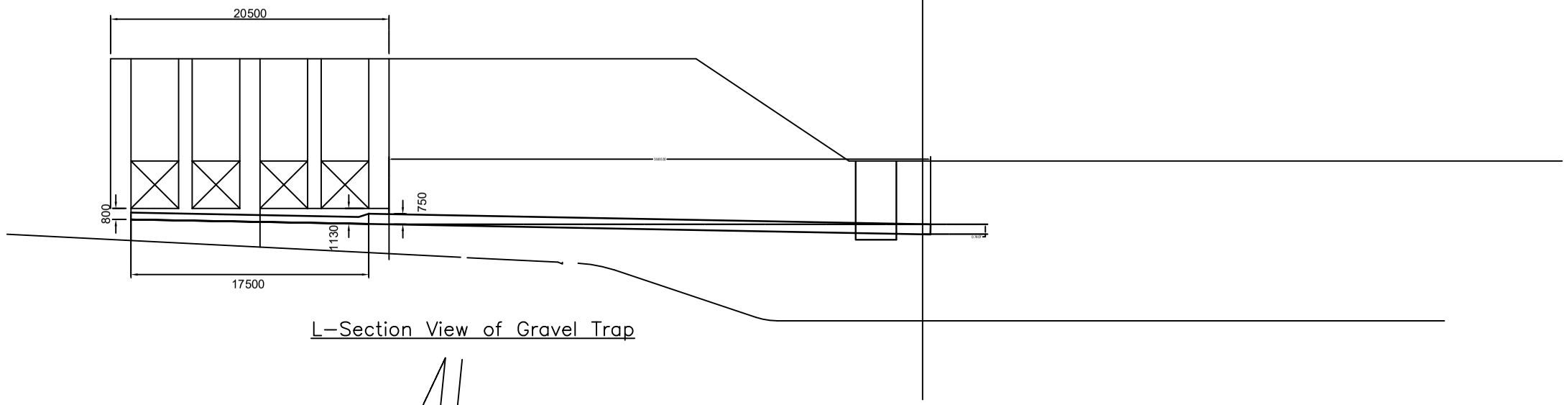
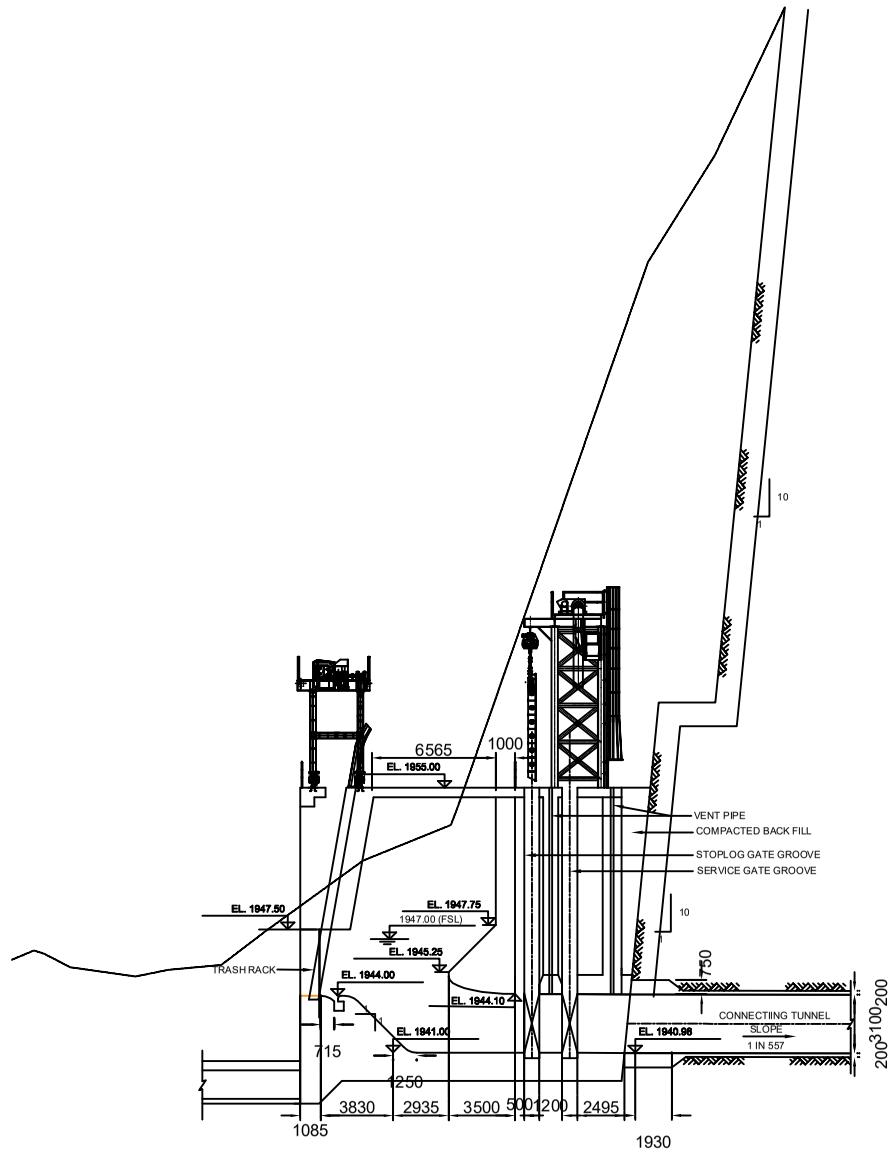
Drawing No. A–4	Revision



Top View of Gravel Trap



L–Section View of Gravel Trap



Section View of Intake and Gravel Trap

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GHUNSA KHOLA HEP

INTAKE SECTION DETAILS

BASE CASE DESIGN SECTION

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Contract No.	Scale
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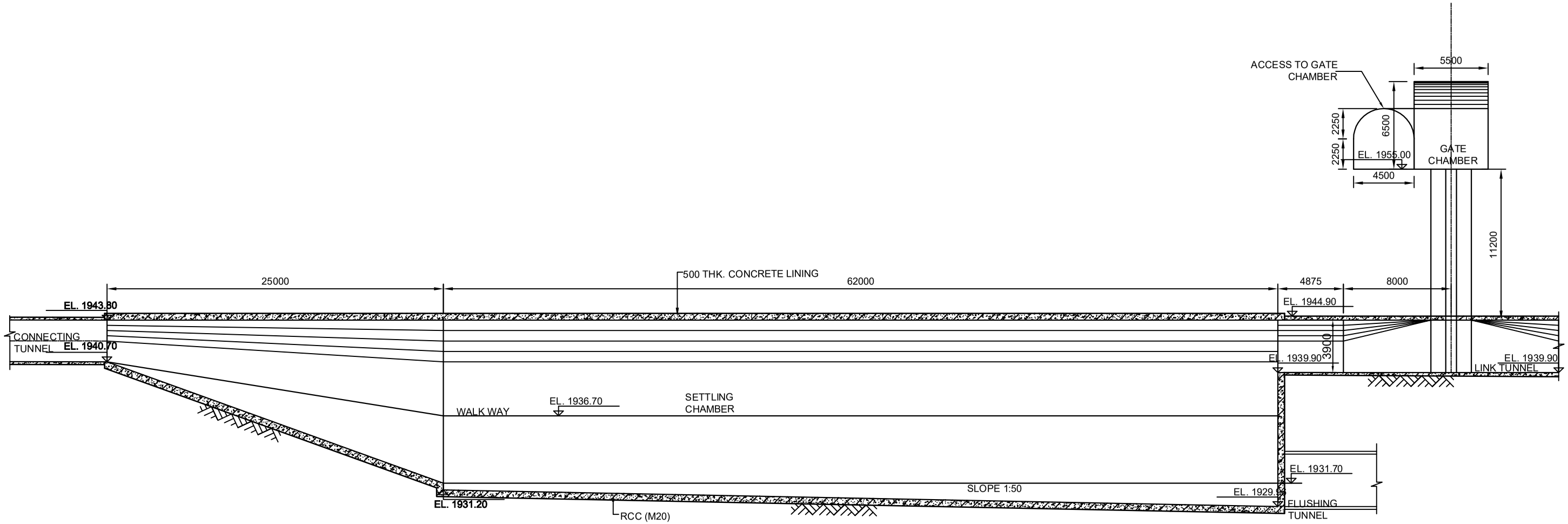
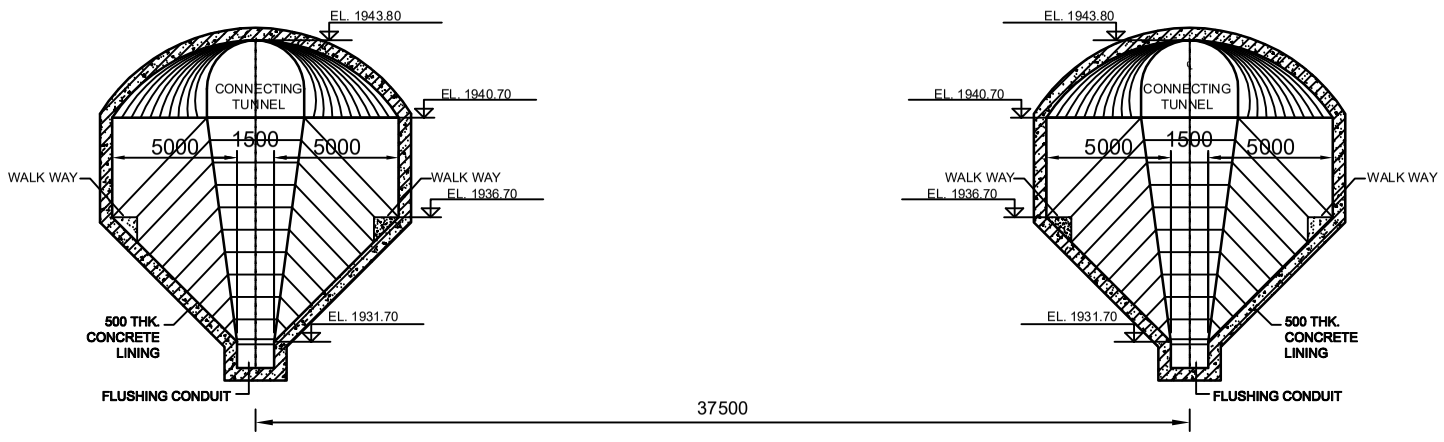
Drawing No. A–5	Revision



L–Section View P–P



Section View Q–Q

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GHUNSA KHOLA HEP

SETTLING BASIN SECTION VIEW

BASE CASE DESIGN SECTION

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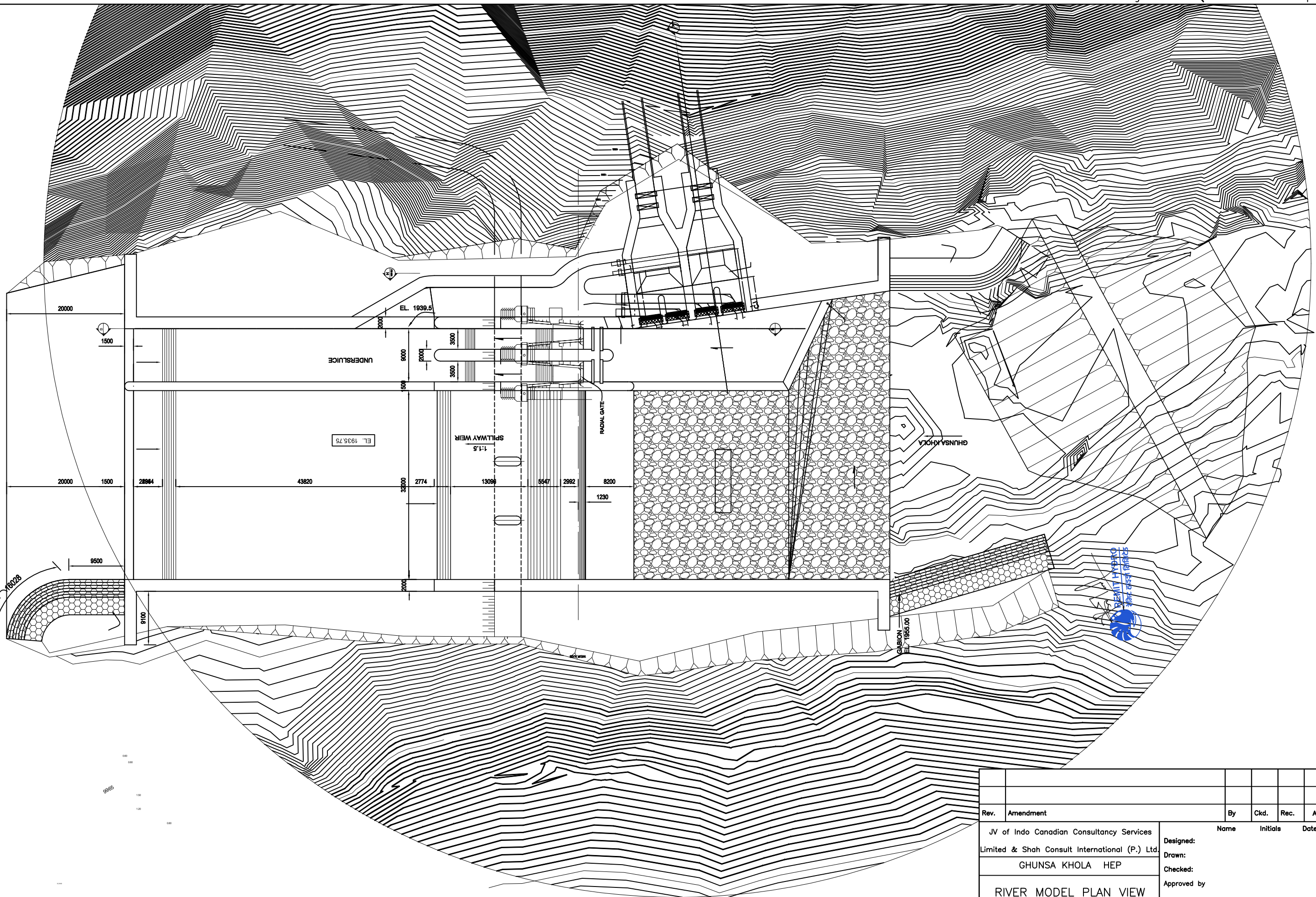

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RIVER MODEL PLAN VIEW

NMDA–II PLAN VIEW

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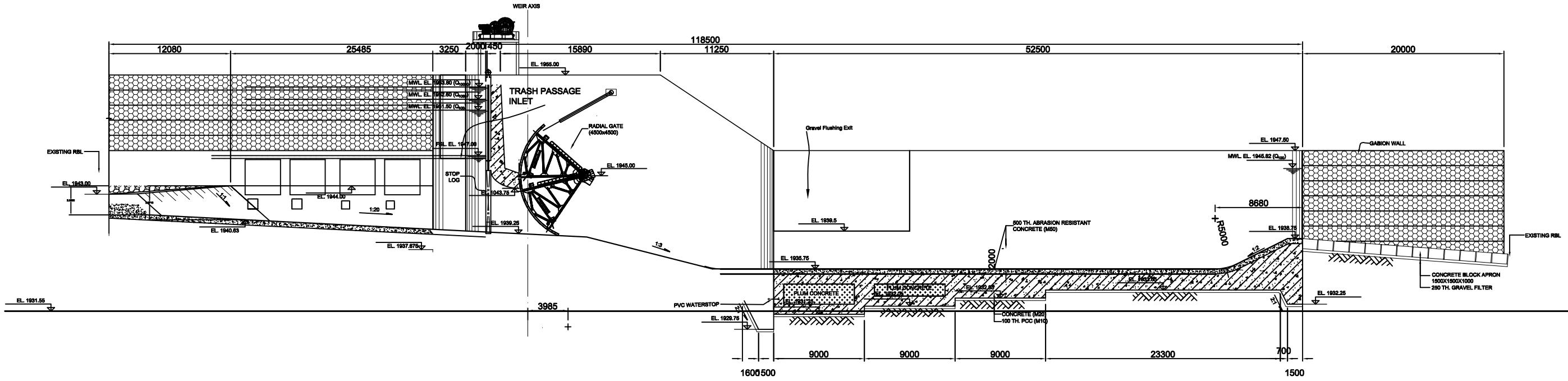

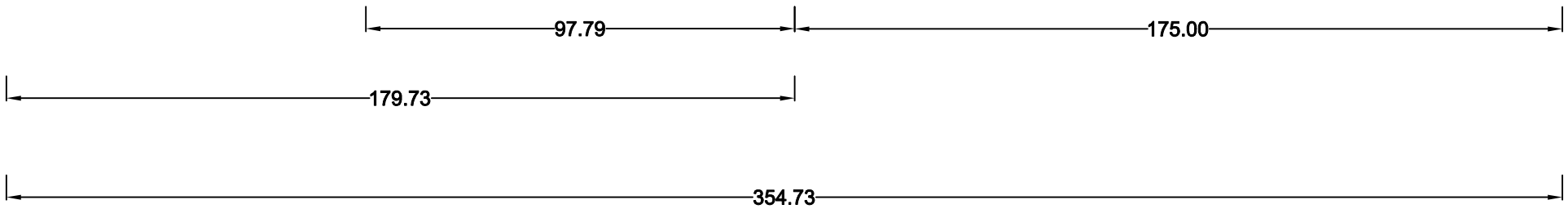

Section B : Radial gate undersluice sectional view

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GHUNSA KHOLA HEP

WEIR L-SECTION VIEW
NMDA-II SECTIONAL VIEW

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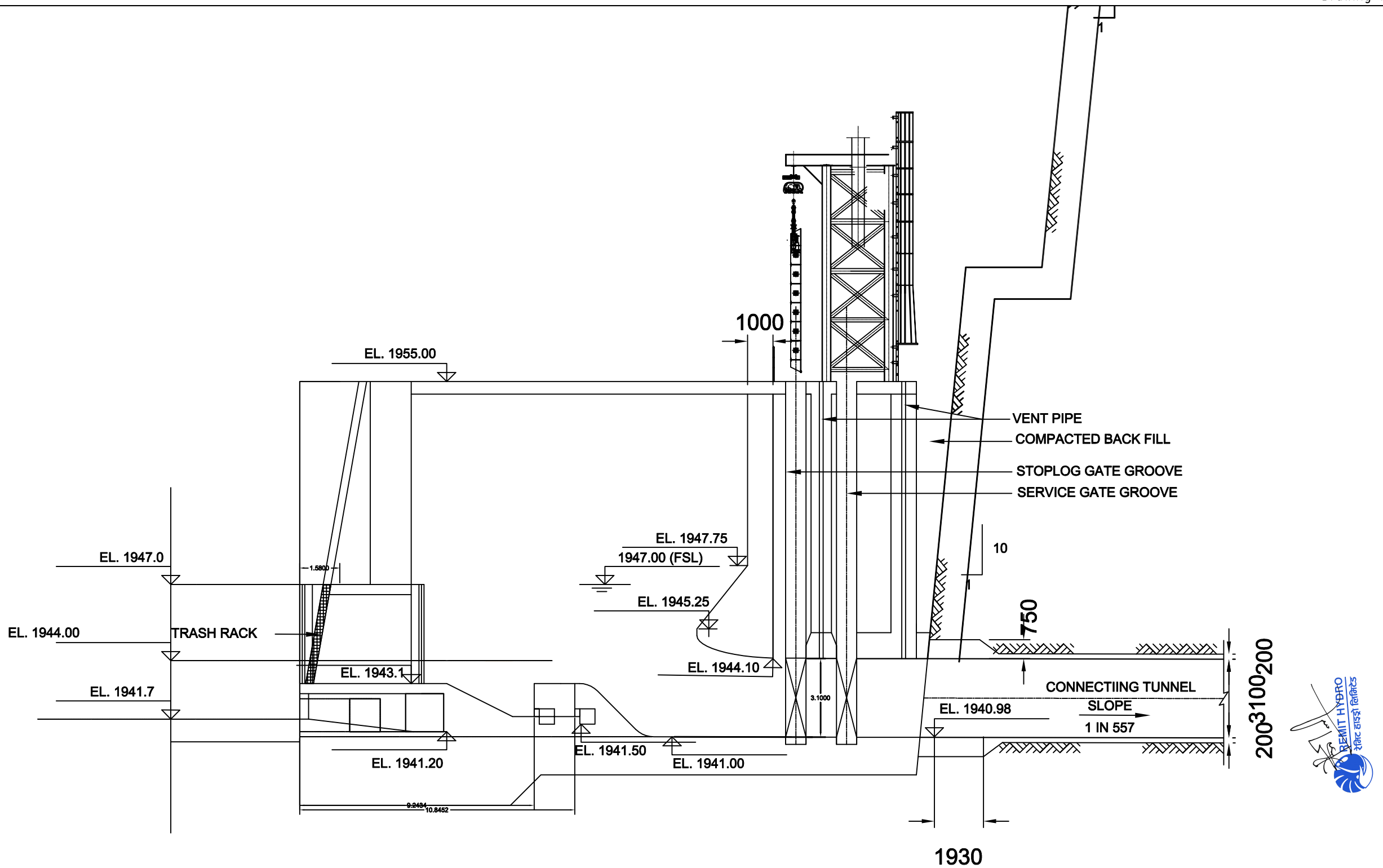

Section A : Intake sectional view

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INTAKE–SECTION VIEW
NMDA–II SECTIONAL VIEW

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Approved by			

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Drawing No.	Revision
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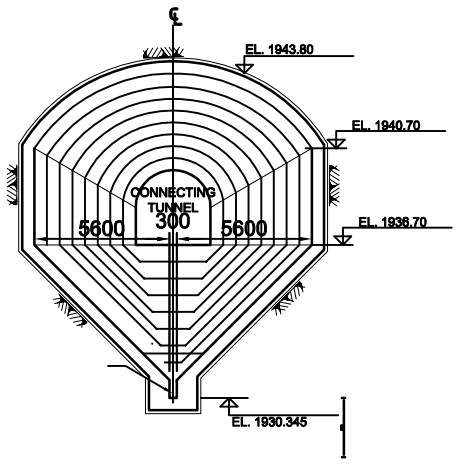
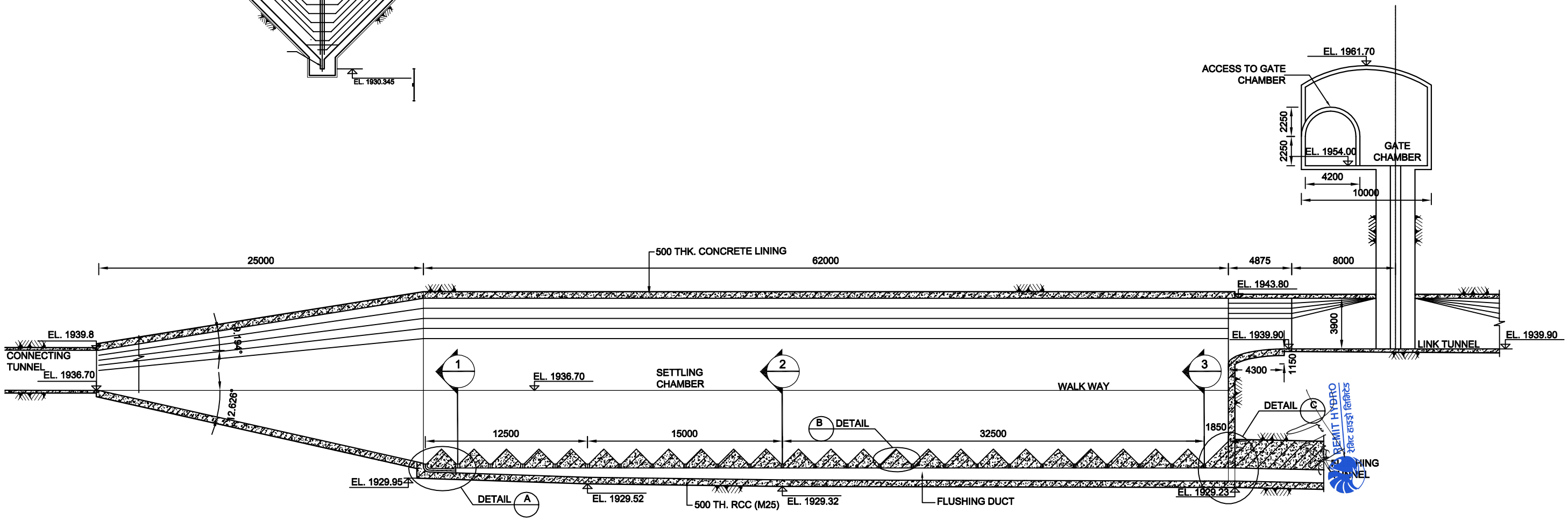
Section D : Settling Basin Cross sectional view





Section C : Settling Basin L-sectional view

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SETTLING BASIN VIEW
NMDA–II SECTIONAL VIEW

	Name	Initials	Date
Designed:			
Drawn:			
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Approved by			

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Drawing No.	Revision
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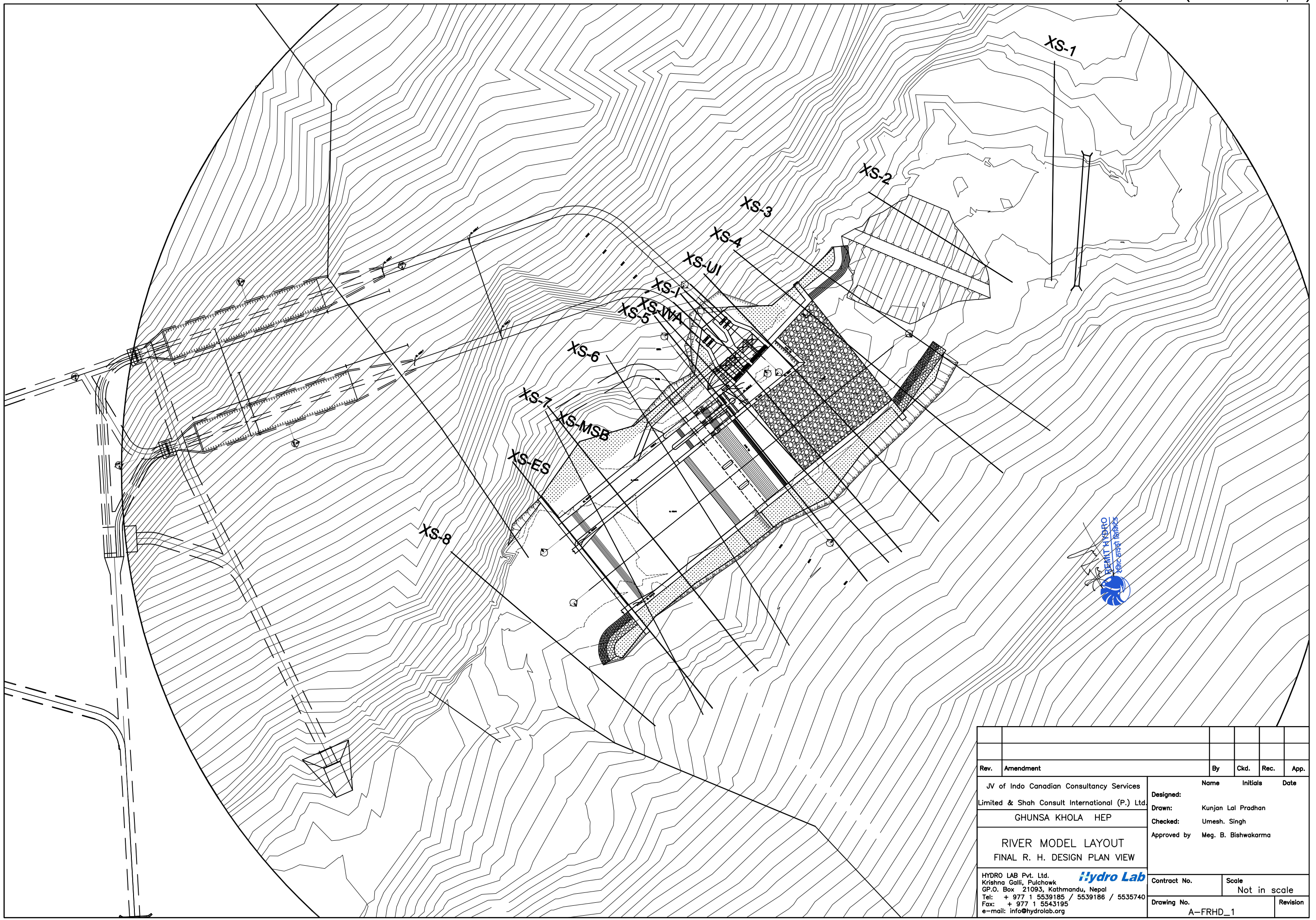

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GHUNSA KHOLA HEP

RIVER MODEL LAYOUT
FINAL R. H. DESIGN PLAN VIEW

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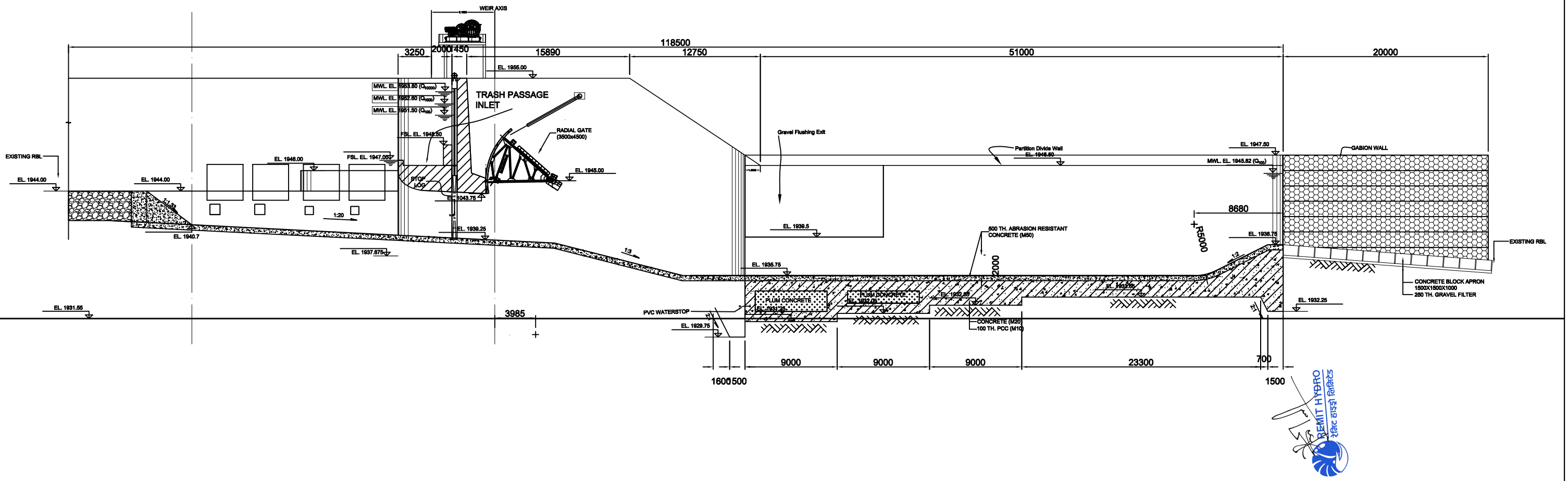
Contract No.	Scale
	Not in scale
Drawing No. A–FRHD_1	Revision



Section B : Radial gate undersluice sectional view

Rev.	Amendment	By	Ckd.	Rec.	App.

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GHUNSA KHOLA HEP

UNDER SLUICE L–SECTION
FINAL R. H. DESIGN SECTION VIEW

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Contract No.

Scale: Not in scale

Drawing No.: A–FRHD_2

Revision

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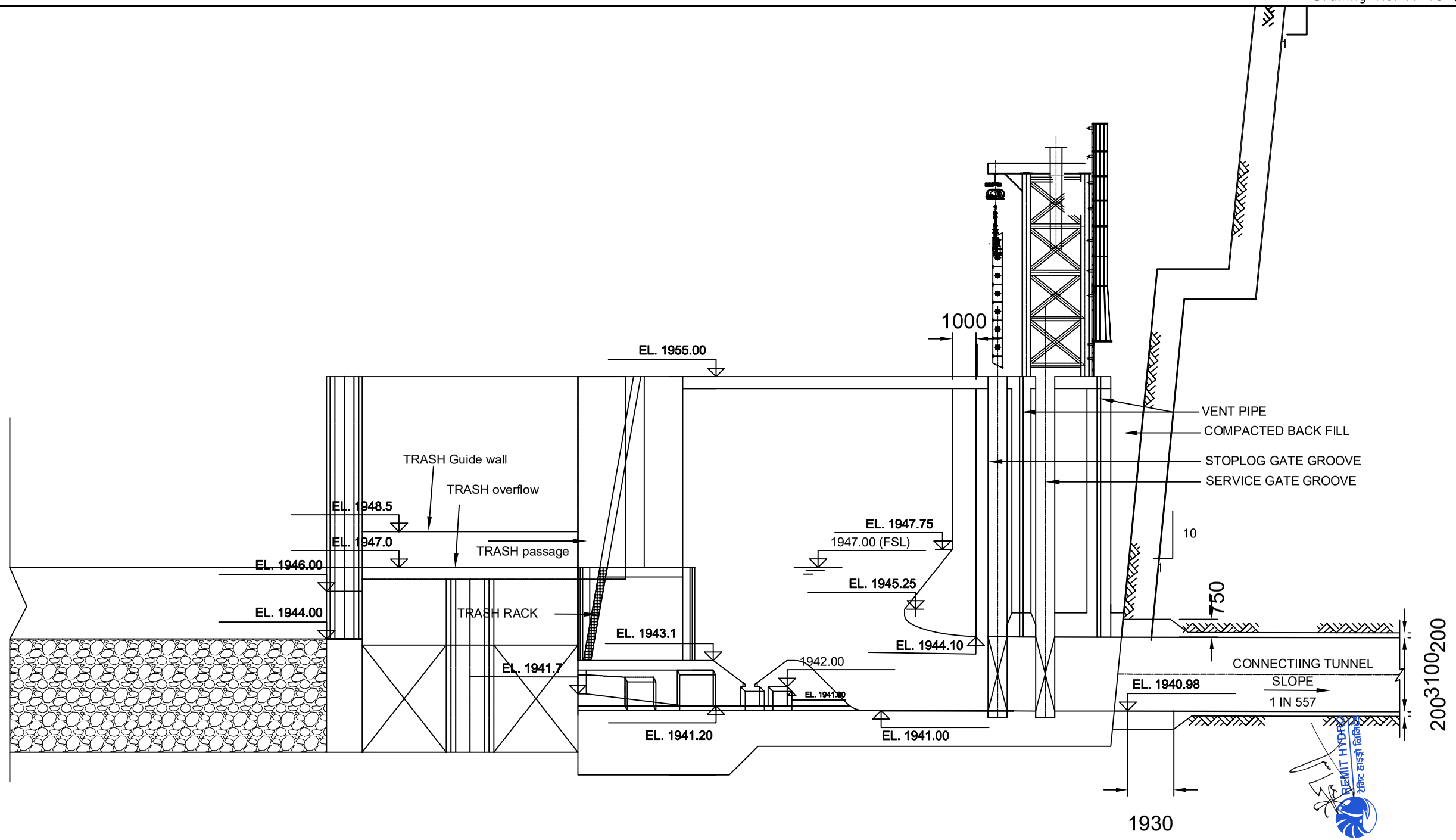

Section A : Intake sectional view

Rev.	Amendment	By	Ckd.	Rec.	App.

JV of Indo Canadian Consultancy Services Limited & Shah Consult International (P.) Ltd.

GHUNSA KHOLA HEP

INTAKE–SECTION VIEW

FINAL R. H. DESIGN SECTION VIEW

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Approved by	Meg. B. Bishwakarma		

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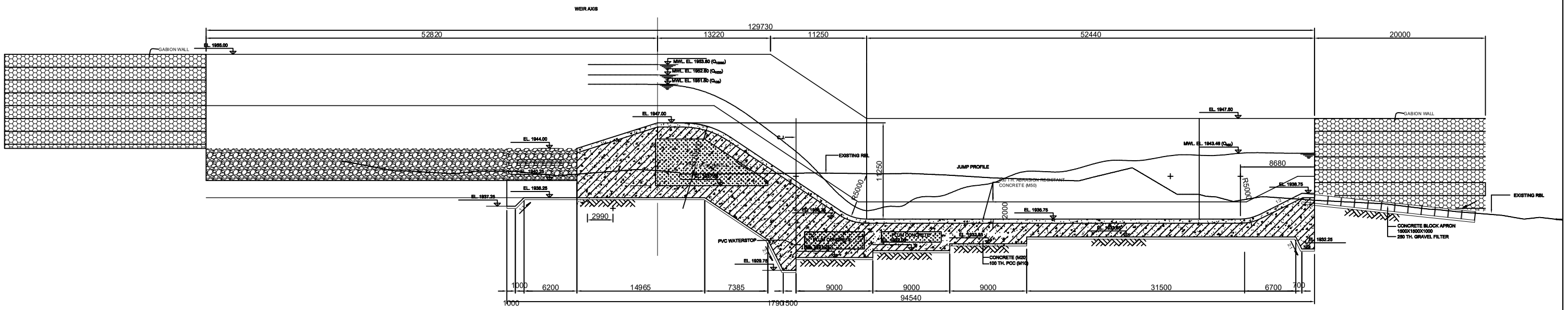

Section C: Sectional view along the diversion weir and stilling basin

Rev.	Amendment	By	Ckd.	Rec.	App.

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GHUNSA KHOLA HEP

WEIR L–SECTION VIEW
FINAL R. H. DESIGN SECTION VIEW

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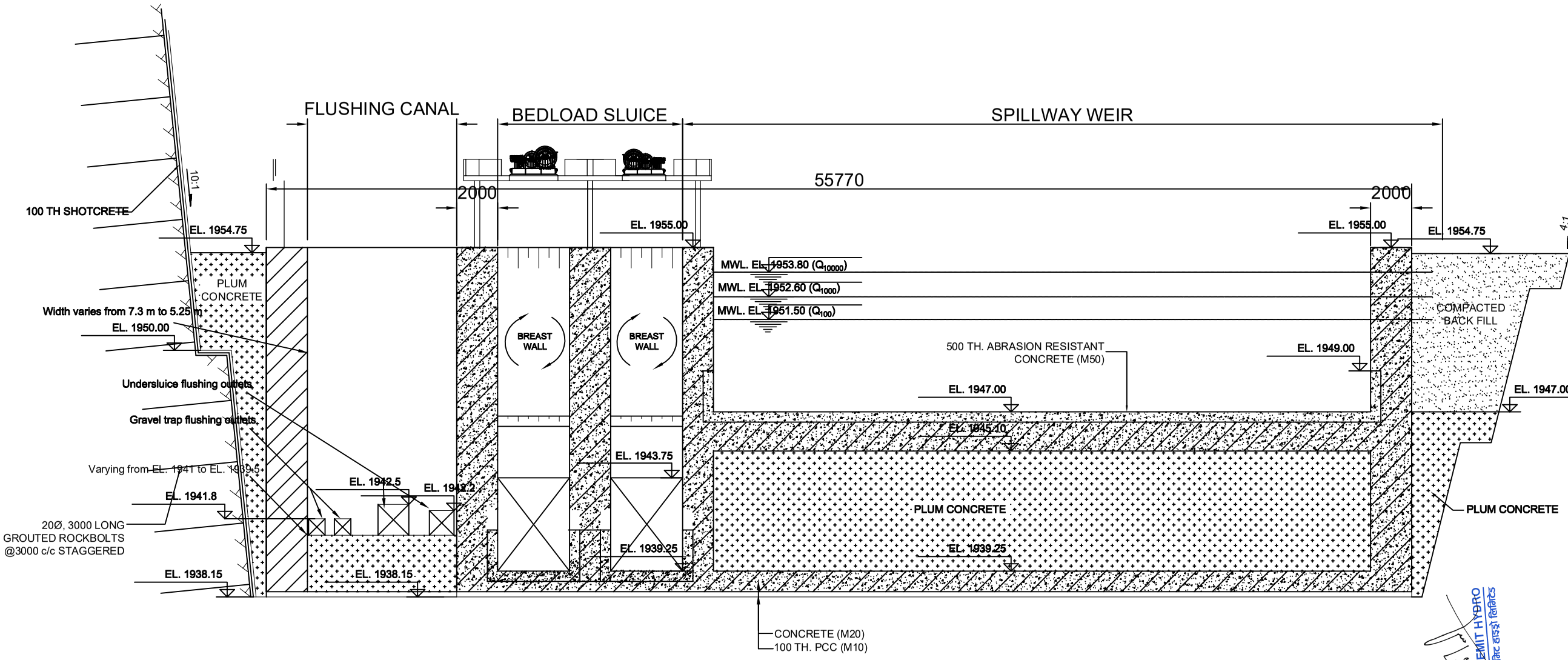
Designed:			
Drawn:	Kunjan Lal Pradhan		
Checked:	Umesh. Singh		
Approved by	Meg. B. Bishwakarma		

Contract No.

Scale: Not in scale

Drawing No.: A–FRHD_4

Revision



Section D : Sectional view across the diversion weir

Rev.	Amendment	By	Ckd.	Rec.	App.

JV of Indo Canadian Consultancy Services
Limited & Shah Consult International (P.) Ltd.

GHUNSA KHOLA HEP

WEIR SECTION VIEW
FINAL R. H. DESIGN SECTION VIEW

HYDRO LAB Pvt. Ltd.
Krishna Galli, Pulchowk
GP.O. Box 21093, Kathmandu, Nepal
Tel: + 977 1 5539185 / 5539186 / 5535740
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	Name	Initials	Date
Designed:			
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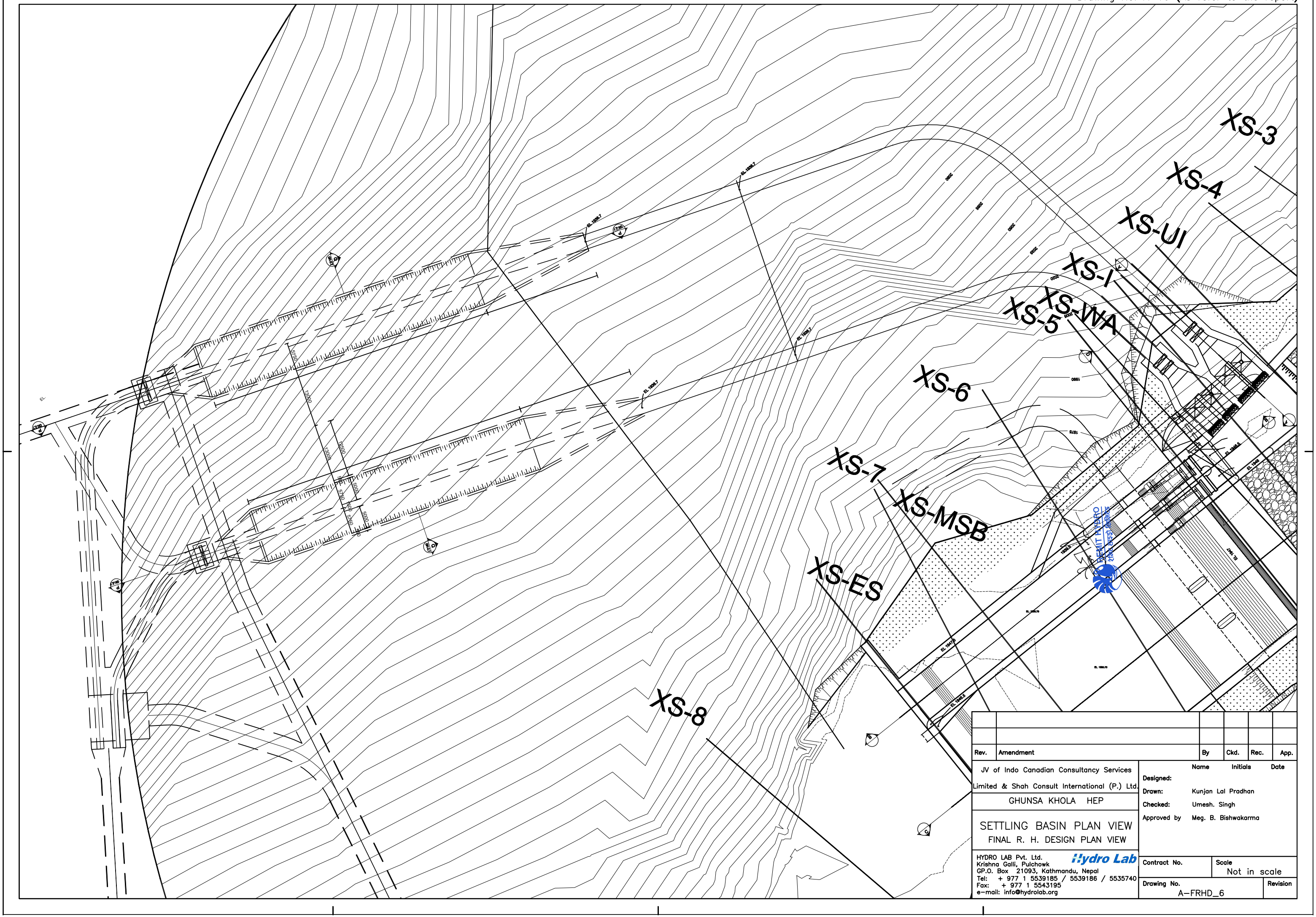
Checked:	Umesh. Singh		
Approved by	Meg. B. Bishwakarma		

Contract No.	Scale
	Not in scale
Drawing No. A–FRHD_5	Revision

Drawing No: A–16 (As refer to the report)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XS-4
XS-UI
XS-I
XS-WA
XS-5
XS-6
XS-7
XS-MSB
XS-ES
XS-8
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EL. 1968.7
EL. 1968.7
EL. 1968.7
EL. 1940
EL. 1947.5
EL. 1944.5
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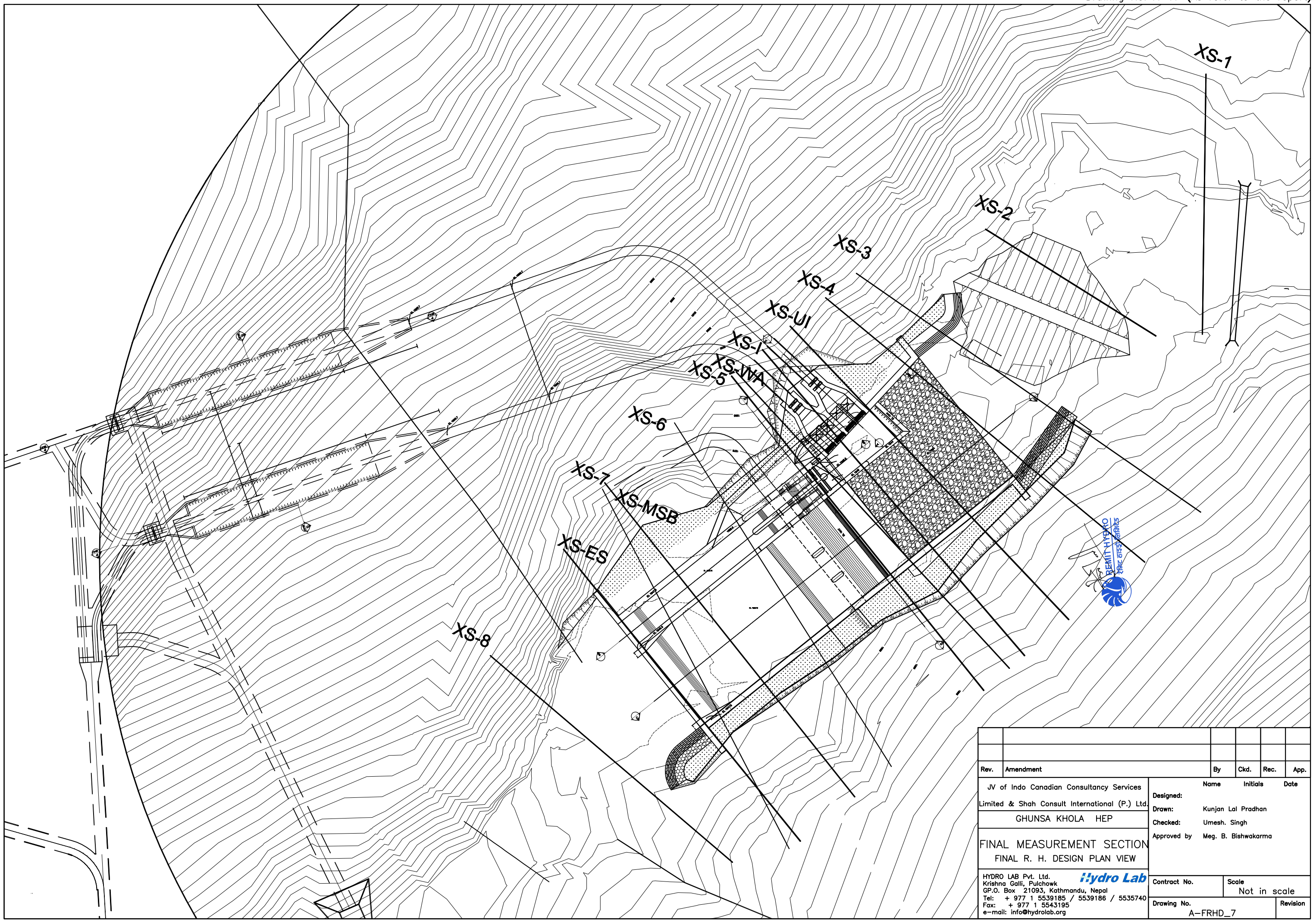
Drawing No: A–17 (As refer to the report)



Rev.	Amendment	By	Ckd.	Rec.	App.

JV of Indo Canadian Consultancy Services
Limited & Shah Consult International (P.) Ltd.

GHUNSA KHOLA HEP

FINAL MEASUREMENT SECTION
FINAL R. H. DESIGN PLAN VIEW

	Name	Initials	Date
Designed:			
Drawn:	Kunjan Lal Pradhan		
Checked:	Umesh. Singh		
Approved by	Meg. B. Bishwakarma		

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Hydro Lab

Contract No.	Scale
	Not in scale
Drawing No. A–FRHD_7	Revision

APPENDIX B
Figures

Physical Hydraulic Model Study of the Headworks of Ghunsa Khola HEP (78 MW)

Final Report

December 2020

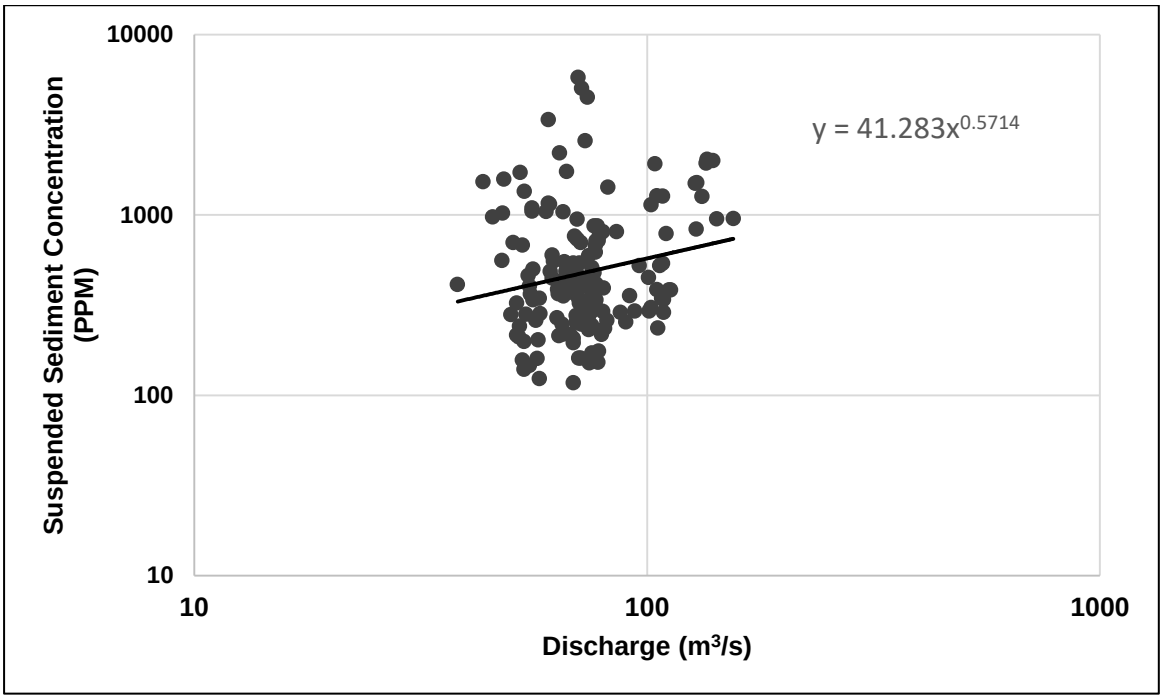

Figure B- 1: Recommended Sediment Rating Curve

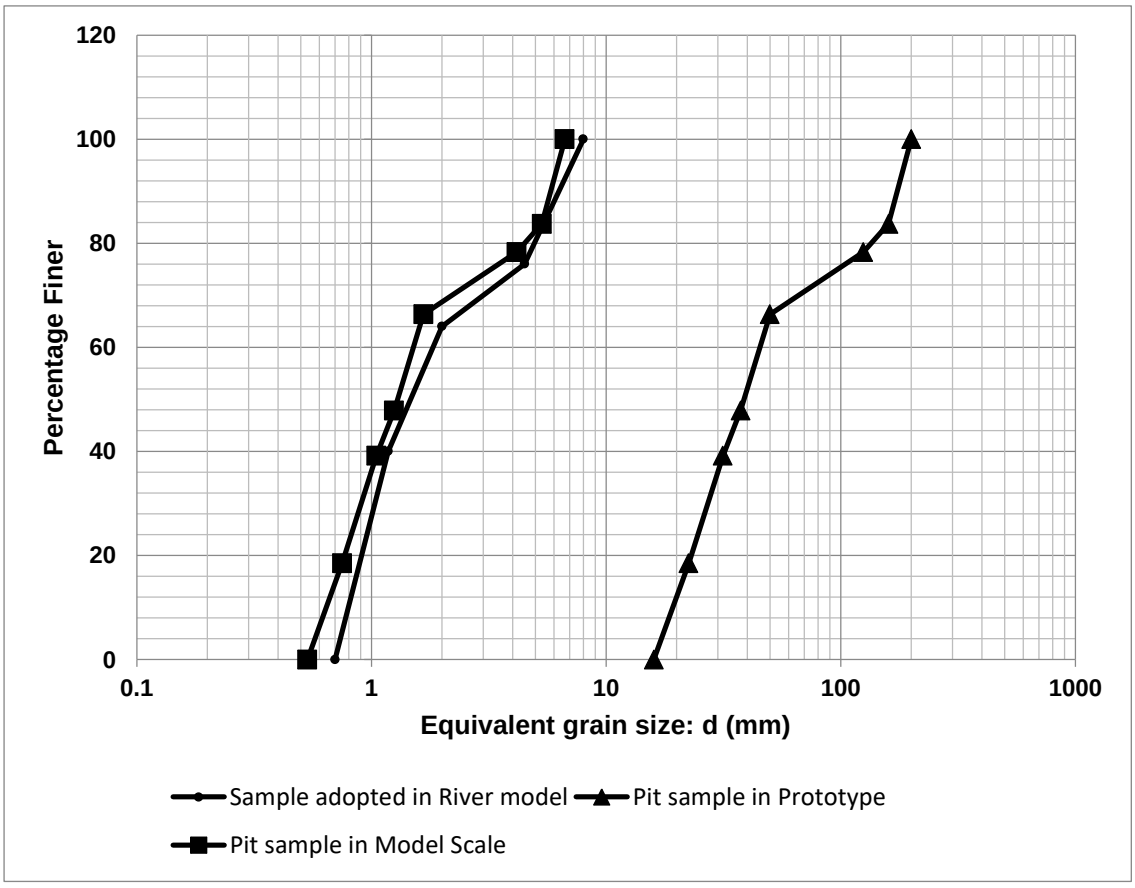

Figure B- 2: Adopted particle size distribution curve PSD, (Bed load)

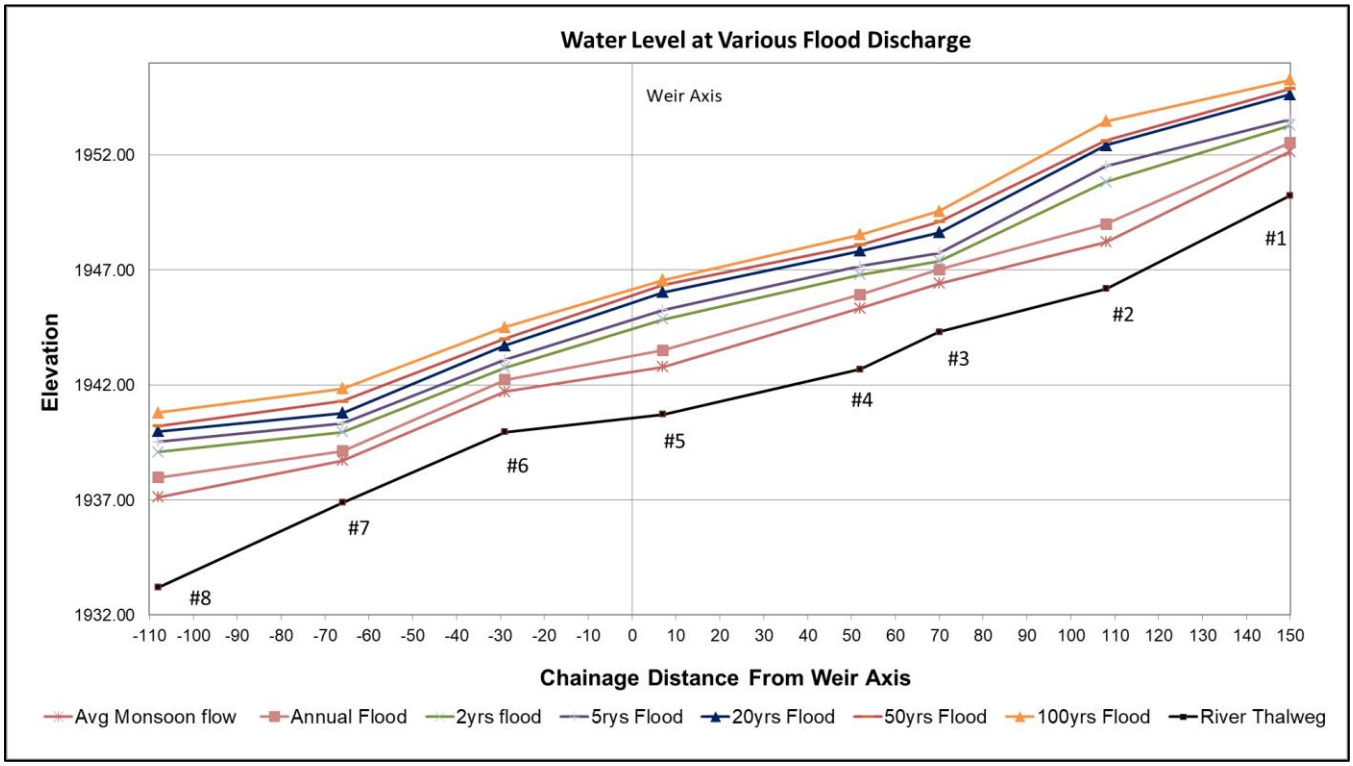

Figure B- 3: Avg. Flow depth of the sections at selected steady flow, (existing situation test)

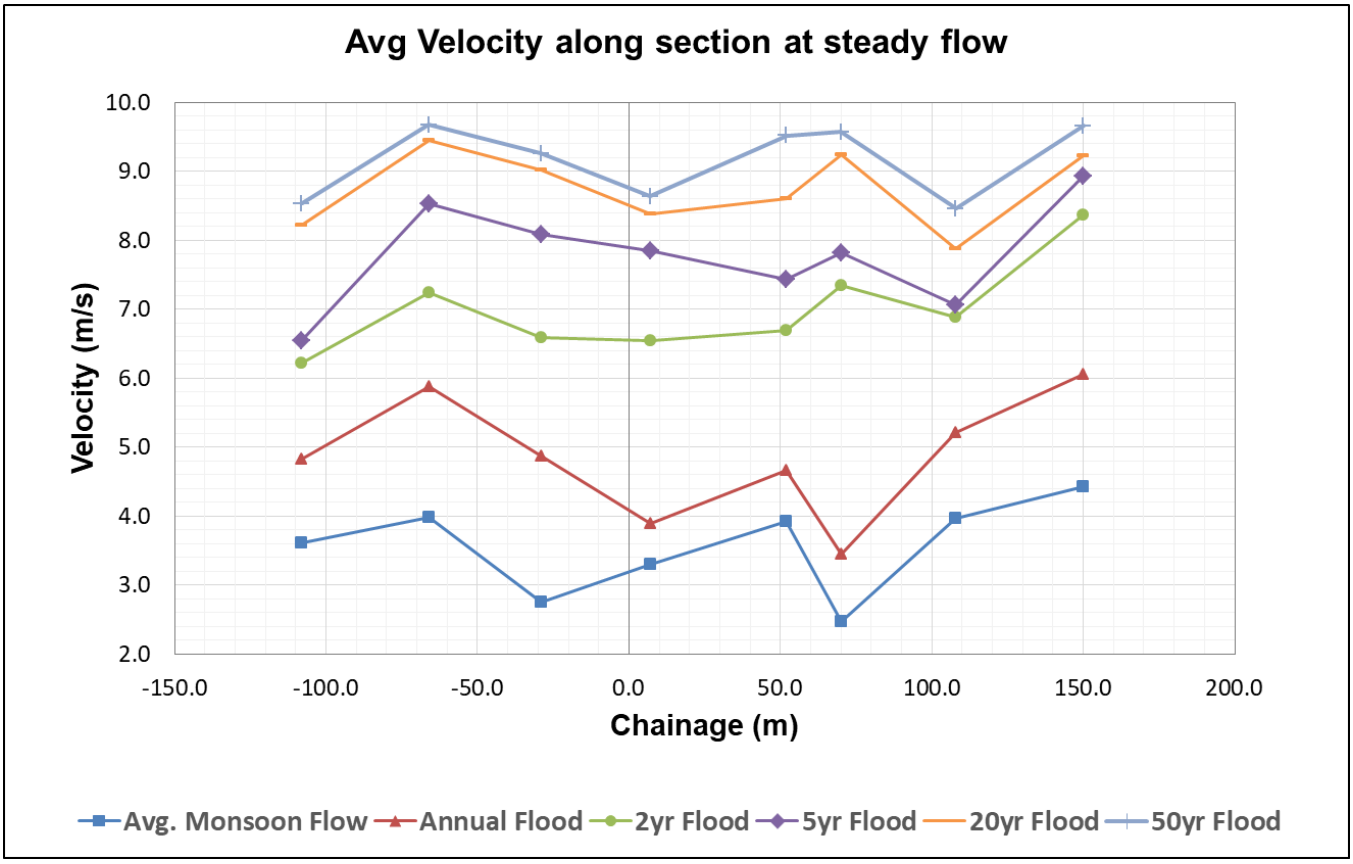

Figure B- 4: Avg. velocity along the sections at selected steady flow, (existing situation test)

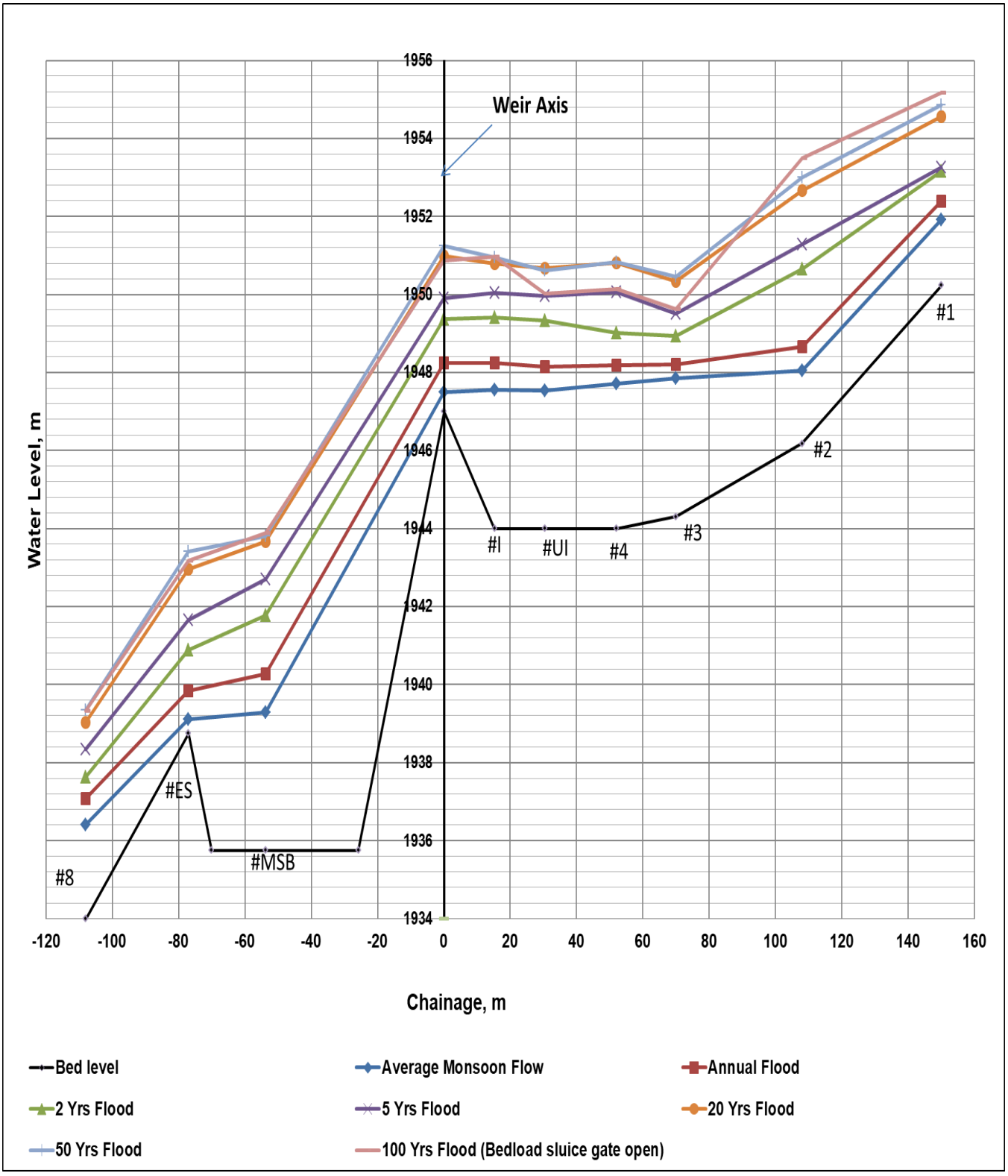

Figure B- 5: Avg. Flow depth of the sections at selected steady flow, (Final Recommended Hydraulic Design test)

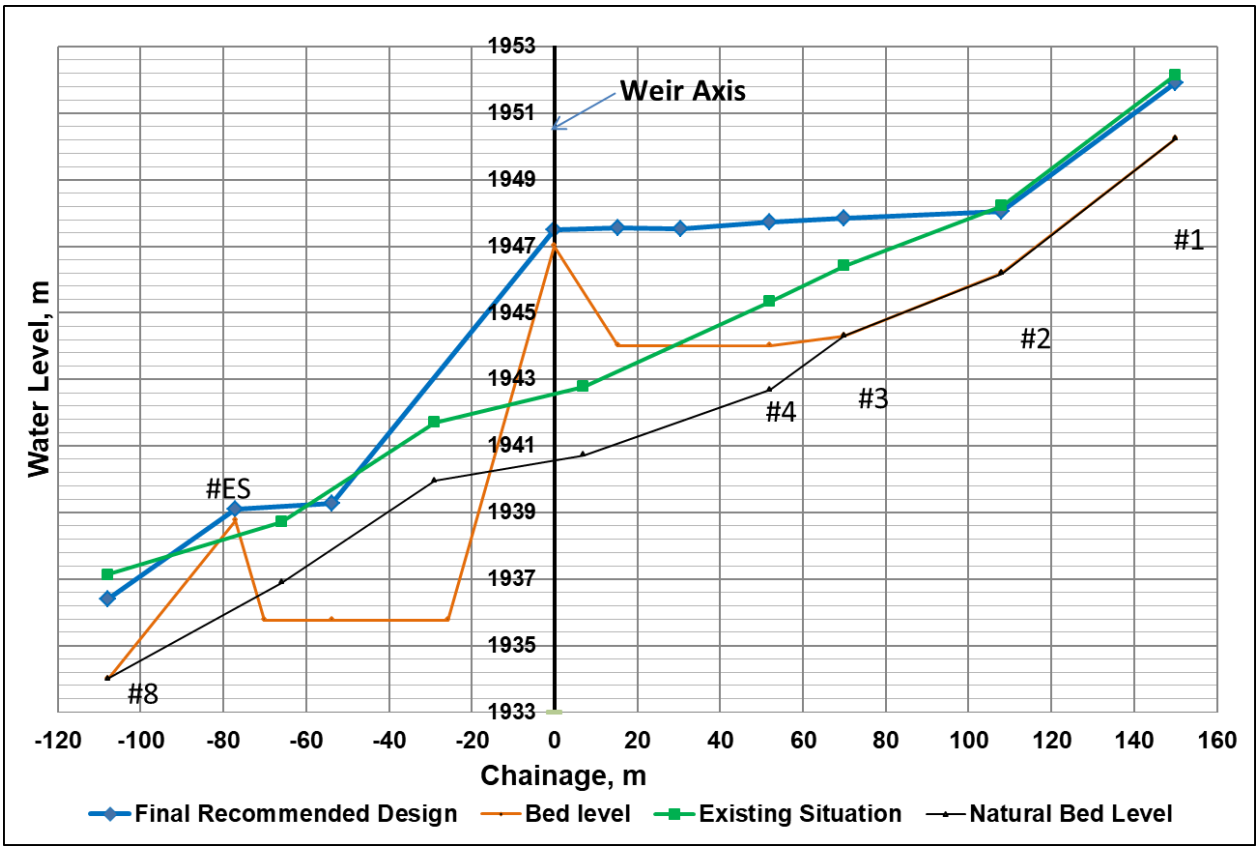

Figure B- 6: Water level profile for Average monsoon flow (Existing situation and Final Recommended Design)

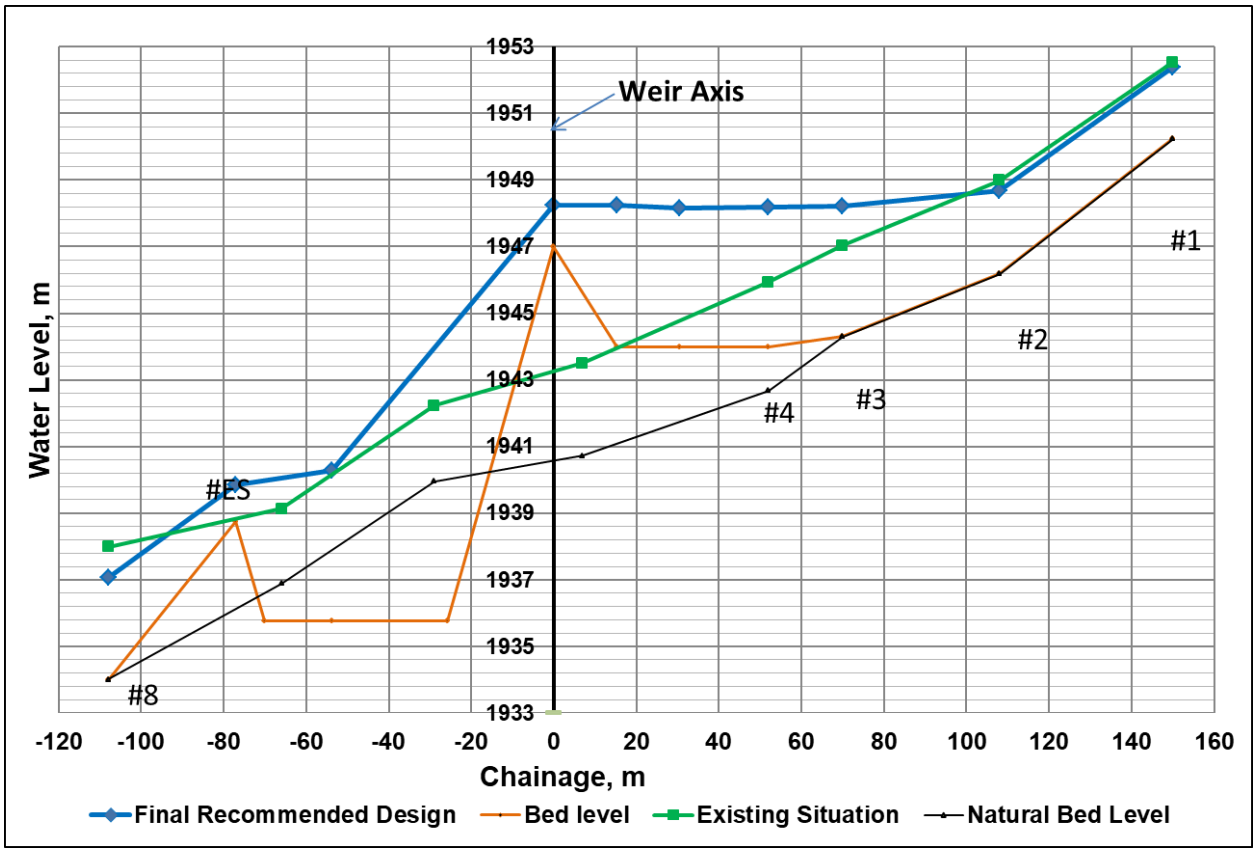

Figure B- 7: Water level profile for annual flood (Existing situation and Final Recommended Design)

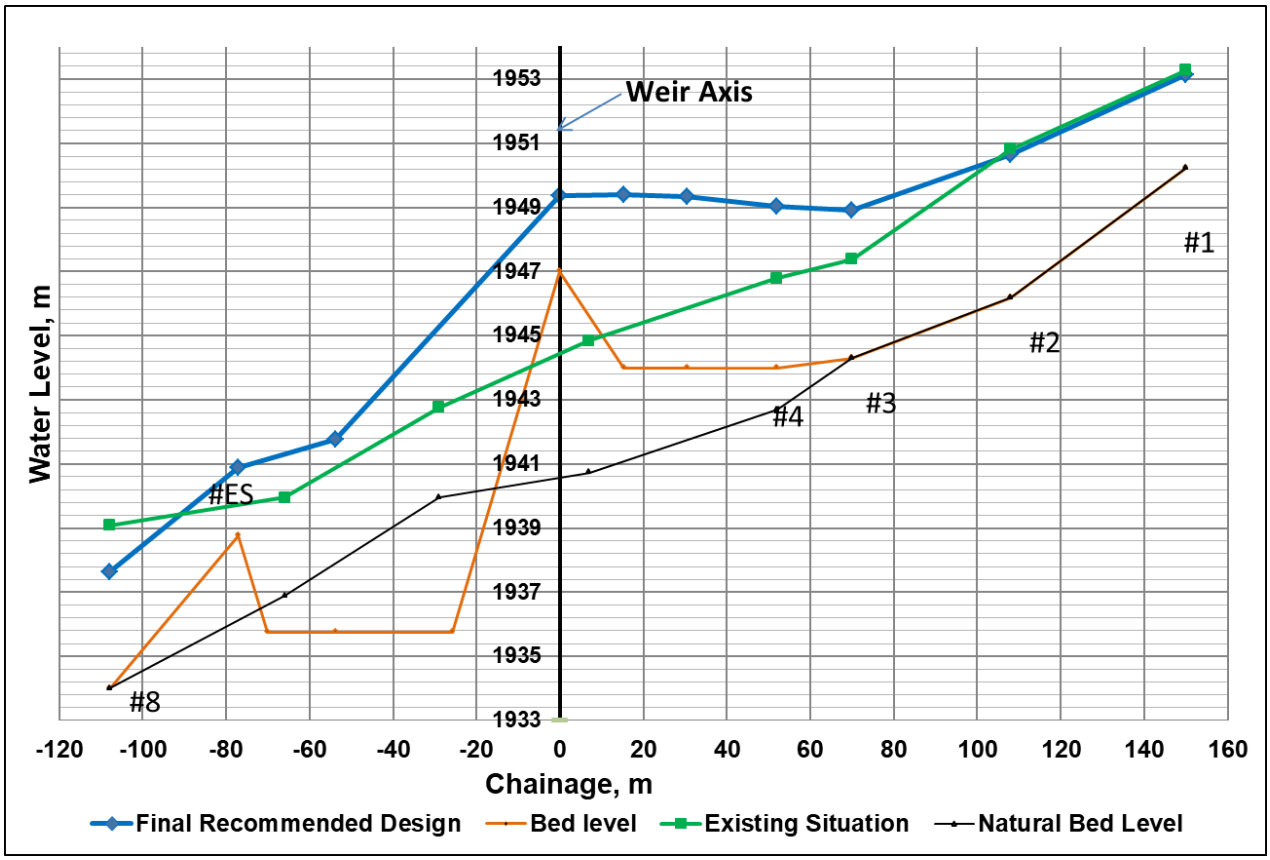

Figure B- 8: Water level profile for 2 years flood (Existing situation and Final Recommended Design)

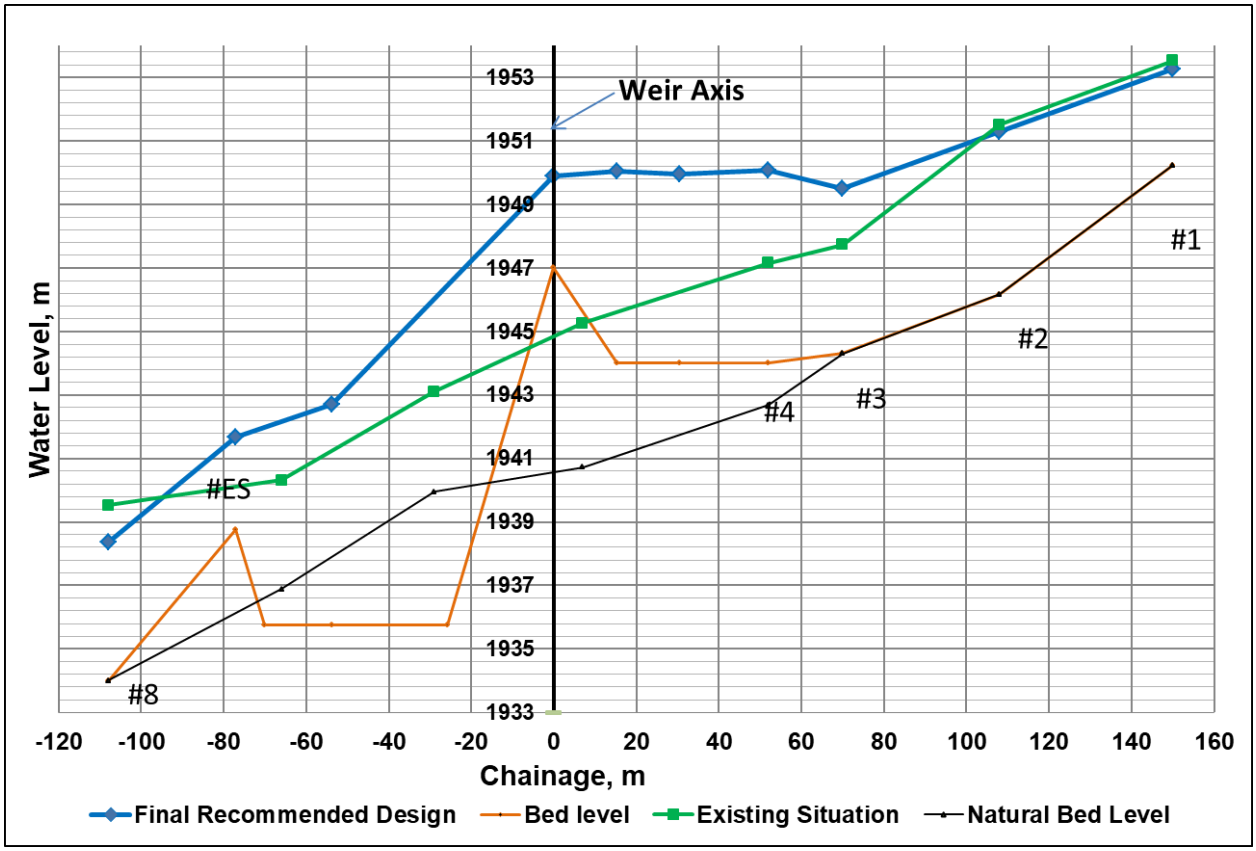

Figure B- 9: Water level profile for 5 years flood (Existing situation and Final Recommended Design)

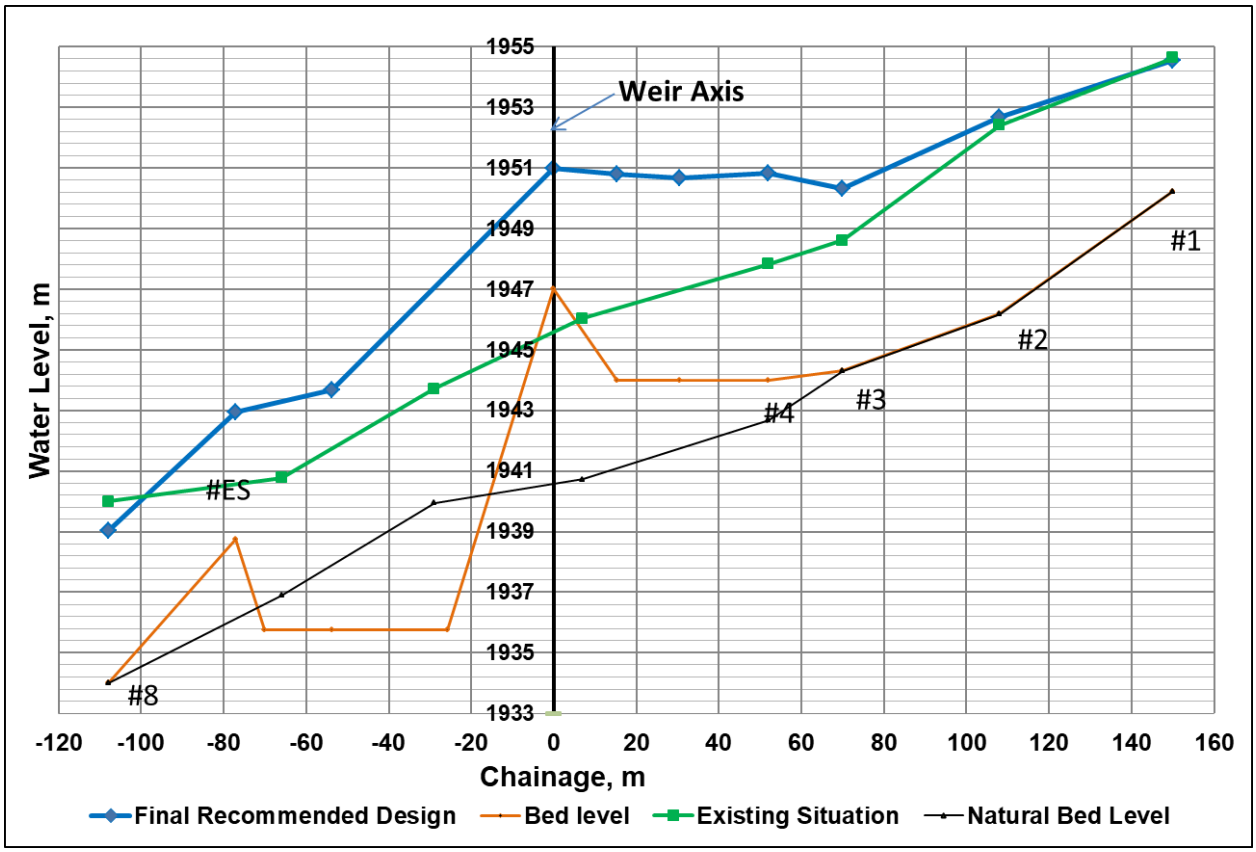

Figure B- 10: Water level profile for 20 years flood (Existing situation and Final Recommended Design)

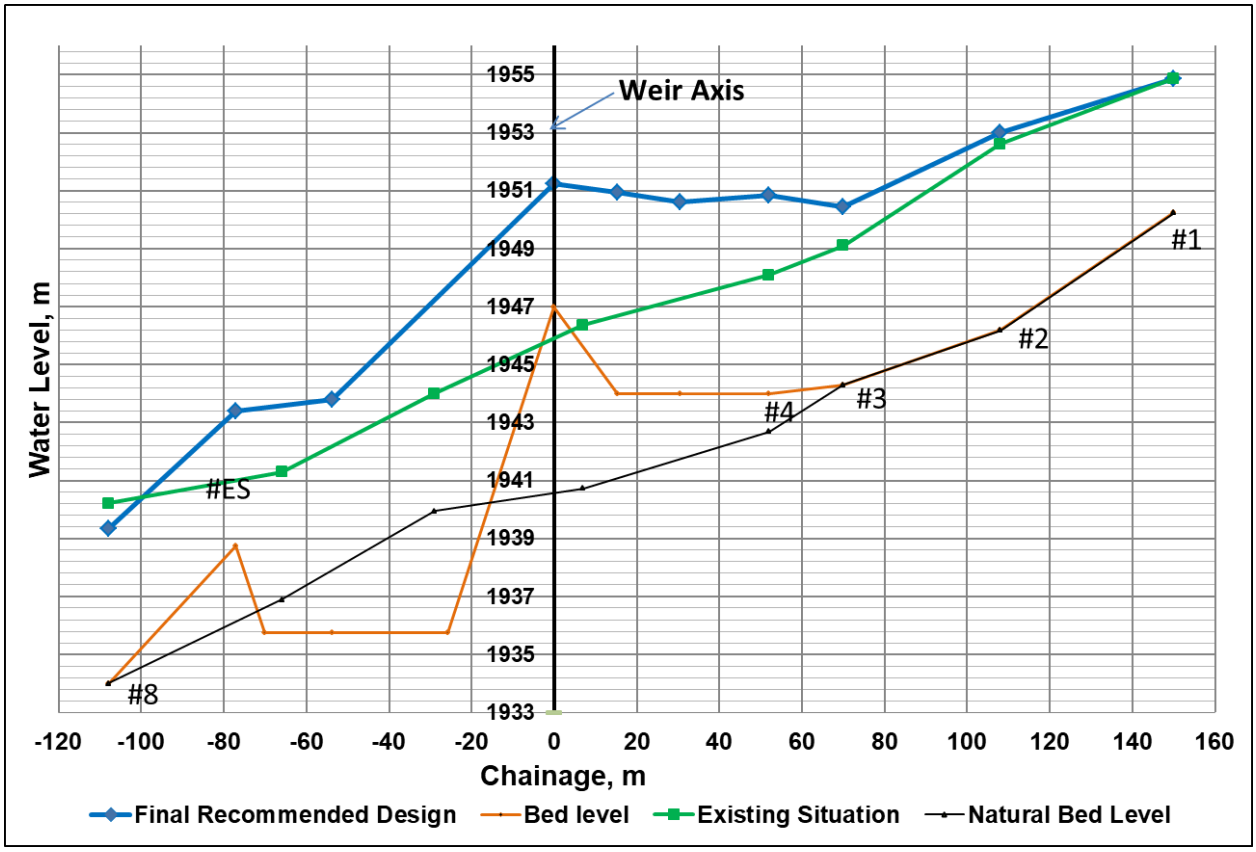

Figure B- 11: Water level profile for 50 years flood (Existing situation and Final Recommended Design)

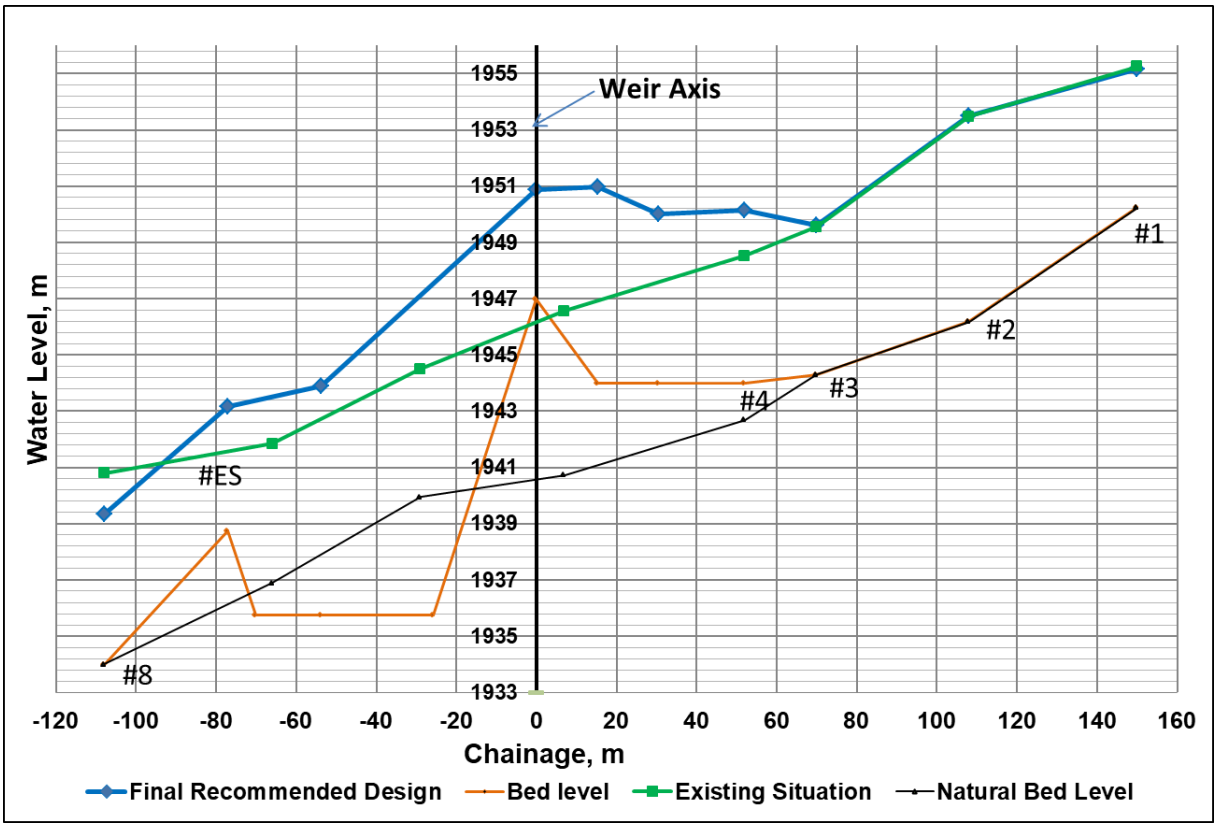

Figure B- 12: Water level profile for 100 years flood (Existing situation and Final Recommended Design with bedload flushing gate open)

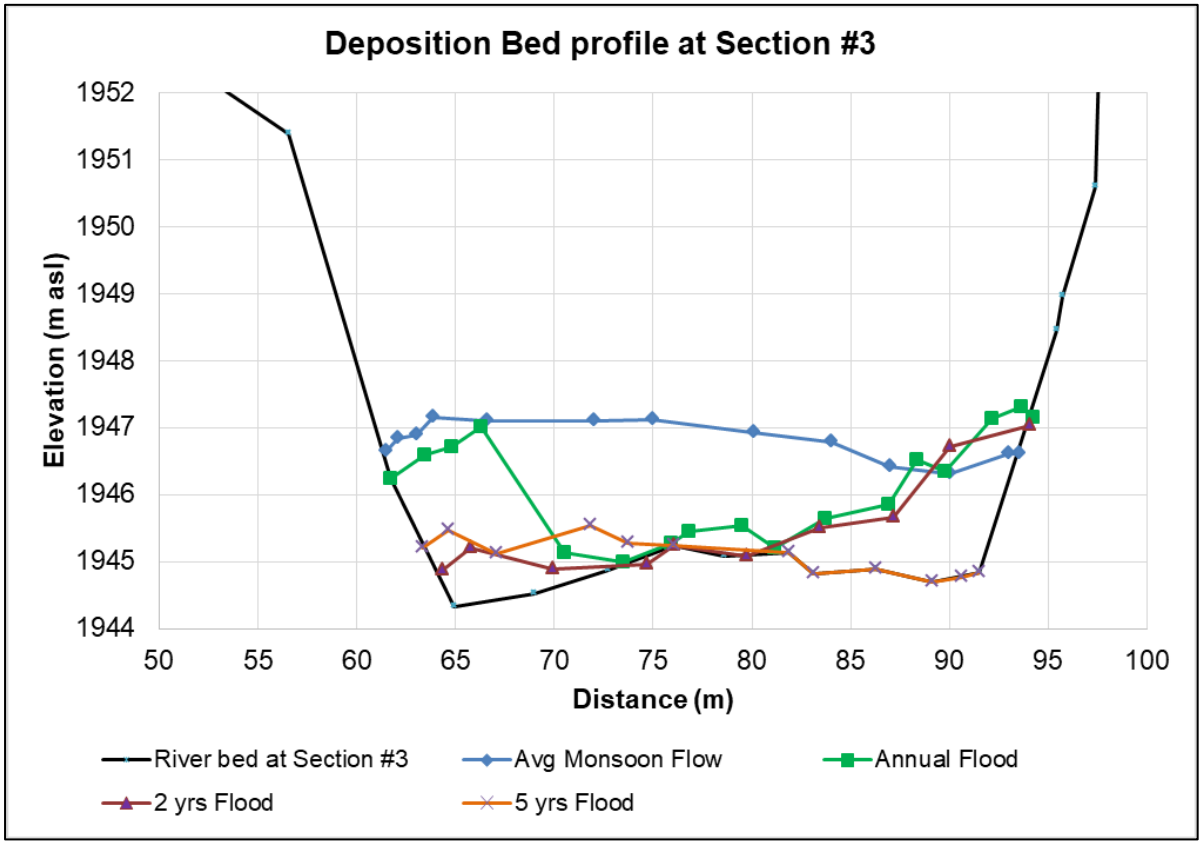

Figure B- 13: D/S view of Bed level profile at Section #3 at various year floods for Final Recommended Design

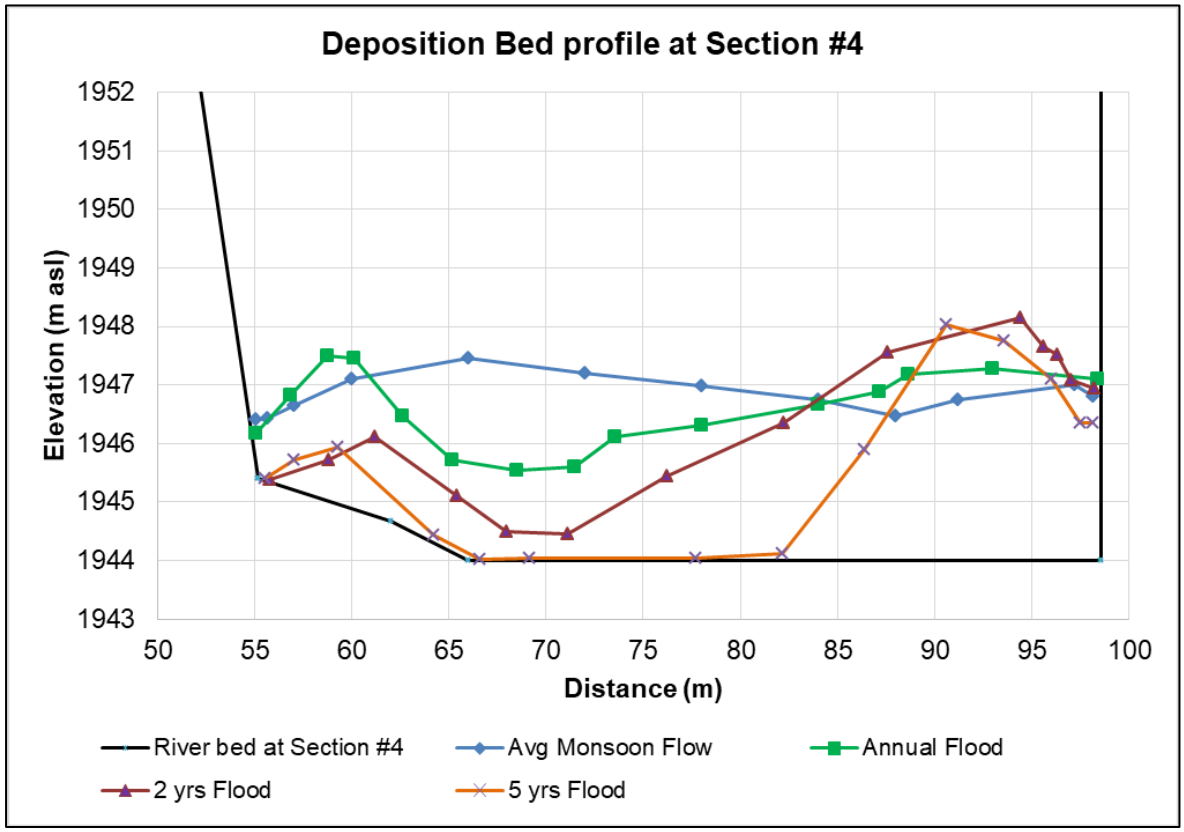

Figure B- 14: D/S view of Bed level profile at Section #4 at various year floods for Final Recommended Design

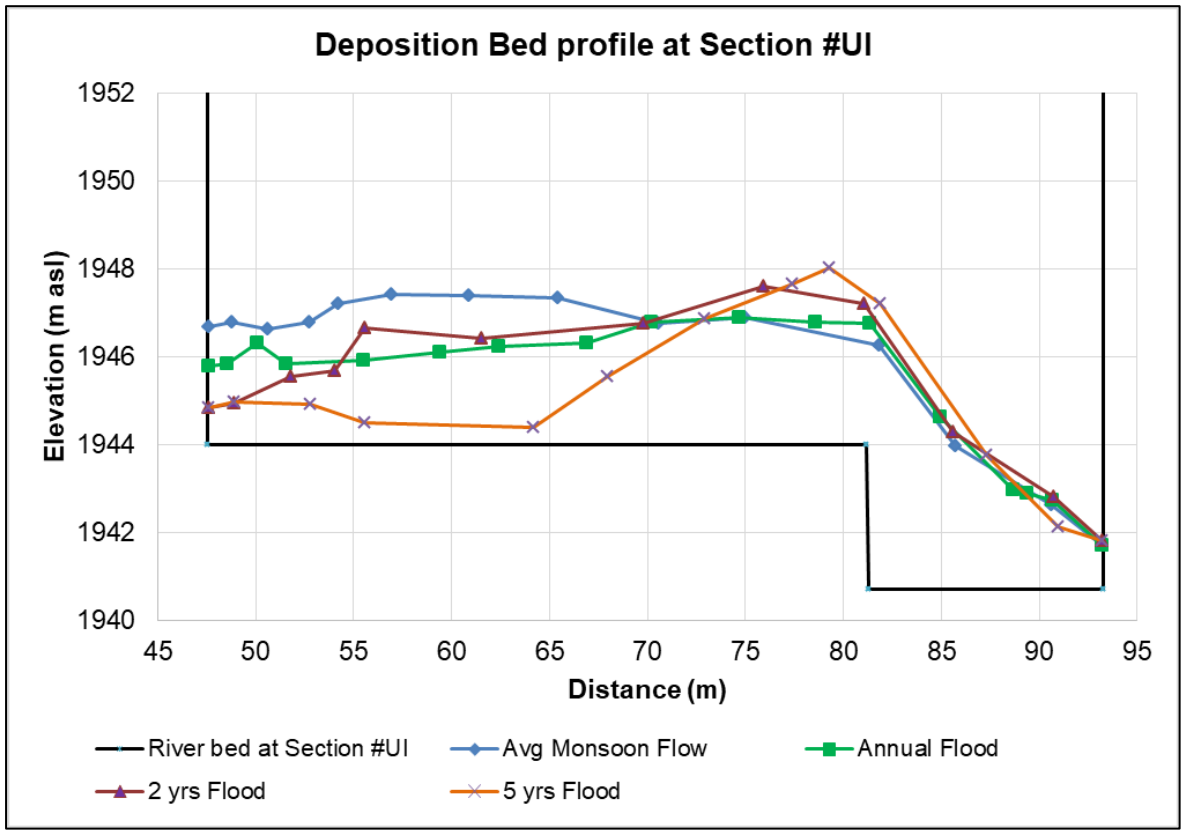

Figure B- 15: D/S view of Bed level profile at Section #UI at various year floods for Final Recommended Design

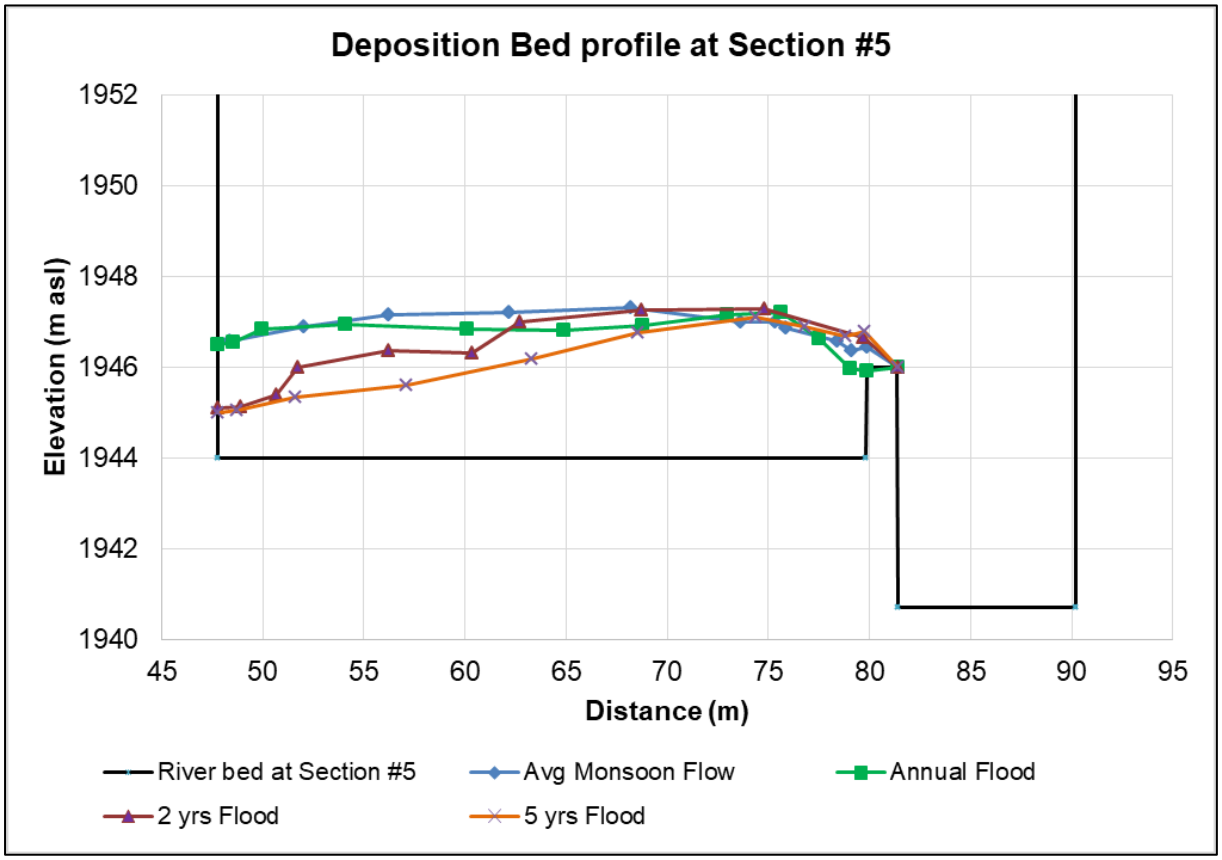

Figure B- 16: D/S view of Bed level profile at Section #5 at various year floods for Final Recommended Design

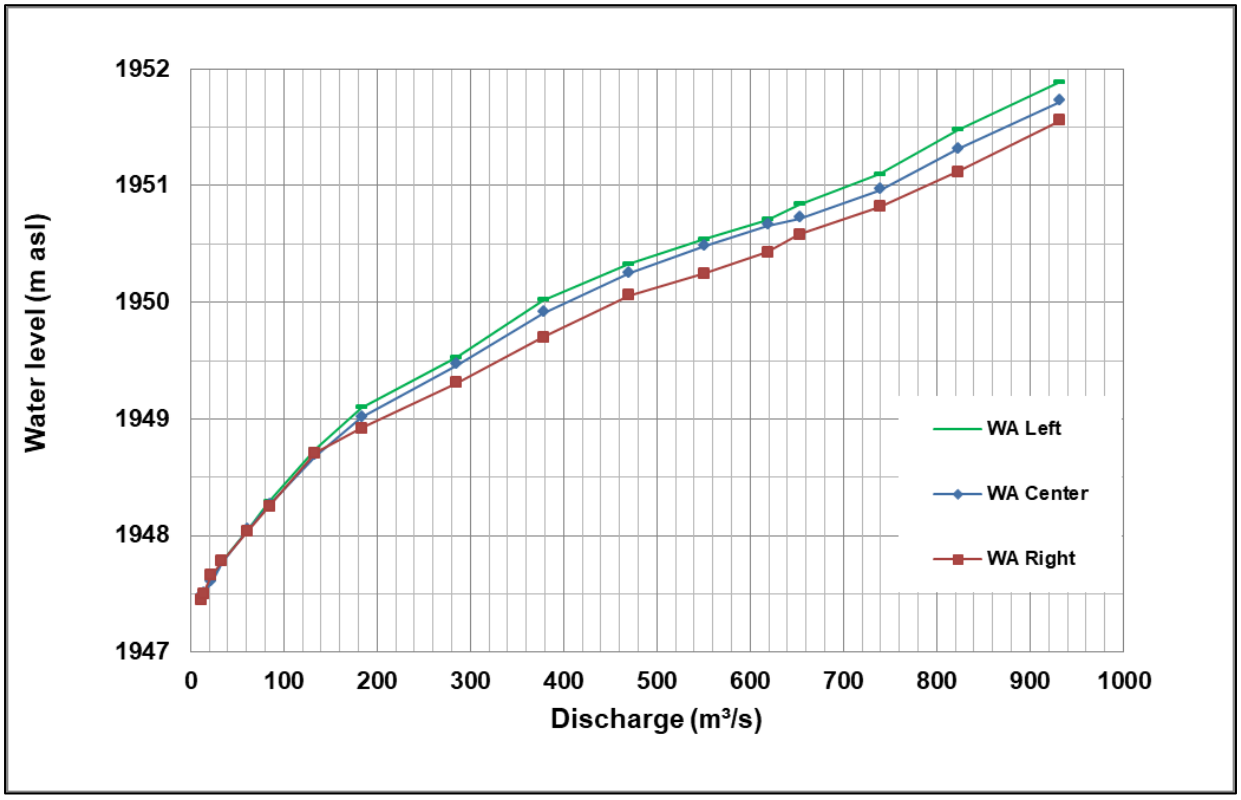

Figure B- 17: Rating curve for weir axis, data collected at Weir Axis (Final Recommended design)

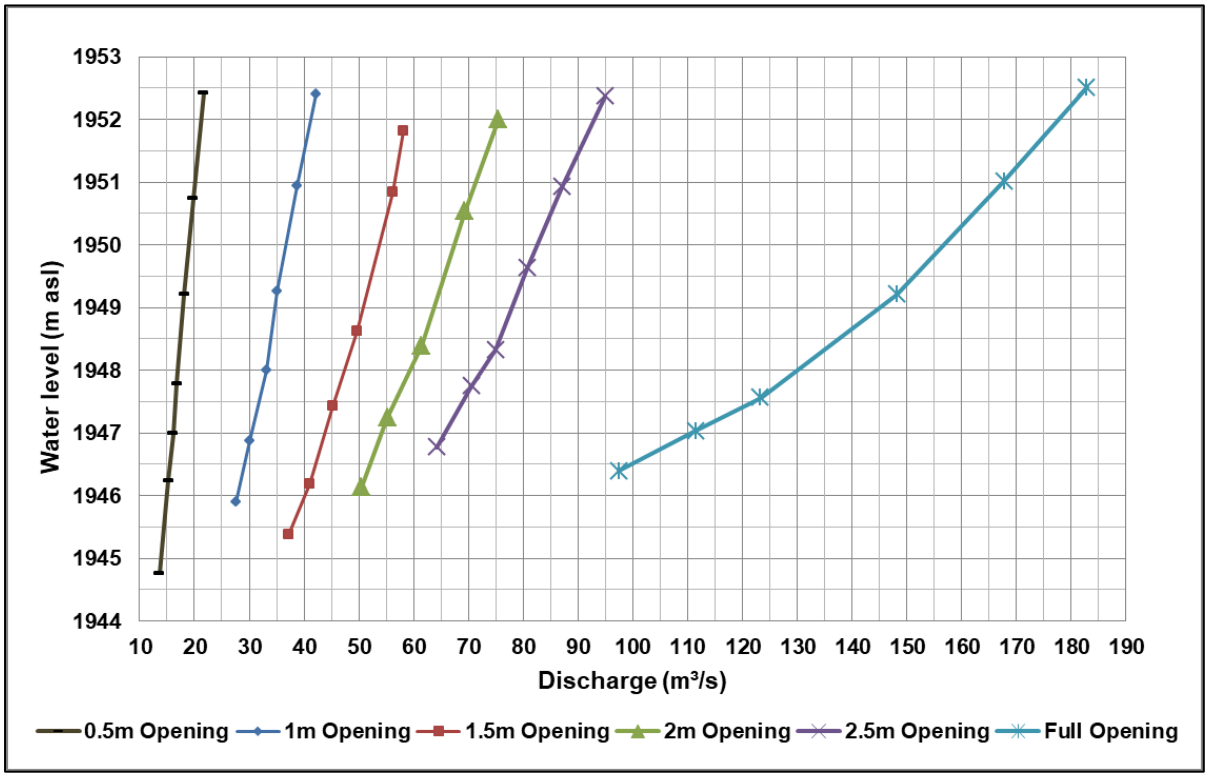

Figure B- 18: Rating curve for Bedload Flushing gate, data collected at Intake section (#XS-I) (Final Recommended design)

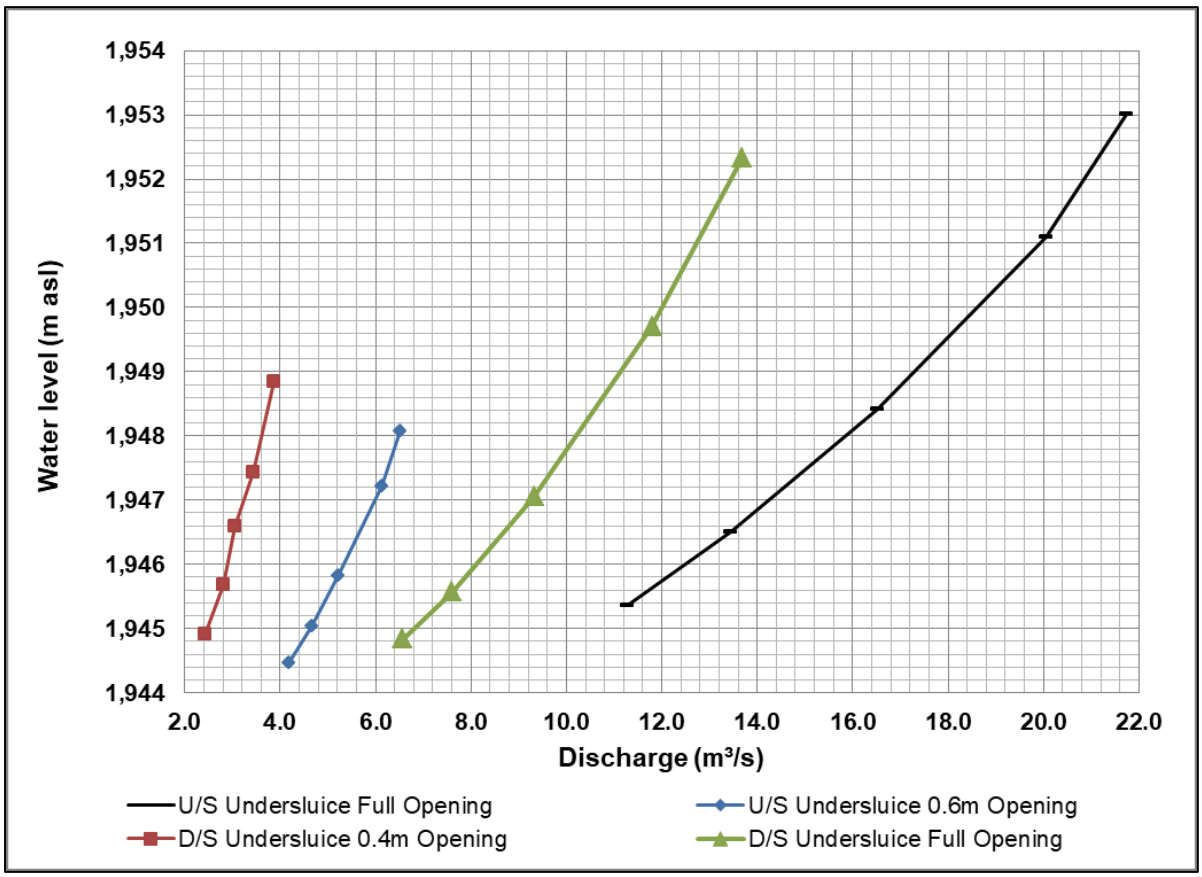

Figure B- 19: Rating curve for Undersluice flushing gate, data collected at Intake section (#XS-I) (Final Recommended design)

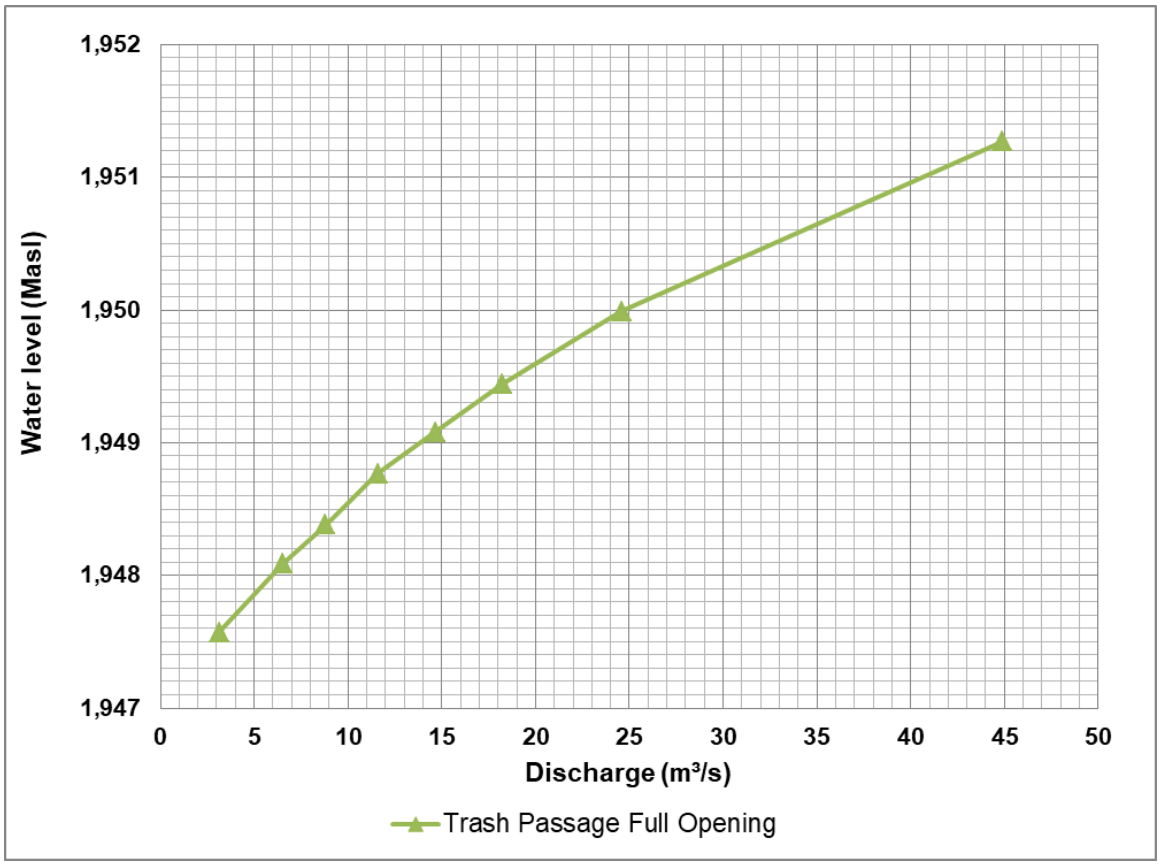

Figure B- 20: Rating curve for Trash Passage, data collected at Intake section (#XS-I) (Final Recommended design)

APPENDIX C
Tables

Physical Hydraulic Model Study of the Headworks of Ghunsa Khola HEP (78 MW)

Final Report

December 2020

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Table C- 1: List of input data received from the Client for the model study

S. No.	Description	Data/ documents received (Yes or No)	Responsible party	Remarks
1	Topographical survey map of the river stretch being modelled in Auto CAD	Yes	The Client	
2	River Cross-sections	Yes	The Client	
3	Boulder Data	Yes	The Client	
4	Particle size distribution curves of bed material obtained from test pit excavations	Yes	The Client	
5	Discharge vs Suspended Sediment rating curve	Yes	The Client	
6	Design and drawings of the headworks structures in AutoCAD format	Yes	The Client	
7	Hydrological report including the design discharge, flood discharges including design flood and water level at known high discharge required for model validation.	Yes	The Client	HEC_RAS model results provided for model validation

Table C- 2: Measured water levels in the river model at 75 m^3/s during validation

Measurement sections	Chainage from Weir Axis, m	Measured water levels at final attempt, masl				HEC-RAS water level, masl	Deviation, m
		Right	Centre	Left	Average		
XS-3	70	1946.59	1946.32	1946.35	1946.42	1946.5	0.08
XS-5	7	1942.84	1942.96	1942.54	1942.78	1942.71	-0.07
XS-6	-29	1941.79	1941.67	1941.67	1941.71	1942.05	0.34
XS-7	-66	1938.82	1938.67	1938.64	1938.71	1938.6	-0.11

Table C- 3: Measured water levels in the river model at 164 m^3/s during validation

Measurement sections	Chainage from Weir Axis, m	Measured water levels at final attempt, masl				HEC-RAS water level, masl	Deviation, m
		Right	Centre	Left	Average		
XS-3	70	1947.22	1947.025	1946.83	1947.025	1947.32	0.295
XS-5	7	1943.665	1943.62	1943.245	1943.51	1943.3	-0.21
XS-6	-29	1942.36	1942.24	1942.09	1942.23	1942.57	0.34
XS-7	-66	1939.33	1938.925	1939.12	1939.125	1939.07	-0.055

Table C- 4: Measured water levels in the river model at 593 m^3/s during validation

Measurement sections	Chainage from Weir Axis, m	Measured water levels at final attempt, masl				HEC-RAS water level, masl	Deviation, m
		Right	Centre	Left	Average		
XS-3	70	1948.9	1948.27	1947.04	1948.07	1947.73	-0.34
XS-5	7	1945.72	1945.27	1945.33	1945.44	1945.2	-0.24
XS-6	-29	1943.44	1943.59	1943.605	1943.545	1943.85	0.305
XS-7	-66	1941.04	1940.17	1940.41	1940.54	1940.53	-0.01

Table C- 5: Observed water levels during average monsoon flow (75 m^3/s), existing situation test

Section at	Chainage, (m)	Water Levels, (masl)				Remarks
		Right Bank	Centre	Left Bank	Avg	
XS-1	150.00	1952.23	1952.20	1952.02	1952.15	
XS-2	108.00	1948.21	1948.03	1948.39	1948.21	
XS-3	70.00	1946.59	1946.32	1946.35	1946.42	
XS-4	52.00	1945.36	1945.36	1945.30	1945.34	
XS-5	7.00	1942.84	1942.96	1942.54	1942.78	
XS-6	-29.00	1941.79	1941.67	1941.67	1941.71	
XS-7	-66.00	1938.82	1938.67	1938.64	1938.71	
XS-8	-108.00	1937.14	1937.08	1937.17	1937.13	

Table C- 6: Observed water levels during annual flood (164 m^3/s), existing situation test

Section at	Chainage, (m)	Water Levels, (masl)				Remarks
		Right Bank	Centre	Left Bank	Avg	
XS-1	150.00	1952.77	1952.32	1952.53	1952.54	
XS-2	108.00	1948.90	1948.72	1949.35	1948.99	
XS-3	70.00	1947.22	1947.03	1946.83	1947.03	
XS-4	52.00	1945.90	1945.69	1946.20	1945.93	
XS-5	7.00	1943.67	1943.62	1943.25	1943.51	
XS-6	-29.00	1942.36	1942.24	1942.09	1942.23	
XS-7	-66.00	1939.33	1938.93	1939.12	1939.13	
XS-8	-108.00	1937.47	1937.89	1938.58	1937.98	

Table C- 7: Observed water levels during two years flood (364 m^3/s), existing situation test

Section at	Chainage, (m)	Water Levels, (masl)				Remarks
		Right Bank	Centre	Left Bank	Avg	
XS-1	150.00	1954.12	1953.07	1952.68	1953.29	
XS-2	108.00	1949.68	1950.07	1952.71	1950.82	
XS-3	70.00	1948.27	1947.40	1946.47	1947.38	
XS-4	52.00	1947.40	1946.50	1946.50	1946.80	
XS-5	7.00	1945.27	1944.58	1944.70	1944.85	
XS-6	-29.00	1943.32	1942.69	1942.27	1942.76	
XS-7	-66.00	1940.17	1939.81	1939.90	1939.96	
XS-8	-108.00	1938.22	1938.67	1940.35	1939.08	

Table C- 8: Observed water levels during five years flood (502 m^3/s), existing situation test

Section at	Chainage, (m)	Water Levels, (masl)				Remarks
		Right Bank	Centre	Left Bank	Avg	
XS-1	150.00	1953.97	1953.22	1953.40	1953.53	
XS-2	108.00	1949.56	1950.52	1954.45	1951.51	
XS-3	70.00	1948.78	1948.09	1946.35	1947.74	
XS-4	52.00	1948.51	1946.50	1946.47	1947.16	

XS-5	7.00	1945.39	1945.21	1945.18	1945.26	
XS-6	-29.00	1944.07	1942.75	1942.48	1943.10	
XS-7	-66.00	1940.68	1939.93	1940.35	1940.32	
XS-8	-108.00	1938.64	1938.52	1941.43	1939.53	

Table C- 9: Observed water levels during twenty years flood (681 m^3/s), existing situation test

Section at	Chainage, (m)	Water Levels, (masl)				Remarks
		Right Bank	Centre	Left Bank	Avg	
XS-1	150.00	1955.02	1954.36	1954.51	1954.63	
XS-2	108.00	1949.50	1951.57	1956.16	1952.41	
XS-3	70.00	1949.62	1948.33	1947.91	1948.62	
XS-4	52.00	1949.98	1946.83	1946.68	1947.83	
XS-5	7.00	1946.83	1945.78	1945.48	1946.03	
XS-6	-29.00	1945.21	1943.14	1942.81	1943.72	
XS-7	-66.00	1941.16	1940.35	1940.83	1940.78	
XS-8	-108.00	1938.91	1939.00	1942.06	1939.99	

Table C- 10: Observed water levels during Fifty years flood (794 m^3/s), existing situation test

Section at	Chainage, (m)	Water Levels, (masl)				Remarks
		Right Bank	Centre	Left Bank	Avg	
XS-1	150.00	1955.23	1954.75	1954.66	1954.88	
XS-2	108.00	1950.43	1950.55	1956.88	1952.62	
XS-3	70.00	1949.83	1948.78	1948.69	1949.10	
XS-4	52.00	1950.40	1947.07	1946.83	1948.10	
XS-5	7.00	1947.46	1945.87	1945.75	1946.36	
XS-6	-29.00	1945.33	1943.68	1943.02	1944.01	
XS-7	-66.00	1941.88	1940.62	1941.40	1941.30	
XS-8	-108.00	1938.70	1939.33	1942.63	1940.22	

Table C- 11: Observed water levels during hundred years flood (880 m^3/s), existing situation test

Section at	Chainage, (m)	Water Levels, (masl)				Remarks
		Right Bank	Centre	Left Bank	Avg	
XS-1	150.00	1955.59	1955.11	1955.08	1955.26	
XS-2	108.00	1952.26	1951.00	1957.15	1953.47	
XS-3	70.00	1949.95	1949.44	1949.29	1949.56	
XS-4	52.00	1950.91	1947.67	1947.01	1948.53	
XS-5	7.00	1947.40	1946.20	1946.08	1946.56	
XS-6	-29.00	1946.11	1944.25	1943.17	1944.51	
XS-7	-66.00	1942.90	1940.77	1941.85	1941.84	
XS-8	-108.00	1939.48	1939.87	1943.02	1940.79	

Table C- 12: Observed velocities during average monsoon flow (75 m^3/s), existing situation test

Section at	Chainage, (m)	Velocity, (m/s)				Remarks
		Right Bank	Centre	Left Bank	Avg	
XS-1	150.00	3.33	5.33	4.61	4.42	
XS-2	108.00	5.40	4.04	2.48	3.97	
XS-3	70.00	1.11	3.53	2.77	2.47	
XS-4	52.00	2.75	3.86	5.15	3.92	
XS-5	7.00	2.83	4.33	2.74	3.30	
XS-6	-29.00	1.11	3.85	3.30	2.75	
XS-7	-66.00	1.77	5.00	5.16	3.98	
XS-8	-108.00	4.70	3.09	3.03	3.61	

Table C- 13: Observed velocities during annual flood (164 m^3/s), existing situation test

Section at	Chainage, (m)	Velocity, (m/s)				Remarks
		Right Bank	Centre	Left Bank	Avg	
XS-1	150.00	2.51	8.27	7.38	6.05	
XS-2	108.00	7.26	5.57	2.81	5.21	
XS-3	70.00	1.83	4.55	3.97	3.45	
XS-4	52.00	2.44	5.53	6.02	4.66	
XS-5	7.00	4.33	3.60	3.74	3.89	
XS-6	-29.00	3.31	6.81	4.51	4.87	
XS-7	-66.00	4.55	6.85	6.24	5.88	
XS-8	-108.00	5.53	4.24	4.68	4.82	

Table C- 14: Observed velocities during two years flood (364 m^3/s), existing situation test

Section at	Chainage, (m)	Velocity, (m/s)				Remarks
		Right Bank	Centre	Left Bank	Avg	
XS-1	150.00	5.49	10.50	9.09	8.36	
XS-2	108.00	8.16	7.41	5.07	6.88	
XS-3	70.00	7.34	8.05	6.64	7.34	
XS-4	52.00	5.01	6.98	8.09	6.70	
XS-5	7.00	5.79	7.97	5.86	6.54	
XS-6	-29.00	3.97	8.24	7.54	6.59	
XS-7	-66.00	6.89	9.06	5.76	7.24	
XS-8	-108.00	7.79	6.01	4.85	6.22	

Table C- 15: Observed velocities during five years flood (502 m^3/s), existing situation test

Section at	Chainage, (m)	Velocity, (m/s)				Remarks
		Right Bank	Centre	Left Bank	Avg	
XS-1	150.00	7.53	9.38	9.89	8.93	
XS-2	108.00	8.04	7.50	5.66	7.07	
XS-3	70.00	7.56	8.87	7.00	7.81	
XS-4	52.00	6.97	7.86	7.45	7.43	
XS-5	7.00	8.22	8.63	6.71	7.85	

XS-6	-29.00	6.65	8.79	8.80	8.08	
XS-7	-66.00	8.20	8.75	8.61	8.52	
XS-8	-108.00	7.26	6.09	6.27	6.54	

Table C- 16: Observed velocities during twenty years flood (681 m^3/s), existing situation test

Section at	Chainage, (m)	Velocity, (m/s)				Remarks
		Right Bank	Centre	Left Bank	Avg	
XS-1	150.00	4.47	11.98	11.23	9.23	
XS-2	108.00	7.28	7.83	8.49	7.87	
XS-3	70.00	11.98	10.57	5.16	9.24	
XS-4	52.00	10.24	9.49	6.09	8.61	
XS-5	7.00	8.75	9.11	7.27	8.38	
XS-6	-29.00	7.80	9.80	9.46	9.02	
XS-7	-66.00	8.64	9.86	9.82	9.44	
XS-8	-108.00	9.49	9.24	5.92	8.22	

Table C- 17: Observed velocities during Fifty years flood (794 m^3/s), existing situation test

Section at	Chainage, (m)	Velocity, (m/s)				Remarks
		Right Bank	Centre	Left Bank	Avg	
XS-1	150.00	5.35	11.89	11.73	9.66	
XS-2	108.00	10.04	8.65	6.68	8.46	
XS-3	70.00	12.26	10.82	5.63	9.57	
XS-4	52.00	10.72	11.78	6.05	9.52	
XS-5	7.00	9.49	9.97	6.44	8.63	
XS-6	-29.00	8.16	10.06	9.56	9.26	
XS-7	-66.00	9.04	10.19	9.78	9.67	
XS-8	-108.00	9.64	9.34	6.61	8.53	

Table C- 18: Observed water levels during average monsoon flow (75 m^3/s), FRHD

Section at	Chainage, (m)	Water Levels, (masl)				Remarks
		Right Bank	Centre	Left Bank	Avg	
XS-1	150.00	1951.68	1952.03	1952.04	1951.92	
XS-2	108.00	1948.14	1947.87	1948.16	1948.06	
XS-3	70.00	1947.71	1947.87	1947.99	1947.86	
XS-4	52.00	1947.54	1947.69	1947.93	1947.72	
XS-UI	30.40	1947.44	1947.50	1947.66	1947.53	
XS-I	15.25	1947.56	1947.53	1947.60	1947.56	
WA	0.00	1947.50	1947.39	1947.59	1947.49	
MSB	-53.80	1939.29	1939.23	1939.32	1939.28	
ES	-77.00	1939.10	1939.13	1939.13	1939.12	
XS-8	-108.00	1935.93	1936.50	1936.80	1936.41	

Table C- 19: Observed water levels during annual flood (164 m^3/s), FRHD

Section at	Chainage, (m)	Water Levels, (masl)				Remarks
		Right Bank	Centre	Left Bank	Avg	
XS-1	150.00	1952.27	1952.21	1952.69	1952.39	
XS-2	108.00	1948.70	1948.34	1948.97	1948.67	
XS-3	70.00	1948.16	1948.37	1948.10	1948.21	
XS-4	52.00	1948.01	1948.32	1948.25	1948.19	
XS-UI	30.40	1947.95	1948.13	1948.37	1948.15	
XS-I	15.25	1948.01	1948.37	1948.35	1948.24	
WA	0.00	1948.32	1948.22	1948.22	1948.25	
MSB	-53.80	1940.25	1940.27	1940.33	1940.28	
ES	-77.00	1939.85	1939.86	1939.82	1939.84	
XS-8	-108.00	1937.09	1936.91	1937.22	1937.07	

Table C- 20: Observed water levels during two years flood (364 m^3/s), FRHD

Section at	Chainage, (m)	Water Levels, (masl)				Remarks
		Right Bank	Centre	Left Bank	Avg	
XS-1	150.00	1952.61	1952.97	1953.89	1953.16	
XS-2	108.00	1951.41	1950.12	1950.45	1950.66	
XS-3	70.00	1948.98	1948.65	1949.16	1948.93	
XS-4	52.00	1949.07	1949.03	1948.98	1949.03	
XS-UI	30.40	1949.10	1949.31	1949.58	1949.33	
XS-I	15.25	1949.01	1949.54	1949.67	1949.41	
WA	0.00	1949.09	1949.37	1949.67	1949.38	
MSB	-53.80	1941.78	1941.75	1941.78	1941.77	
ES	-77.00	1940.73	1941.00	1940.94	1940.89	
XS-8	-108.00	1937.85	1937.28	1937.76	1937.63	

Table C- 21: Observed water levels during five years flood (502 m^3/s), FRHD

Section at	Chainage, (m)	Water Levels, (masl)				Remarks
		Right Bank	Centre	Left Bank	Avg	
XS-1	150.00	1952.79	1953.17	1953.81	1953.26	
XS-2	108.00	1953.00	1950.60	1950.27	1951.29	
XS-3	70.00	1949.28	1949.49	1949.76	1949.51	
XS-4	52.00	1949.55	1950.69	1949.97	1950.07	
XS-UI	30.40	1949.73	1950.03	1950.11	1949.96	
XS-I	15.25	1949.67	1950.18	1950.30	1950.05	
WA	0.00	1949.73	1949.79	1950.18	1949.90	
MSB	-53.80	1942.77	1942.62	1942.73	1942.71	
ES	-77.00	1941.69	1941.65	1941.66	1941.67	
XS-8	-108.00	1939.11	1937.64	1938.30	1938.35	

Table C- 22: Observed water levels during twenty years flood (681 m^3/s), FRHD

Section at	Chainage, (m)	Water Levels, (masl)				Remarks
		Right Bank	Centre	Left Bank	Avg	
XS-1	150.00	1954.46	1954.50	1954.71	1954.56	
XS-2	108.00	1955.97	1951.68	1950.36	1952.67	
XS-3	70.00	1950.06	1950.36	1950.57	1950.33	
XS-4	52.00	1950.45	1951.22	1950.78	1950.82	
XS-UI	30.40	1950.45	1950.60	1950.96	1950.67	
XS-I	15.25	1950.66	1950.45	1951.29	1950.80	
WA	0.00	1950.75	1951.23	1950.96	1950.98	
MSB	-53.80	1943.70	1943.57	1943.73	1943.67	
ES	-77.00	1942.86	1942.89	1943.13	1942.96	
XS-8	-108.00	1940.03	1937.99	1939.11	1939.04	

Table C- 23: Observed water levels during Fifty years flood (794 m^3/s), FRHD

Section at	Chainage, (m)	Water Levels, (masl)				Remarks
		Right Bank	Centre	Left Bank	Avg	
XS-1	150.00	1954.77	1954.86	1954.95	1954.86	
XS-2	108.00	1956.09	1952.55	1950.35	1953.00	
XS-3	70.00	1950.09	1950.51	1950.78	1950.46	
XS-4	52.00	1950.41	1951.08	1951.02	1950.84	
XS-UI	30.40	1950.47	1950.30	1951.08	1950.62	
XS-I	15.25	1950.90	1950.68	1951.29	1950.96	
WA	0.00	1950.95	1951.47	1951.31	1951.24	
MSB	-53.80	1943.88	1943.64	1943.88	1943.80	
ES	-77.00	1943.25	1943.37	1943.61	1943.41	
XS-8	-108.00	1940.55	1938.18	1939.32	1939.35	

Table C- 24: Observed water levels during hundred years flood (880 m^3/s), FRHD with Bedload flushing gate open

Section at	Chainage, (m)	Water Levels, (masl)				Remarks
		Right Bank	Centre	Left Bank	Avg	
XS-1	150.00	1955.28	1954.56	1955.70	1955.18	
XS-2	108.00	1956.87	1953.45	1950.15	1953.49	
XS-3	70.00	1948.92	1949.55	1950.39	1949.62	
XS-4	52.00	1949.37	1950.57	1950.48	1950.14	
XS-UI	30.40	1949.58	1949.49	1950.99	1950.02	
XS-I	15.25	1949.94	1951.86	1951.10	1950.97	
WA	0.00	1951.19	1950.81	1950.60	1950.87	
MSB	-53.80	1943.76	1943.76	1944.15	1943.89	
ES	-77.00	1943.19	1943.19	1943.16	1943.18	
XS-8	-108.00	1940.46	1938.17	1939.37	1939.33	

Table C- 25: Bed levels measured during average monsoon flow, FRHD

River bed at Section #3		River bed at Section #4		River bed at Section #UI		River bed at Section #5	
Coordinate starting point 584124.37E, 3049139.75N		Coordinate starting point 584106.31E, 3049123.63N		Coordinate starting point 584081.72E, 3049105.06N		Coordinate starting point 584062.57E, 3049089.34N	
Distance (m)	Elevation (masl)	Distance (m)	Elevation (masl)	Distance (m)	Elevation (masl)	Distance (m)	Elevation (masl)
61.5	1946.65	55.05	1946.41	47.58	1946.68	47.8	1946.5
62.1	1946.845	55.65	1946.435	48.78	1946.78	48.4	1946.575
63	1946.89	57	1946.65	50.64	1946.62	52	1946.89
63.84	1947.16	60	1947.103	52.71	1946.78	56.2	1947.145
66.6	1947.1	66	1947.46	54.21	1947.22	62.2	1947.205
72	1947.1	72	1947.205	56.91	1947.43	68.2	1947.31
75	1947.115	78	1946.98	60.9	1947.4	73.6	1947.01
80.1	1946.935	84	1946.74	65.4	1947.34	75.31	1946.995
84	1946.785	88.02	1946.47	70.56	1946.773	75.85	1946.86
87	1946.41	91.2	1946.755	75	1946.905	78.4	1946.59
90	1946.32	97.2	1947.01	81.84	1946.26	79.09	1946.365
93	1946.62	98.16	1946.8	85.71	1943.965	79.9	1946.455
93.5	1946.62			90.6	1942.63	81.4	1946
				93.2	1941.7		

Table C- 26: Bed levels measured during annual flood, FRHD

River bed at Section #3		River bed at Section #4		River bed at Section #UI		River bed at Section #5	
Coordinate starting point 584124.37E, 3049139.75N		Coordinate starting point 584106.31E, 3049123.63N		Coordinate starting point 584081.72E, 3049105.06N		Coordinate starting point 584062.57E, 3049089.34N	
Distance (m)	Elevation (masl)	Distance (m)	Elevation (masl)	Distance (m)	Elevation (masl)	Distance (m)	Elevation (masl)
61.71	1946.245	55.05	1946.17	47.58	1945.78	47.8	1946.5
63.45	1946.59	56.85	1946.83	48.54	1945.84	48.52	1946.56
64.8	1946.71	58.74	1947.49	50.1	1946.305	49.99	1946.83
66.3	1947.01	60.09	1947.46	51.54	1945.84	54.1	1946.95
70.5	1945.135	62.61	1946.47	55.5	1945.93	60.1	1946.83
73.5	1945	65.19	1945.72	59.4	1946.095	64.9	1946.815
75.9	1945.27	68.49	1945.54	62.4	1946.23	68.8	1946.92
76.8	1945.45	71.49	1945.6	66.9	1946.32	73	1947.145
79.5	1945.54	73.53	1946.125	70.23	1946.8	75.61	1947.22
81.15	1945.21	78	1946.32	74.7	1946.905	77.5	1946.62
83.7	1945.645	84	1946.665	78.6	1946.8	79.03	1945.96
86.91	1945.855	87.15	1946.89	81.33	1946.77	79.9	1945.93
88.38	1946.53	88.65	1947.19	84.96	1944.64	81.4	1946
89.79	1946.35	93	1947.28	88.68	1942.96		
92.13	1947.13	98.4	1947.1	89.4	1942.9		
93.66	1947.31			90.66	1942.72		
94.23	1947.16			93.2	1941.7		

Table C- 27: Bed levels measured during 2 years flood, FRHD

River bed at Section #3		River bed at Section #4		River bed at Section #UI		River bed at Section #5	
Coordinate starting point 584124.37E, 3049139.75N		Coordinate starting point 584106.31E, 3049123.63N		Coordinate starting point 584081.72E, 3049105.06N		Coordinate starting point 584062.57E, 3049089.34N	
Distance (m)	Elevation (masl)	Distance (m)	Elevation (masl)	Distance (m)	Elevation (masl)	Distance (m)	Elevation (masl)
64.35	1944.88	55.8	1945.39	47.58	1944.84	47.8	1945.1
65.79	1945.21	58.8	1945.72	48.9	1944.94	48.91	1945.12
69.9	1944.895	61.2	1946.11	51.78	1945.54	50.65	1945.39
74.7	1944.97	65.4	1945.105	54.06	1945.675	51.73	1945.99
76.02	1945.24	67.98	1944.49	55.59	1946.65	56.2	1946.365
79.74	1945.09	71.1	1944.46	61.53	1946.41	60.37	1946.305
83.4	1945.51	76.2	1945.45	69.78	1946.755	62.71	1946.995
87.15	1945.66	82.2	1946.35	75.93	1947.61	68.71	1947.25
90	1946.71	87.6	1947.565	81.06	1947.22	74.8	1947.28
94.05	1947.04	94.41	1948.15	85.62	1944.295	79.69	1946.65
		95.58	1947.655	90.72	1942.82	81.4	1946
		96.33	1947.52	93.2	1941.8		
		97.05	1947.085				
		98.19	1946.95				

Table C- 28: Bed levels measured during 5 years flood, FRHD

River bed at Section #3		River bed at Section #4		River bed at Section #UI		River bed at Section #5	
Coordinate starting point 584124.37E, 3049139.75N		Coordinate starting point 584106.31E, 3049123.63N		Coordinate starting point 584081.72E, 3049105.06N		Coordinate starting point 584062.57E, 3049089.34N	
Distance (m)	Elevation (masl)	Distance (m)	Elevation (masl)	Distance (m)	Elevation (masl)	Distance (m)	Elevation (masl)
63.3	1945.2	55.56	1945.405	47.58	1944.84	47.8	1945
64.65	1945.465	57	1945.72	48.9	1944.97	48.7	1945.045
67.05	1945.12	59.31	1945.945	52.8	1944.925	51.61	1945.345
71.85	1945.54	64.2	1944.445	55.59	1944.49	57.1	1945.6
73.71	1945.27	66.6	1944.02	64.2	1944.4	63.28	1946.185
76.02	1945.24	69.15	1944.05	67.95	1945.54	68.56	1946.755
81.9	1945.135	77.73	1944.04	72.9	1946.875	74.41	1947.1
83.1	1944.82	82.14	1944.13	77.4	1947.67	76.78	1946.89
86.28	1944.895	86.4	1945.9	79.26	1948.03	78.79	1946.68
89.1	1944.7	90.6	1948.03	81.9	1947.205	79.78	1946.785
90.6	1944.775	93.6	1947.76	87.3	1943.77	81.4	1946
91.5	1944.835	96	1947.1	90.96	1942.12		
		97.5	1946.35	93.2	1941.8		
		98.16	1946.35				

APPENDIX D

Photographs

Physical Hydraulic Model Study of the Headworks of Ghunsa Khola HEP (78 MW)

Final Report

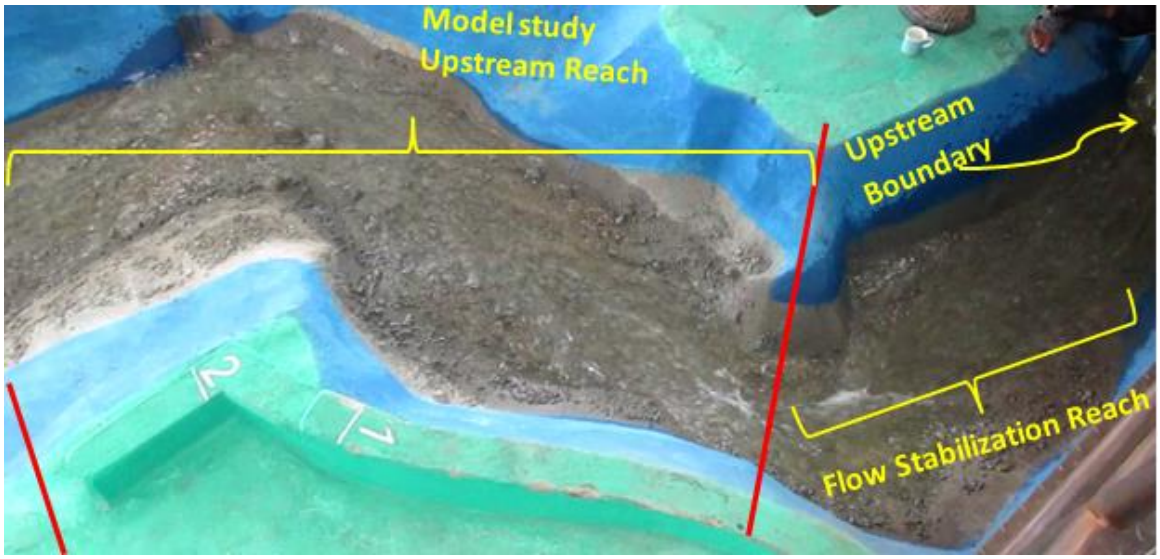
December 2020

REMIT HYDRO
रेमिट हाइड्रो लिमिटेड

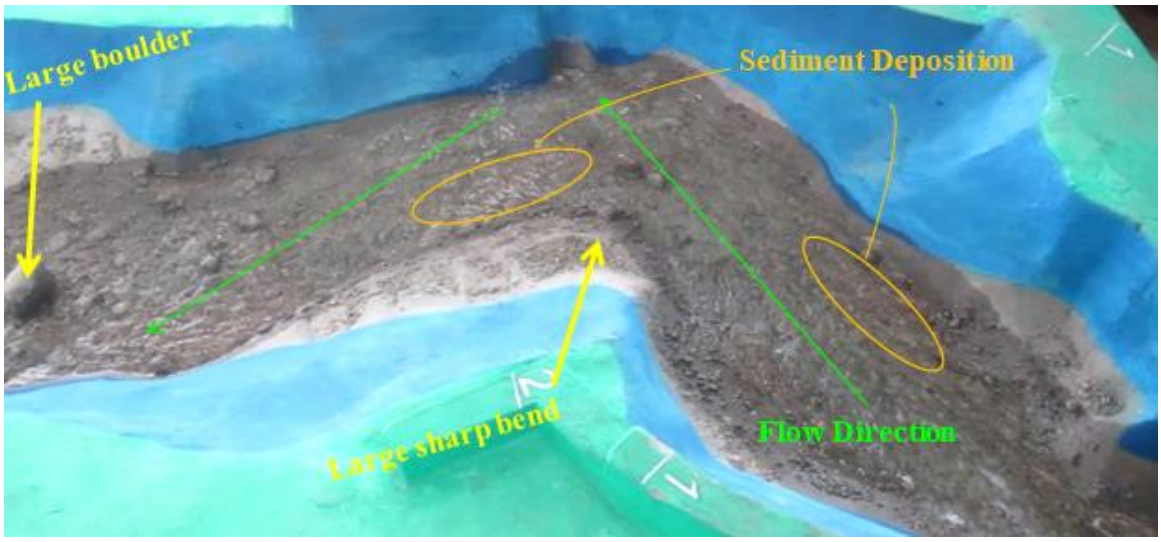
Photograph D-1: Placement of the river cross-sections

Photograph D- 2: View of the river model after boulder placement and roughness application

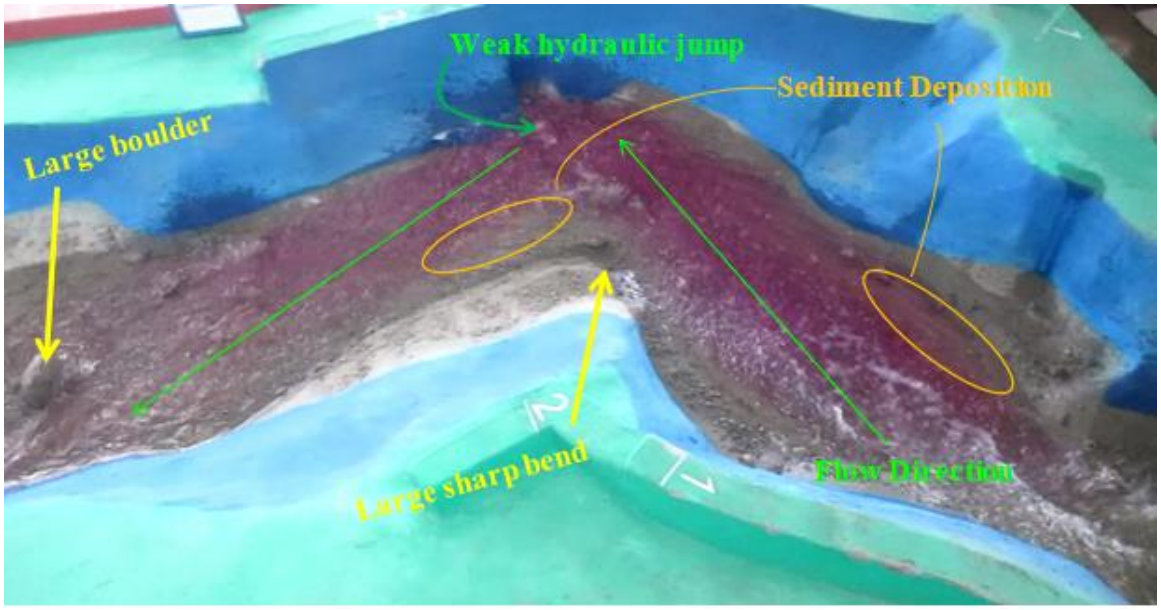
Photograph D- 3: Water level measurements during model validation



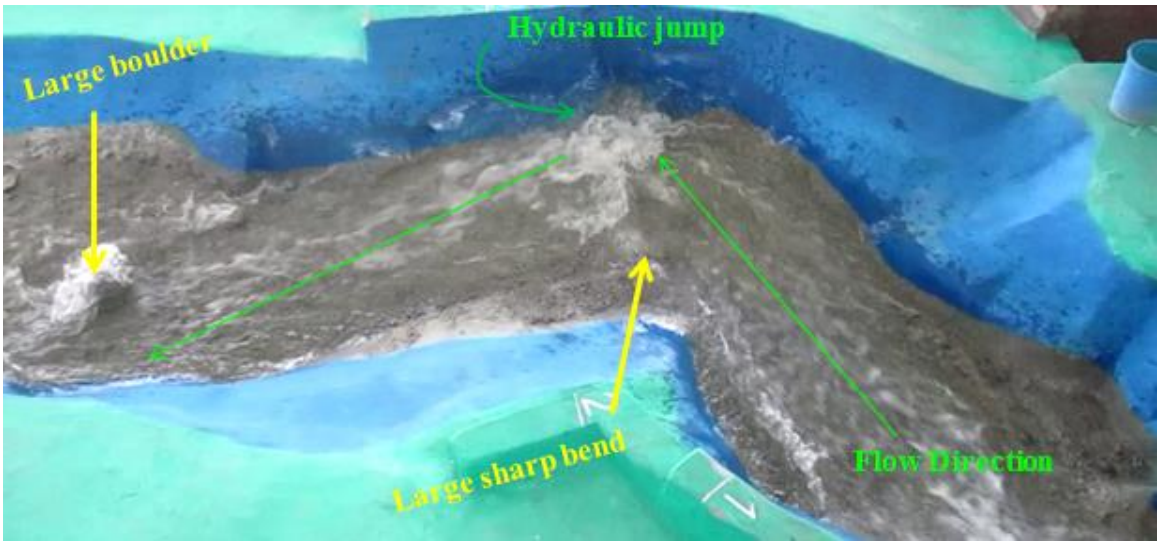
Photograph D- 4: Model study upstream stretch demarcation for existing situation test



Photograph D- 5: Flow scenario in upstream stretch area in river model during average monsoon flow

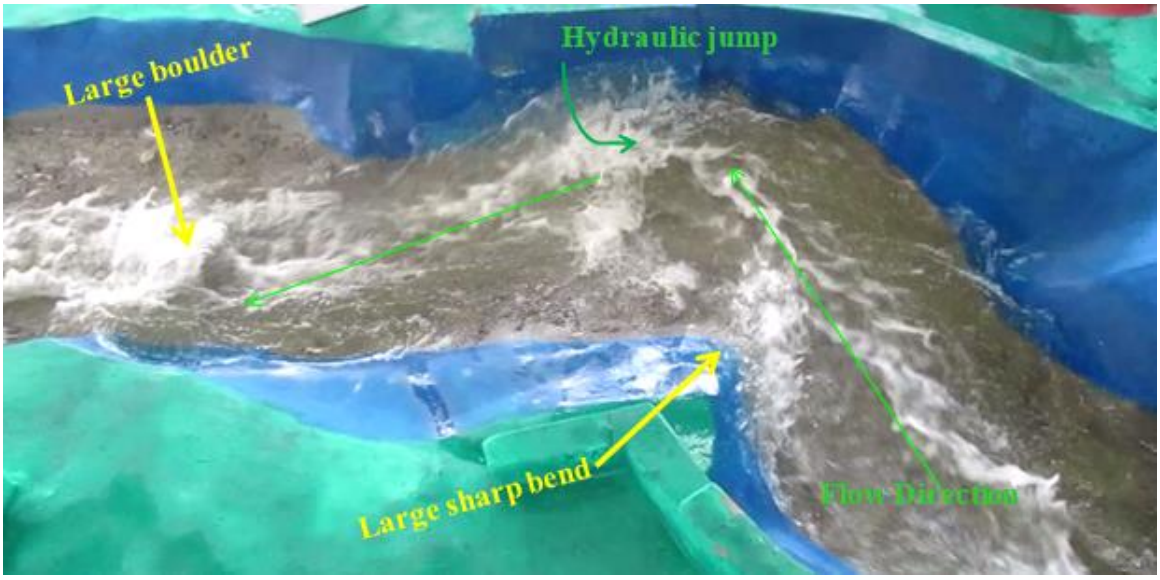


Photograph D- 6: Flow scenario in upstream stretch area in river model during annual flood

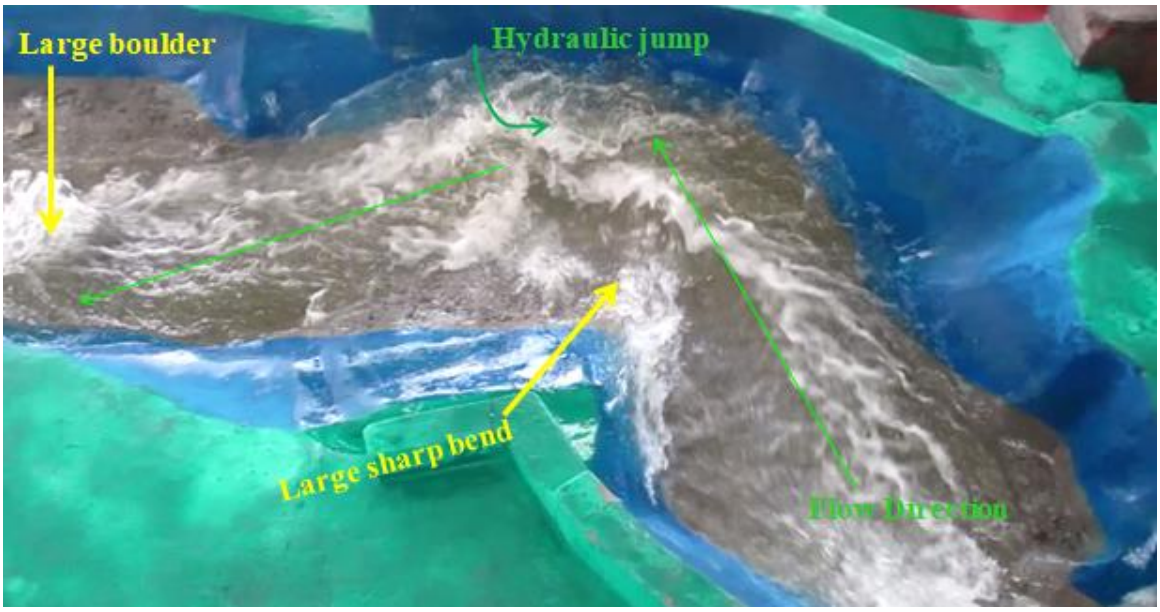


Photograph D- 7: Flow scenario in upstream stretch area in river model during 2 Years flood

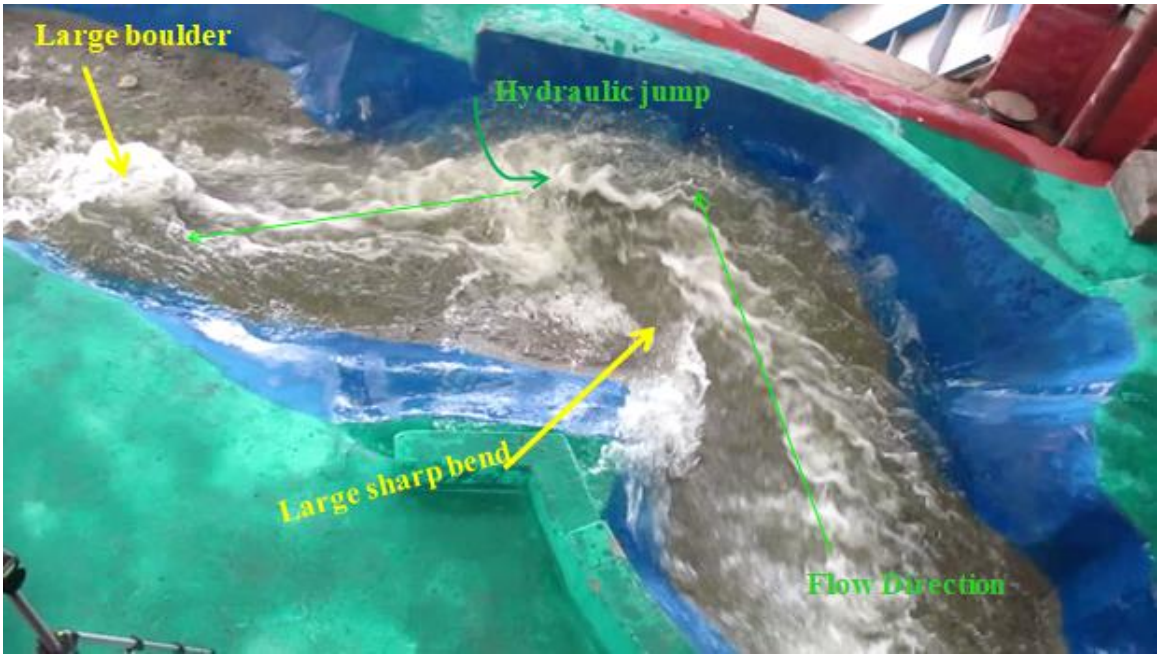
Photograph D- 8: Flow scenario in upstream stretch area in river model during 5 Years flood



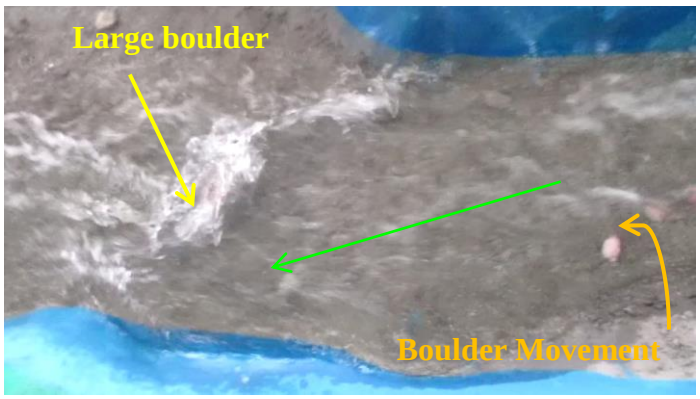
Photograph D- 9: Flow scenario in upstream stretch area in river model during 20 Years flood

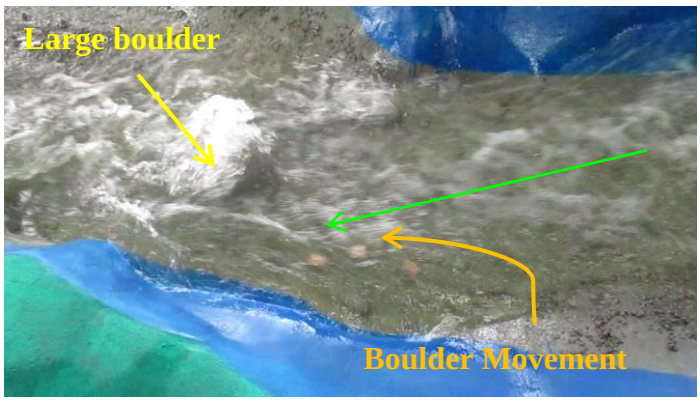


Photograph D- 10: Flow scenario in upstream stretch area in river model during 50 Years flood

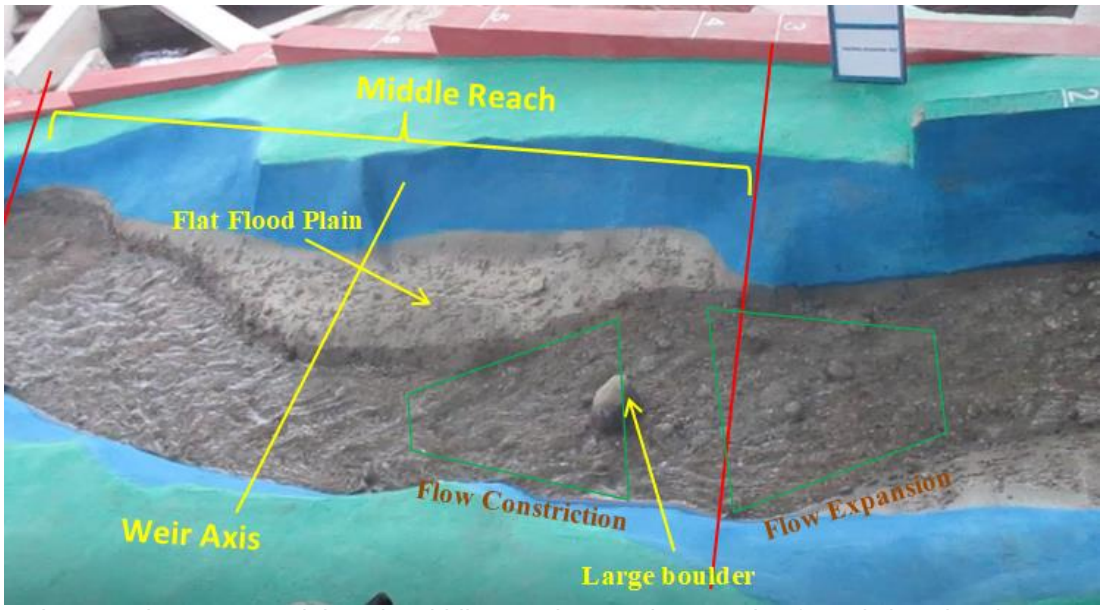


Photograph D- 11: Flow scenario in upstream stretch area in river model during 100 Years flood

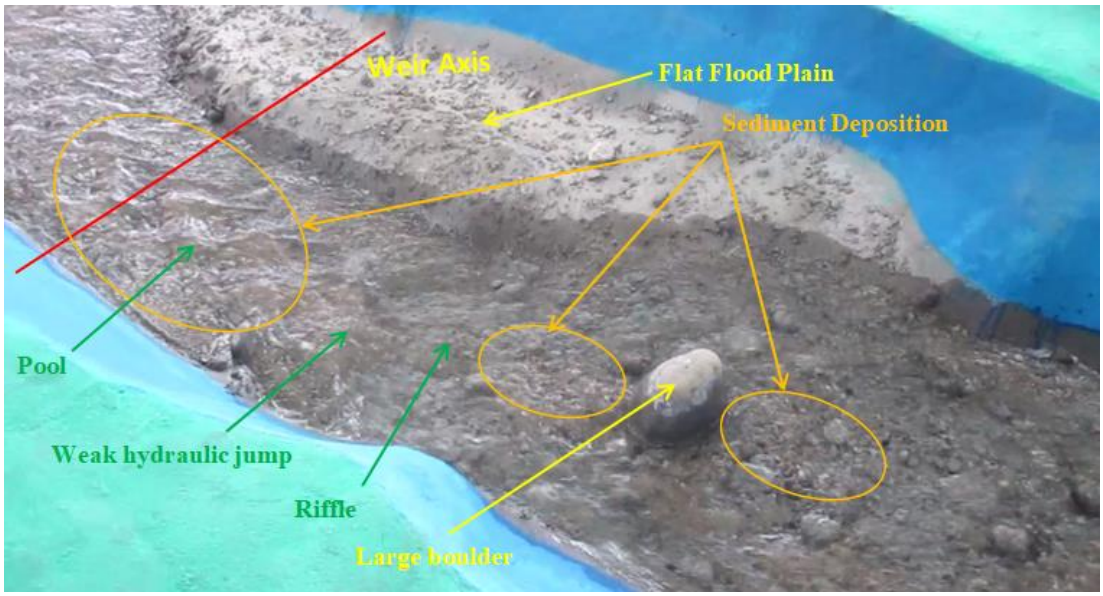




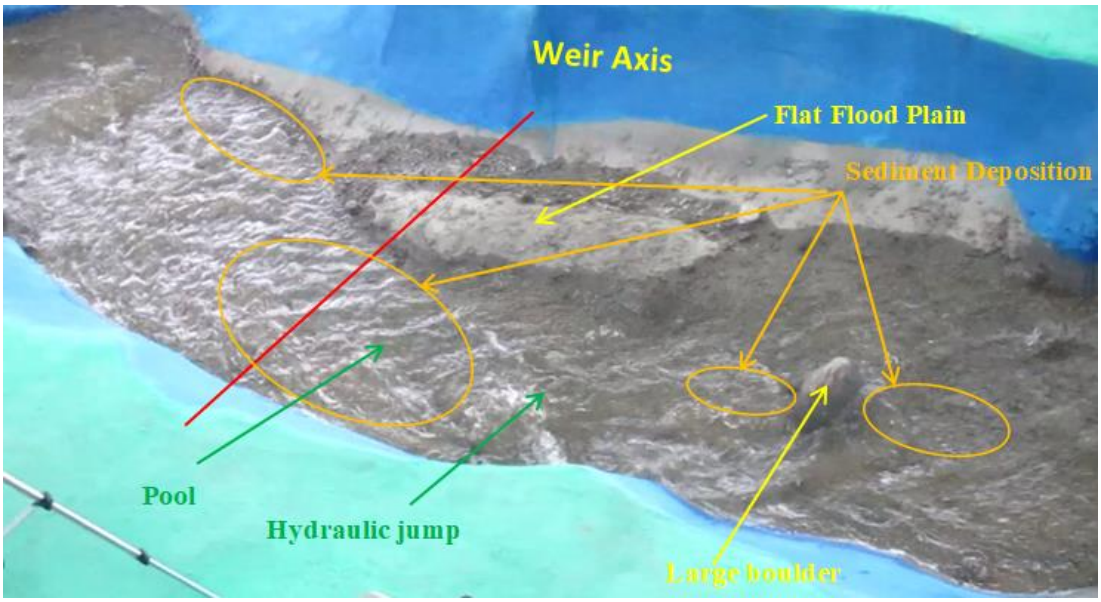
Photograph D- 12: Boulder movement in upstream stretch area in river model during 5years flood



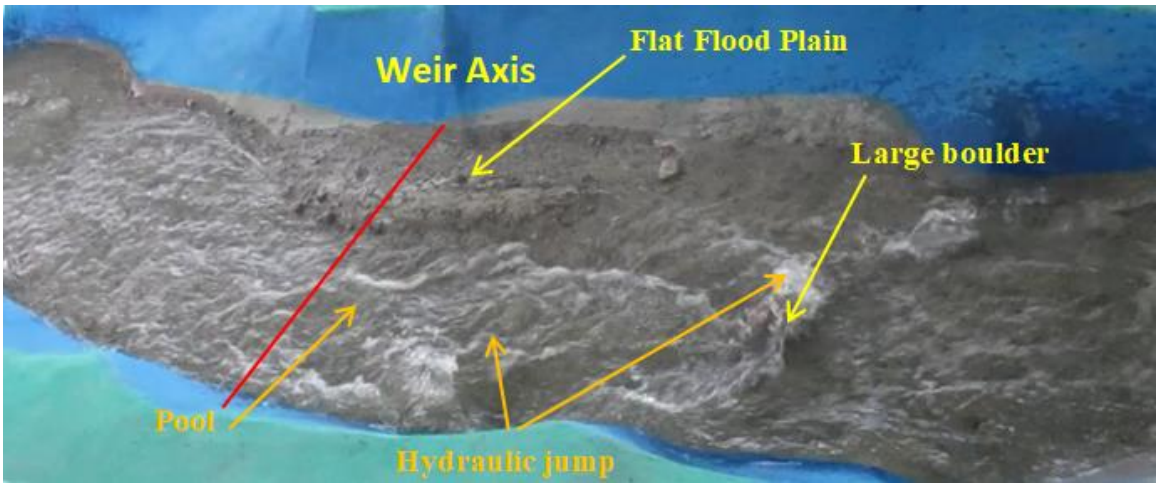
Photograph D- 13: Model study Middle Reach Area demarcation for existing situation test



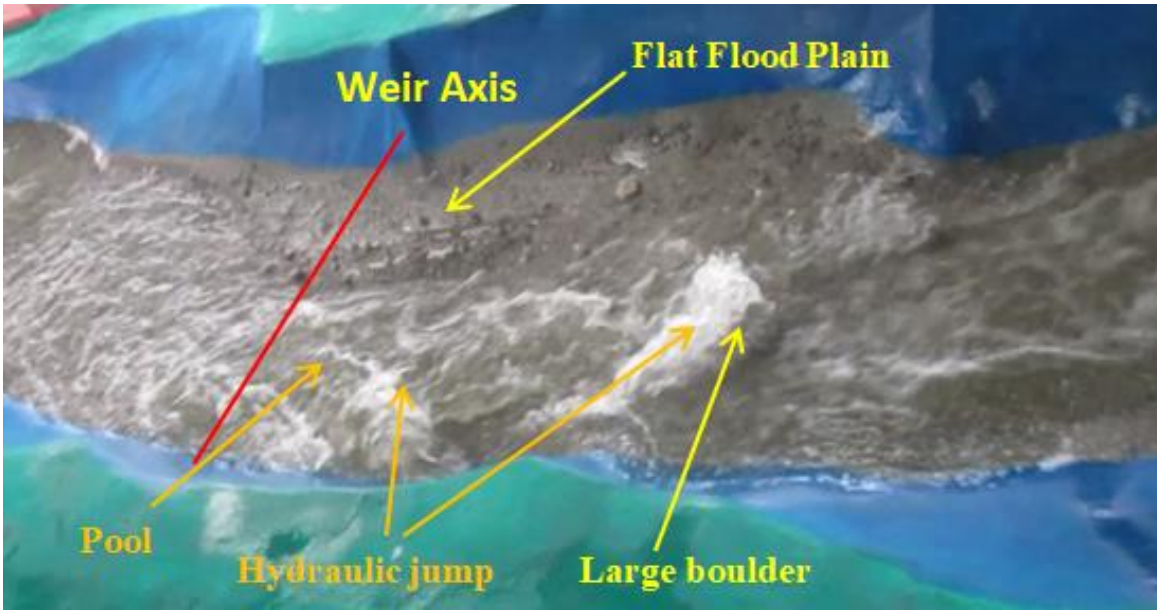
Photograph D- 14: Flow scenario in Middle Reach area in river model during average monsoon flow



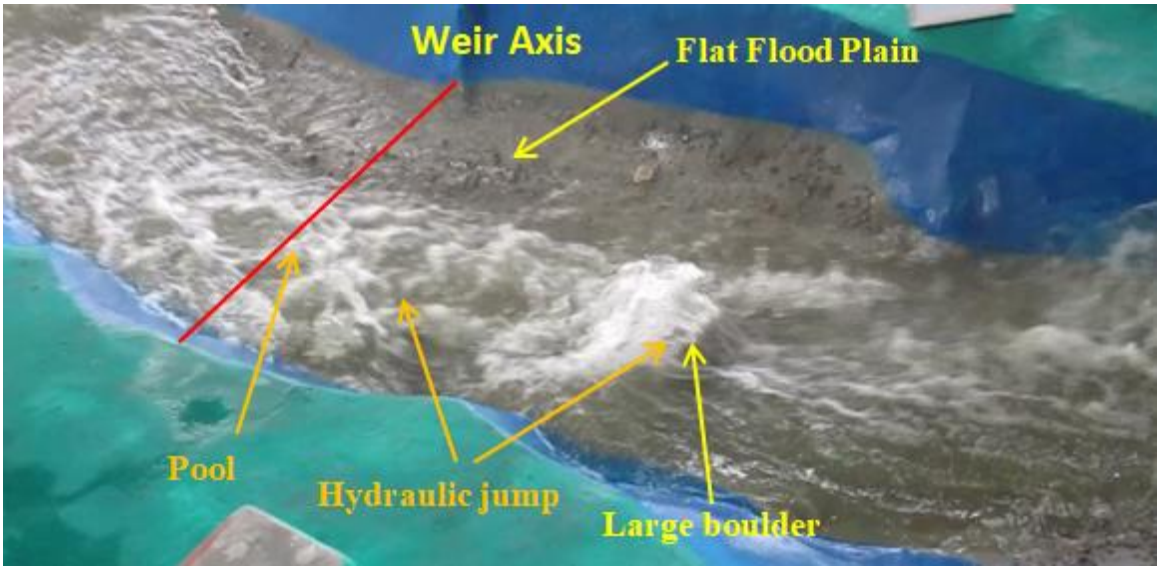
Photograph D- 15: Flow scenario in Middle Reach area in river model during annual Flood



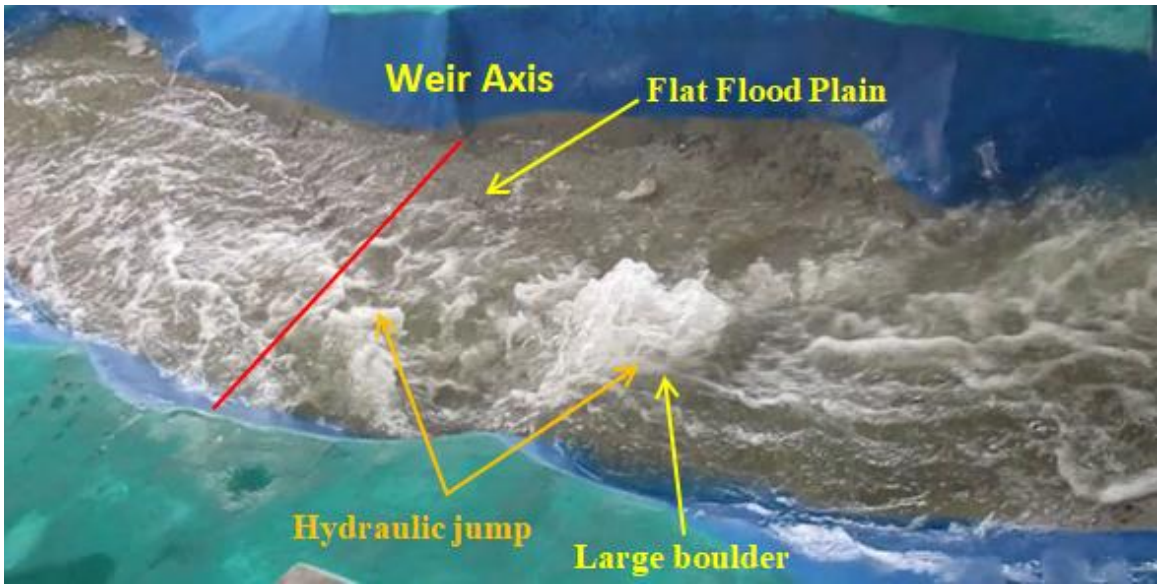
Photograph D- 16: Flow scenario in Middle Reach area in river model during 2 years Flood



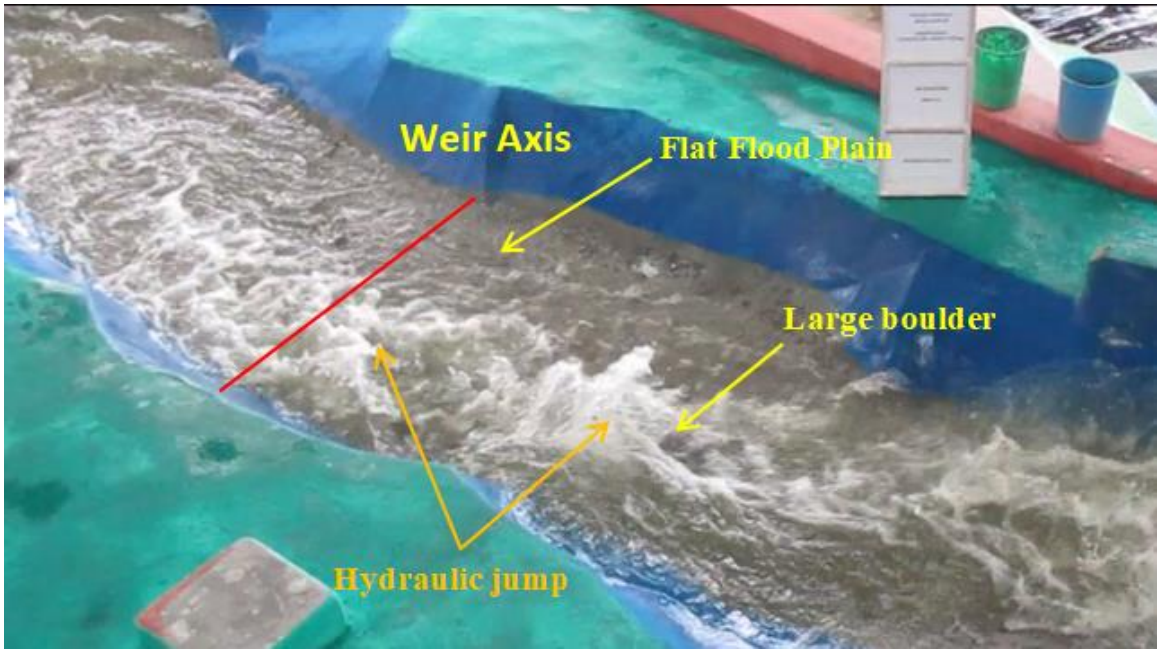
Photograph D- 17: Flow scenario in Middle Reach area in river model during 5 years Flood



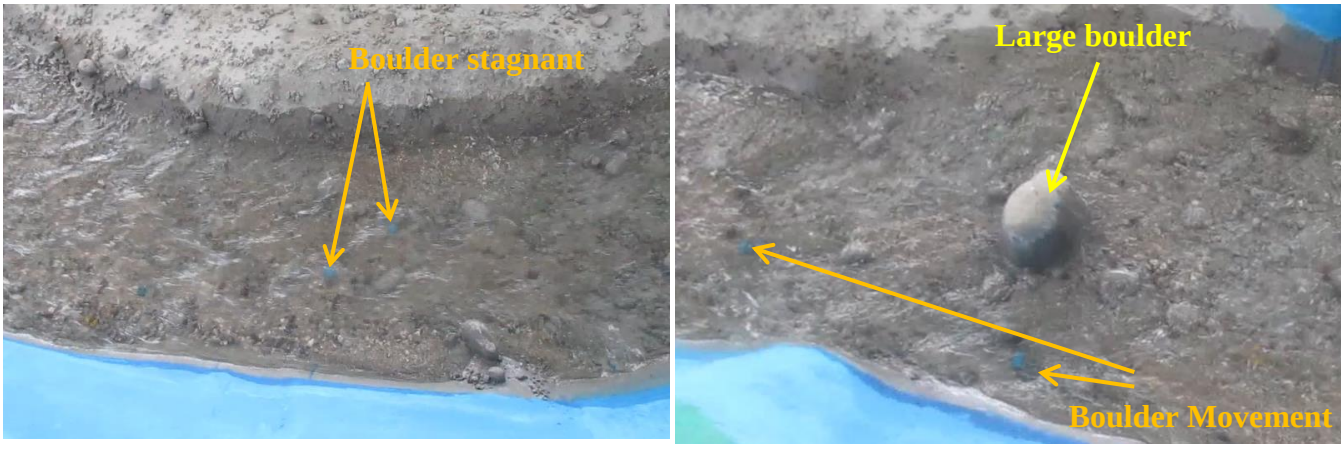
Photograph D- 18: Flow scenario in Middle Reach area in river model during 20 years Flood



Photograph D- 19: Flow scenario in Middle Reach area in river model during 50 years Flood

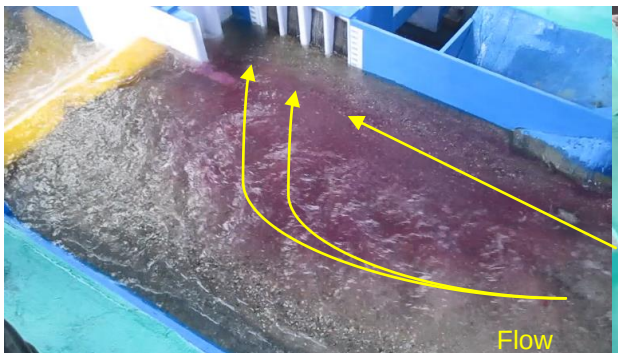


Photograph D- 20: Flow scenario in Middle Reach area in river model during 100 years Flood



Photograph D- 21: Boulder movement in Middle Reach area in river model during average monsoon

Photograph D- 22: Upstream and Downstream view of Base Case Design during avg. monsoon Flow



Photograph D- 23: Approach flow of Base Case Design during annual flood condition

Photograph D- 24: Approach flow of Base Case Design during 5 years flood

Photograph D- 25: Flow condition of Base Case Design during 20years flood

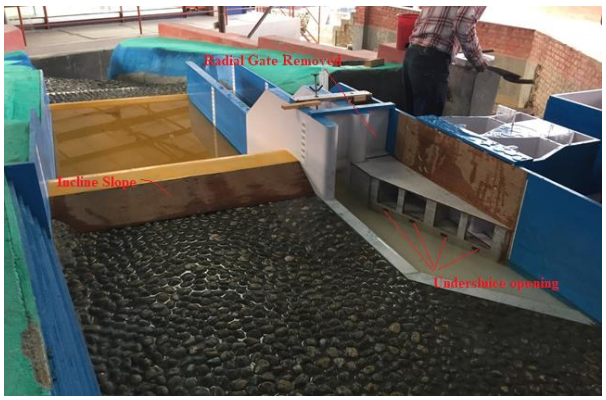
Photograph D- 26: Flow condition of Base Case Design during design flood (100yrs Flood)

Photograph D- 27: Flow scenario of Base Case Design during 2 years return period flood

Photograph D- 28: Flow scenario of Base Case Design during 5 years return period flood

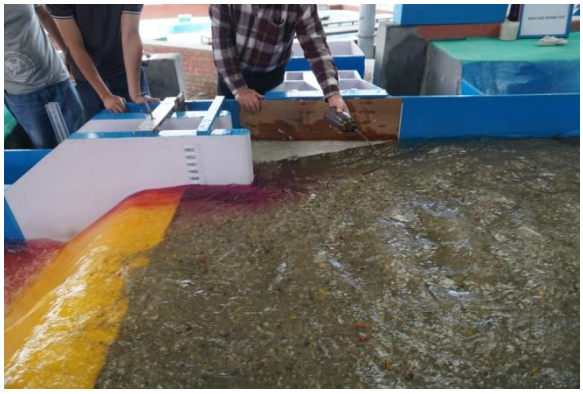
Photograph D- 29: Flow scenario of Base Case Design during 20 years return period flood

Photograph D- 30: Flow scenario of Base Case Design during 100 years return period flood



Photograph D- 31: Conceptual test Modified Arrangement to the Base Case Design

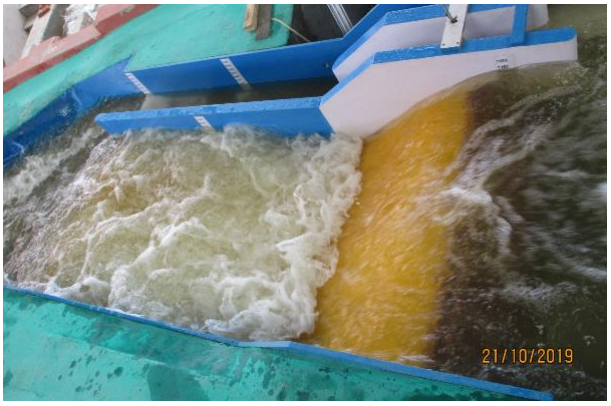
Photograph D- 32: Bed control in front of intake for Conceptual test (average monsoon flow)



Photograph D- 33: Favourable sediment movement over the weir for Conceptual test

Photograph D- 34: Rotational flow in front of intake for Conceptual test (average monsoon flow)

Photograph D- 35: Reduced exit flow path configuration of stilling basin scenario at design flood

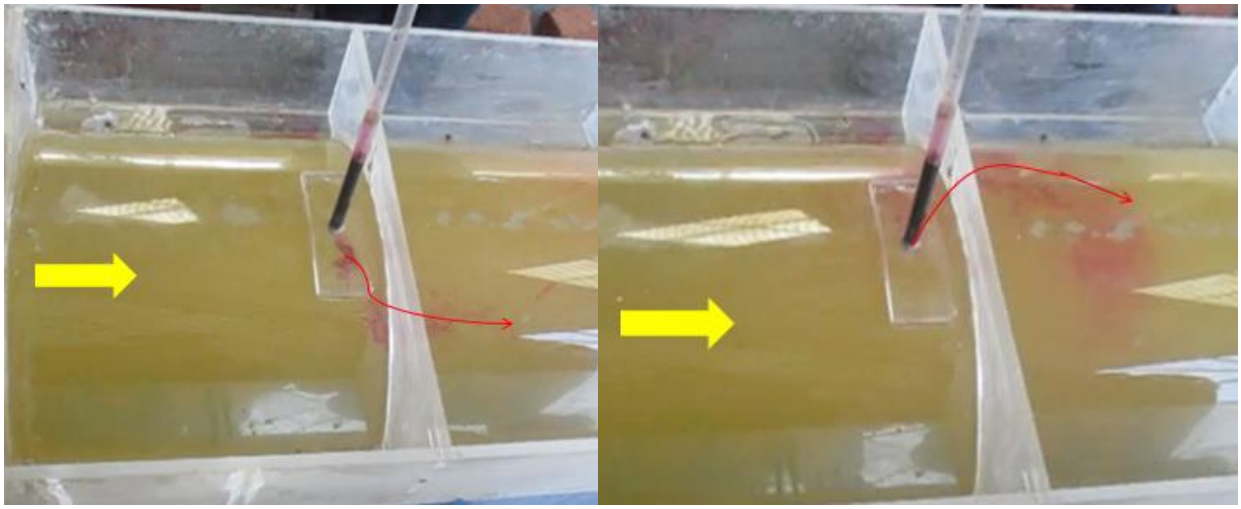


Photograph D- 36: Stilling basin's floor level lowered by 1m for design flood scenario

Photograph D- 37: No back water observed in the undersluice and gravel trap flushing channel up to 10 years flood with radial gates closed for MDA-I

Photograph D- 38: Back water observed in the undersluice and gravel trap flushing channel at 20 years flood with one radial gates open for MDA-I

Photograph D- 39: Trash passage operation during avg. monsoon flow for MDA-I



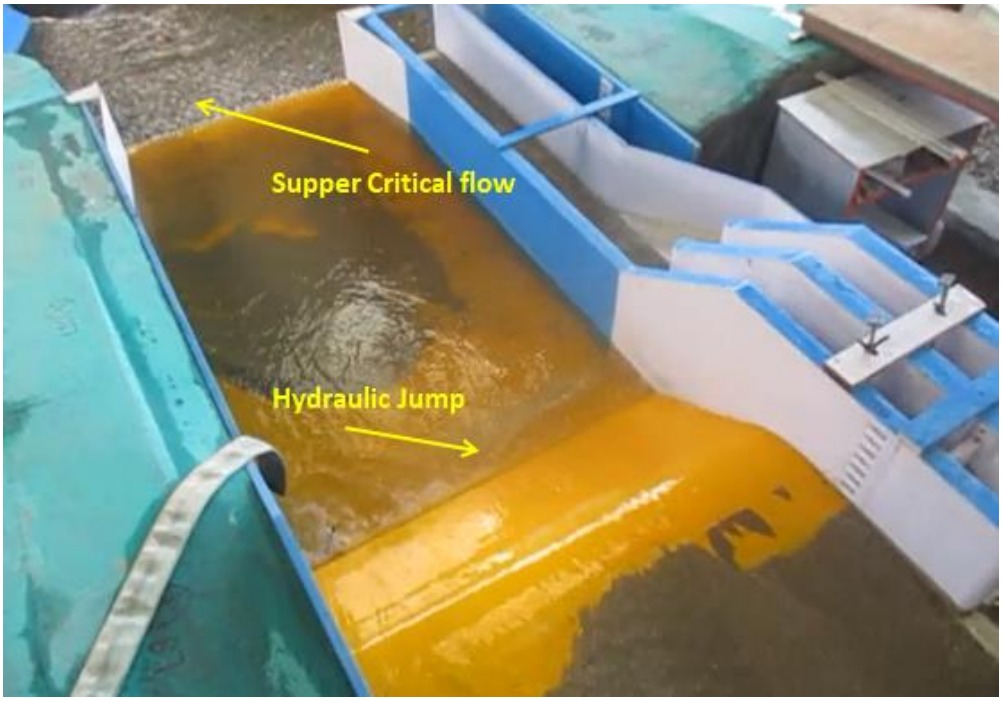
Photograph D- 40: Haunting osculation of flow for Modified Arrangement III Settling Basin

Photograph D- 41: Flow scenario at average monsoon flow, (FRHD)

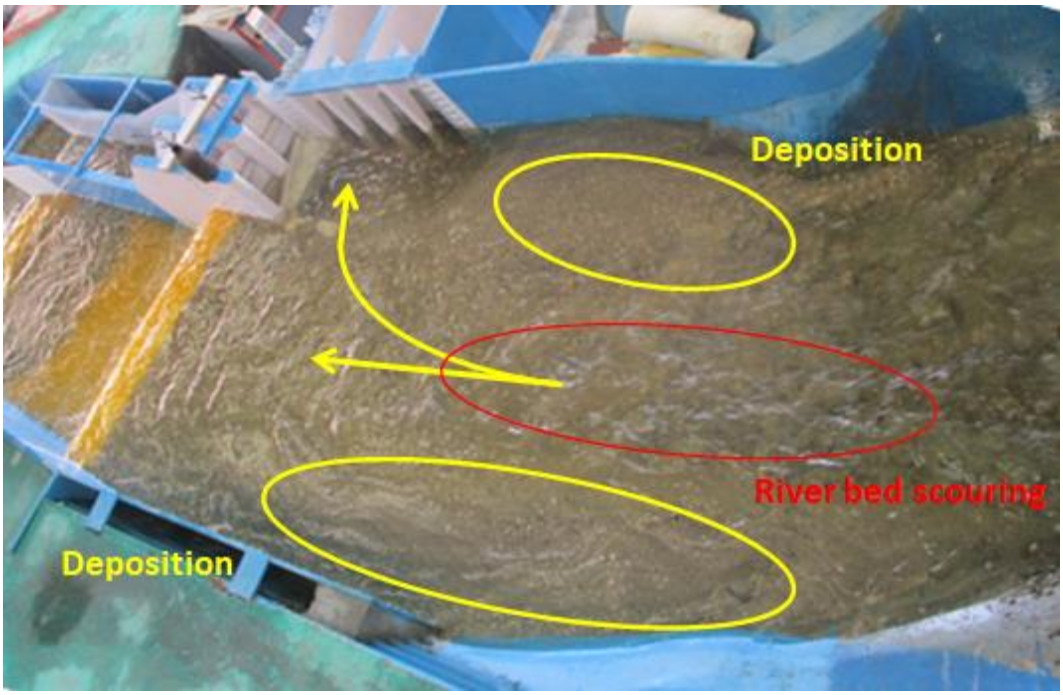
Photograph D- 43: Trash movement over trash passage structure at average monsoon flow, (FRHD)

Photograph D- 44: Secondary flow at average monsoon flow, (FRHD)

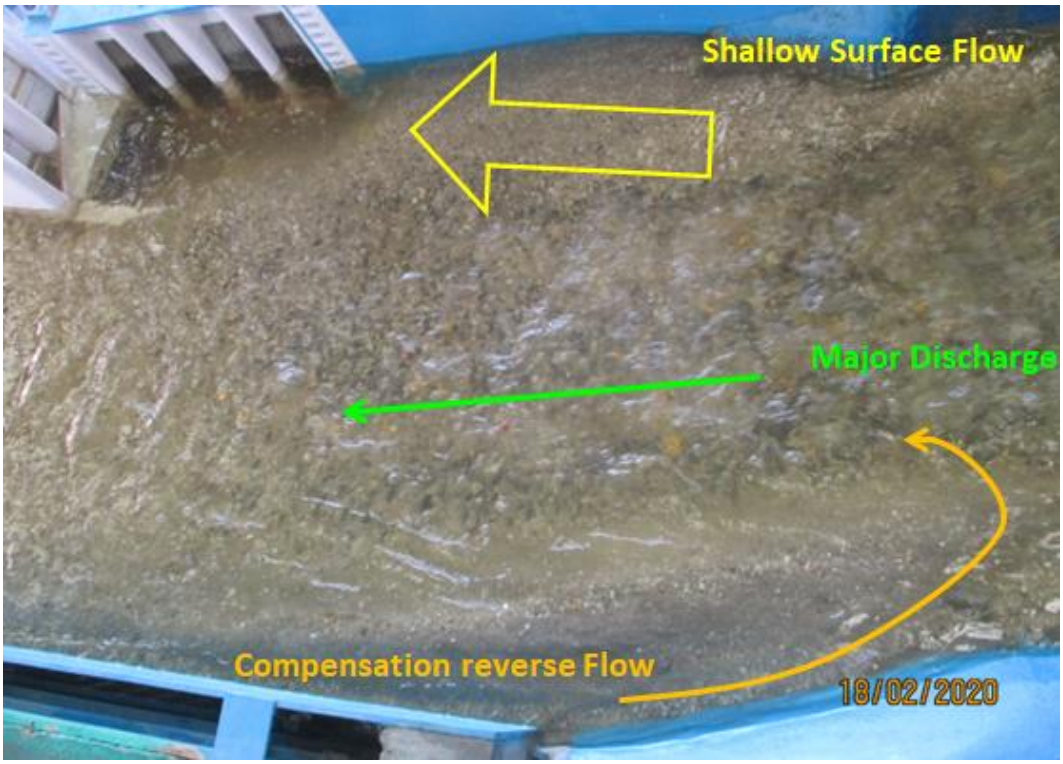
Photograph D- 42: River bed deposition pattern at average monsoon flow, (FRHD)



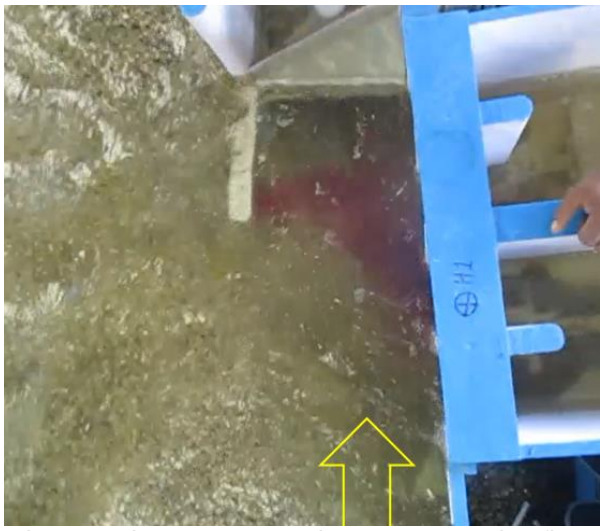
Photograph D- 45: Flow in Stilling basin at average monsoon flow, (FRHD)



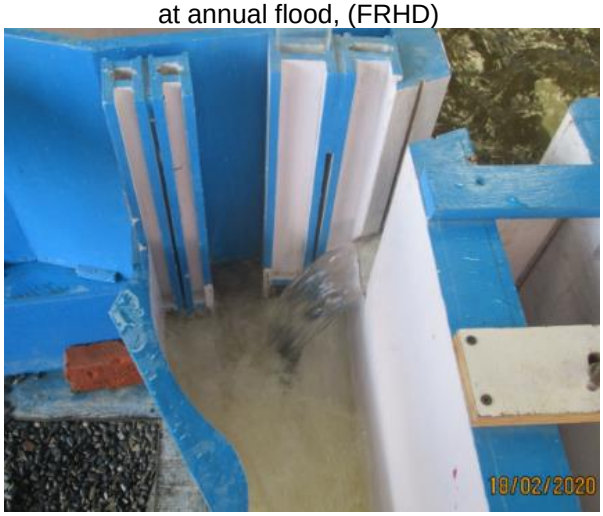
Photograph D- 46: Flow scenario with bed deposition at annual flood, (FRHD)



Photograph D- 47: Flow pattern at annual flood, (FRHD)

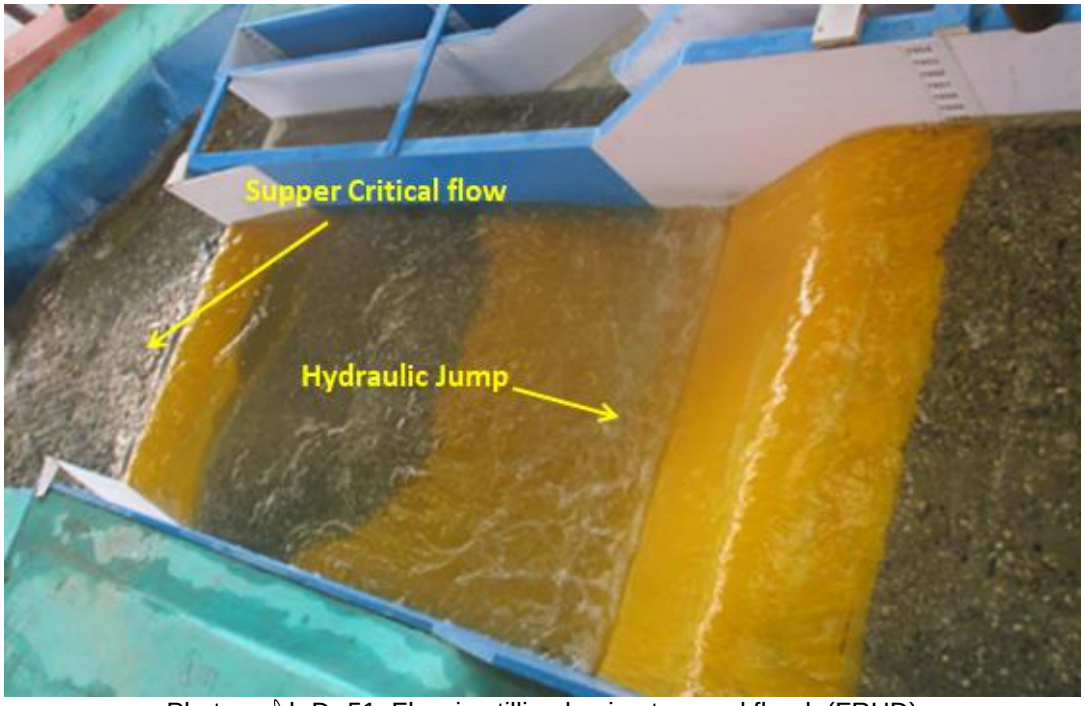


Photograph D- 49: Secondary compensation flow at annual flood, (FRHD)



Photograph D- 50: Gravel trap outlet submerged with overloading flow from trash passage during annual flood, (FRHD)

Photograph D- 48: River bed deposition pattern at annual flood, (FRHD)



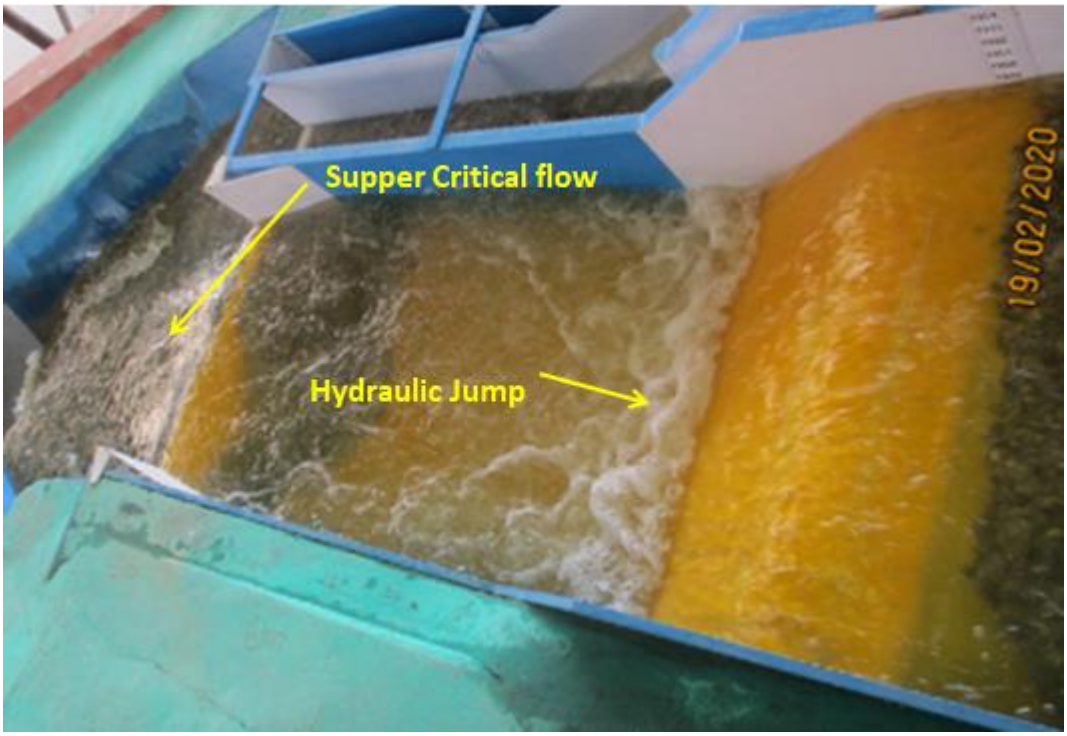
Photograph D- 51: Flow in stilling basin at annual flood, (FRHD)

Photograph D- 52: Flow scenario with bed deposition at 2 years return period flood, (FRHD)

Photograph D- 54: Gravel trap outlet submerged with overloading flow from trash passage during 2yrs flood, (FRHD)

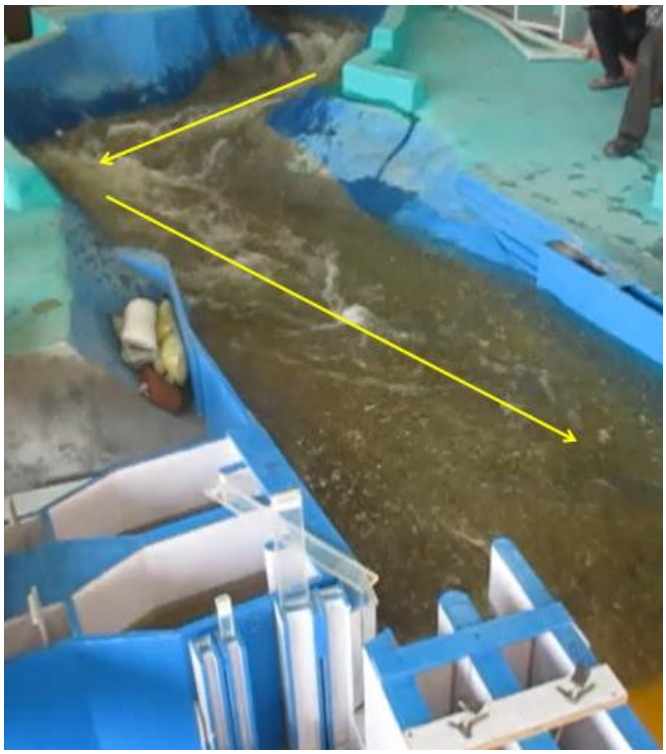
Photograph D- 55: Free flow from Gravel trap outlet with trash passage fully closed during 2 years flood, (FRHD)

Photograph D- 53: River bed deposition pattern at 2 years return period flood, (FRHD)



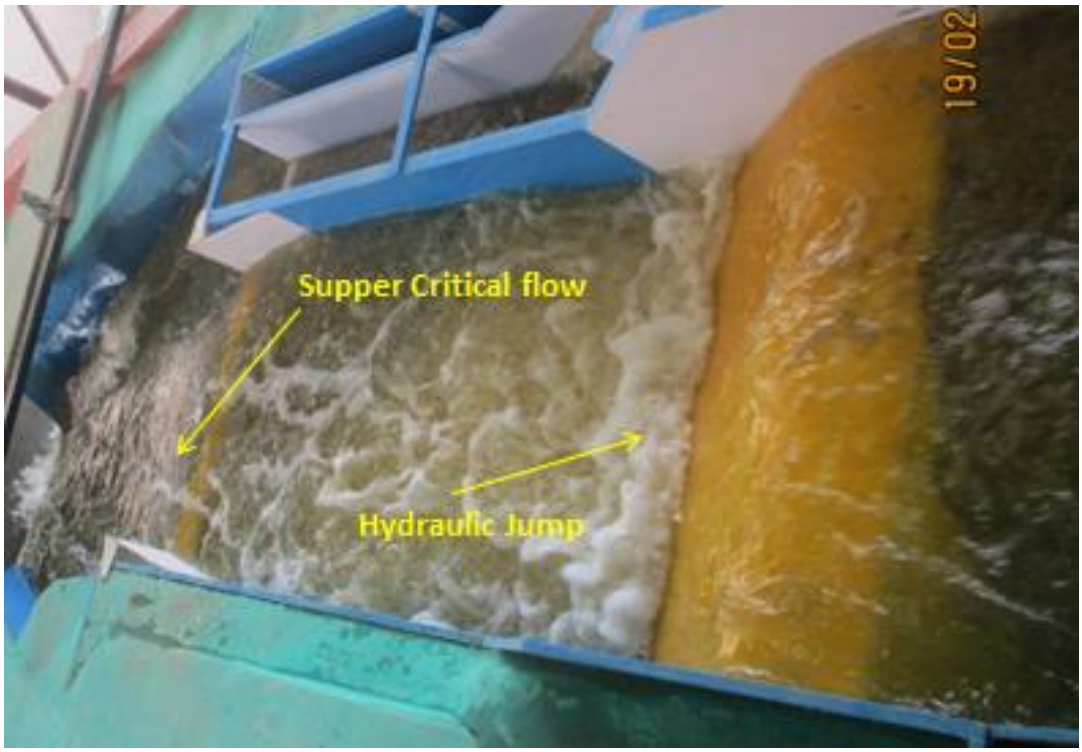
Photograph D- 56: Flow in stilling basin at 2 years return period flood, (FRHD)

Photograph D- 57: Flow scenario with bed deposition at 5 years return period flood, (FRHD)

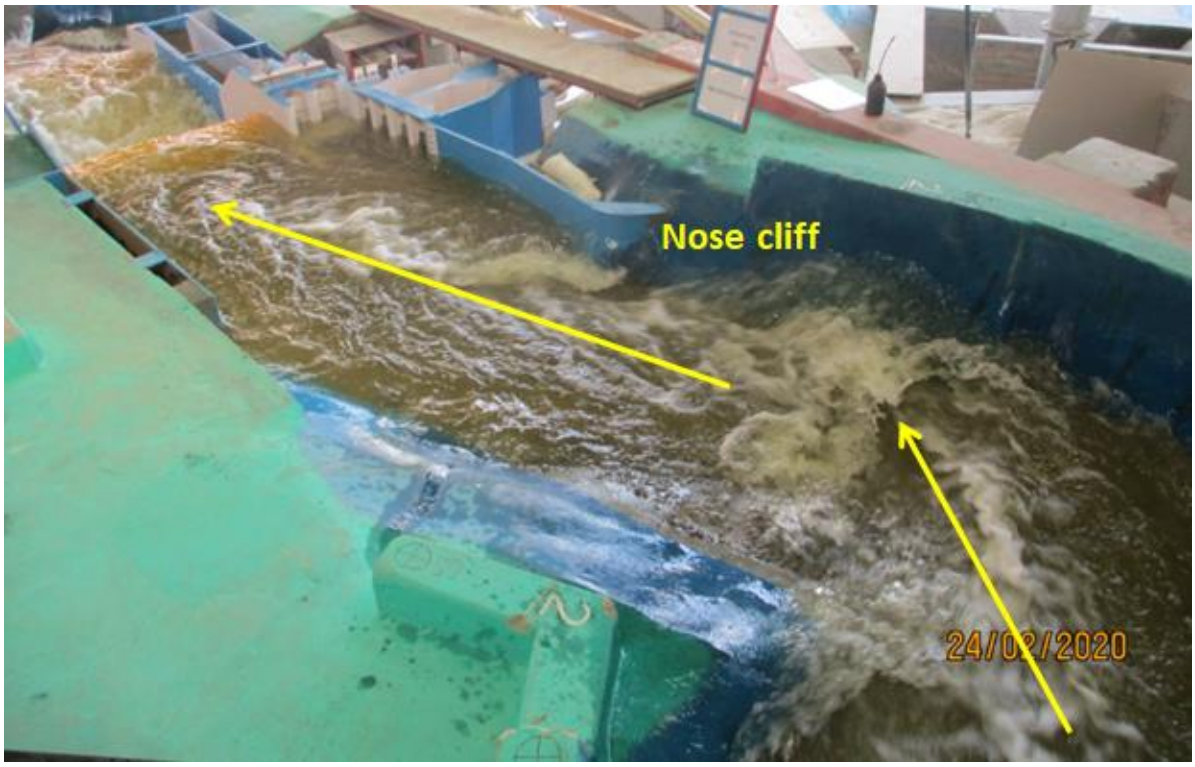


Photograph D- 58: Upstream view of 5 years return period flood, (FRHD)

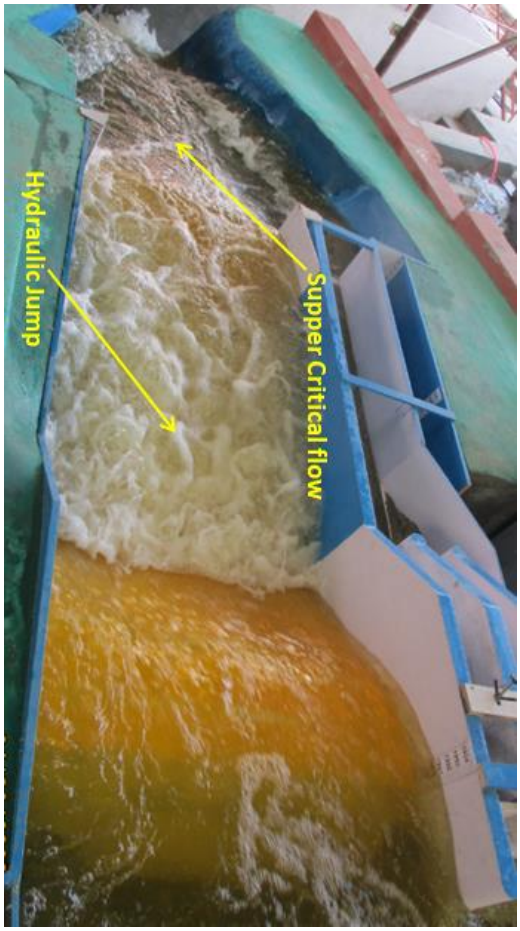
Photograph D- 59: River bed deposition pattern at 5years return period flood, (FRHD)



Photograph D- 60: Flow in stilling basin at 5years return period flood, (FRHD)



Photograph D- 61: Flow scenario at 20 years return period flood, (FRHD)



Photograph D- 62: Flow in stilling basin at 20years return period flood, (FRHD)

Photograph D- 63: Upstream view of 20years return period flood, (FRHD)

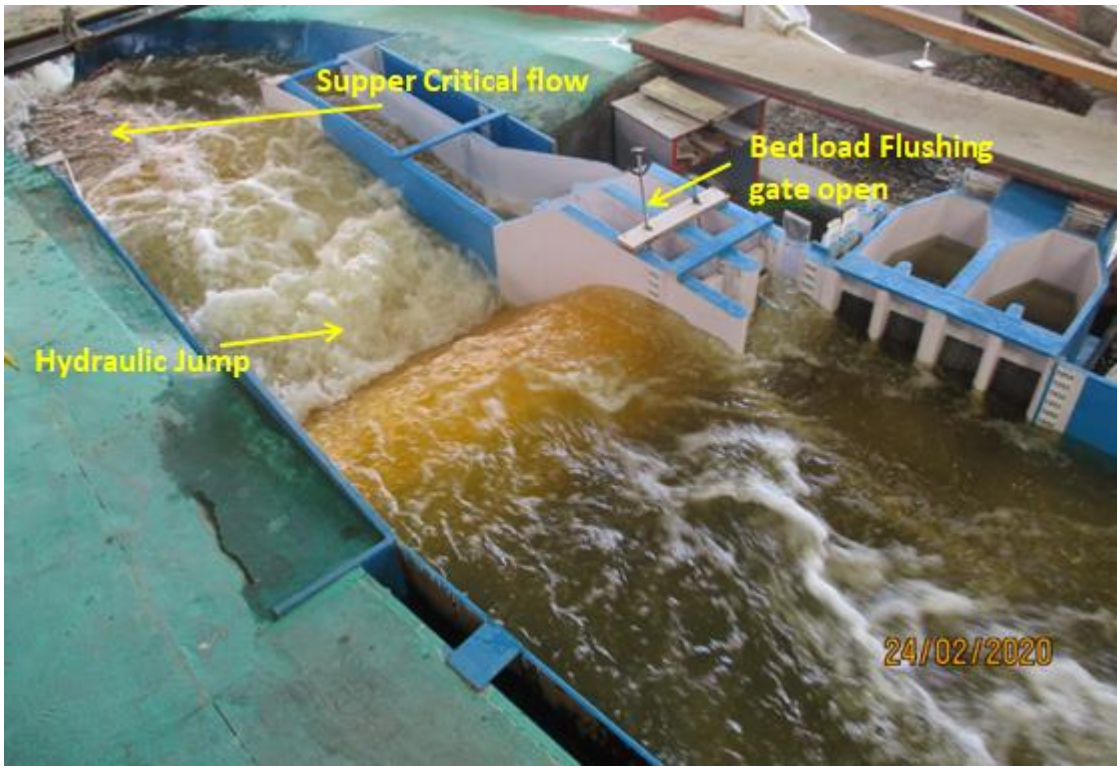
Photograph D- 64: Flow scenario at 50 years return period flood, (FRHD)

Photograph D- 65: Flow in stilling basin at 50years return period flood, (FRHD)

Photograph D- 66: Upstream view of 50years return period flood, (FRHD)

Photograph D- 67: Flow scenario at 100 years return period flood, (FRHD)

Photograph D- 68: Upstream view of 100years return period flood, (FRHD)

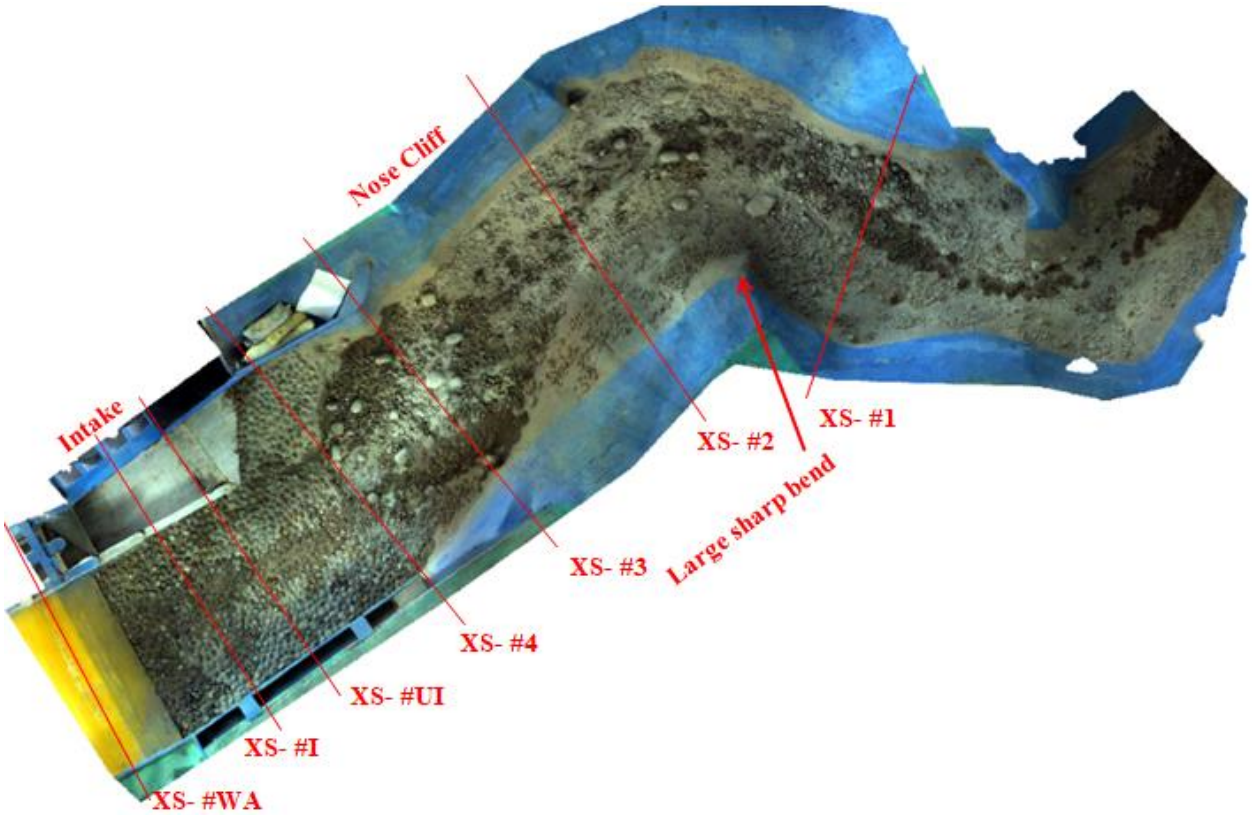


Photograph D- 69: Flow in stilling basin at 100years return period flood, (FRHD)

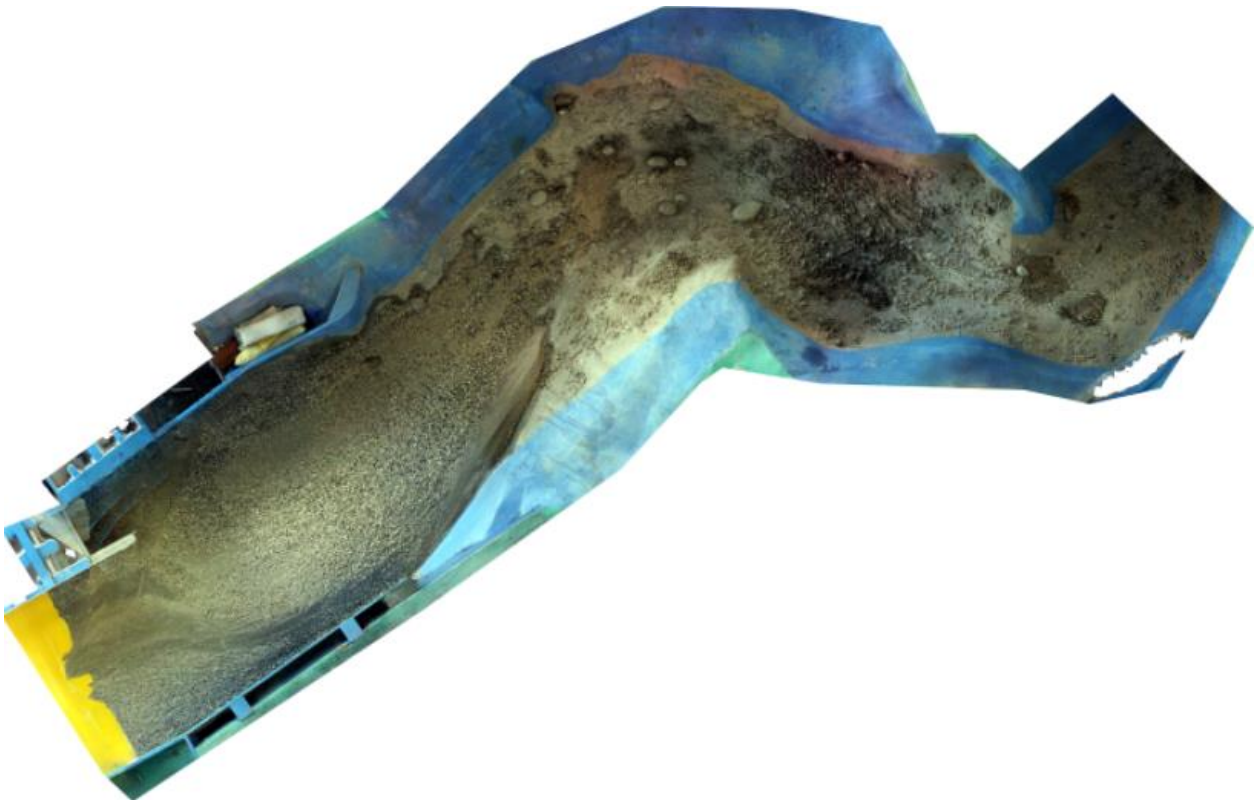
Photograph D- 70: Beginning of Flushing scenario, at 45m3/sec flow, (FRHD)

Photograph D- 71: During Flushing scenario, at 45m3/sec flow, (FRHD)

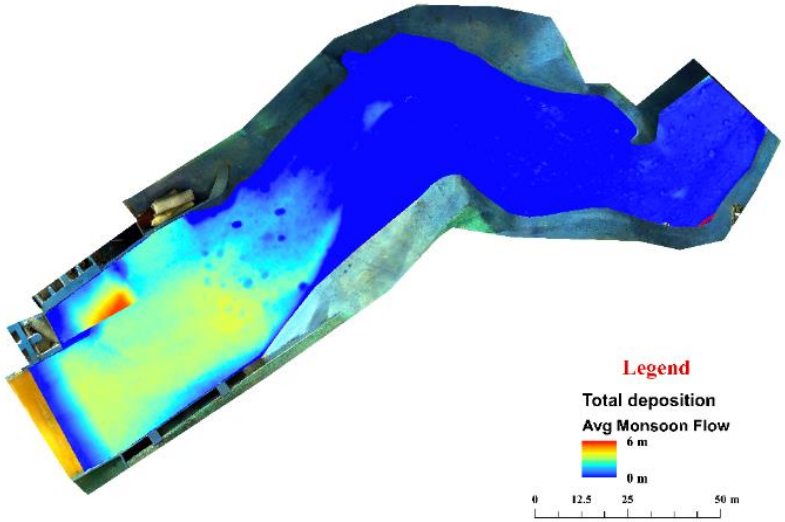
Photograph D- 72: Flushing operation complete scenario, at 45m3/sec flow, (FRHD)



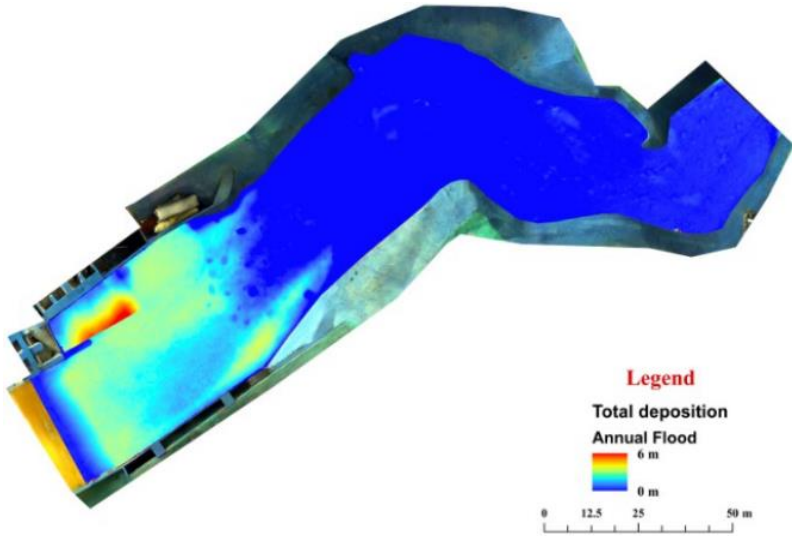
Photograph D- 73: Plan View of Model area for Final Recommended Hydraulic Design (FRHD)



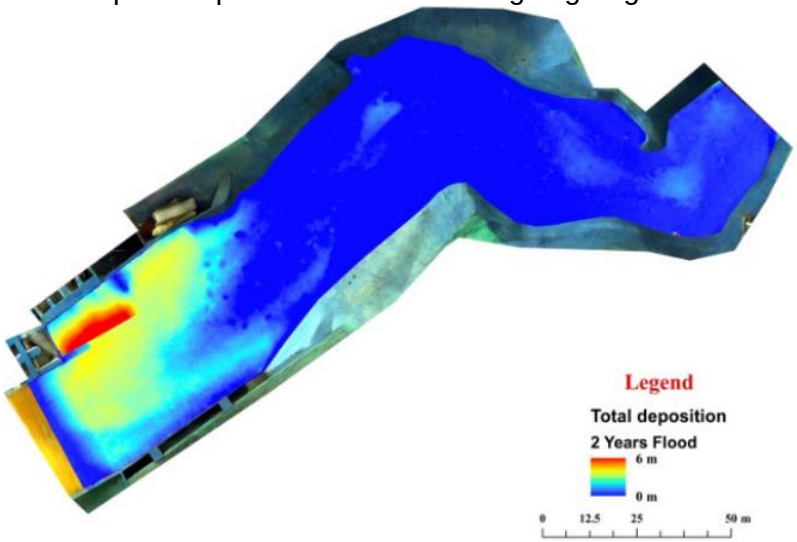
Photograph D- 74 Plan View of River bed deposition pattern at average monsoon flow, (FRHD)



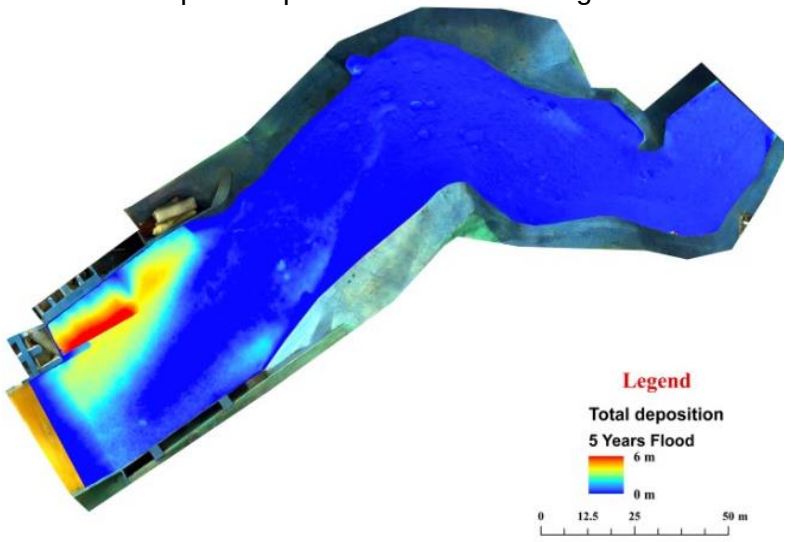
Sediment dipostion pattern observed during avgerage monsson flow



Sediment dipostion pattern observed during annual flood

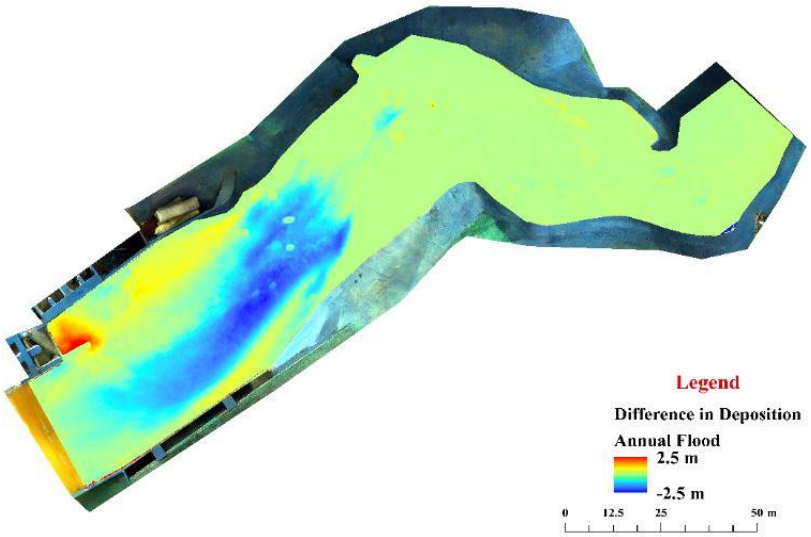


Sediment dipostion pattern observed during 2 years flood

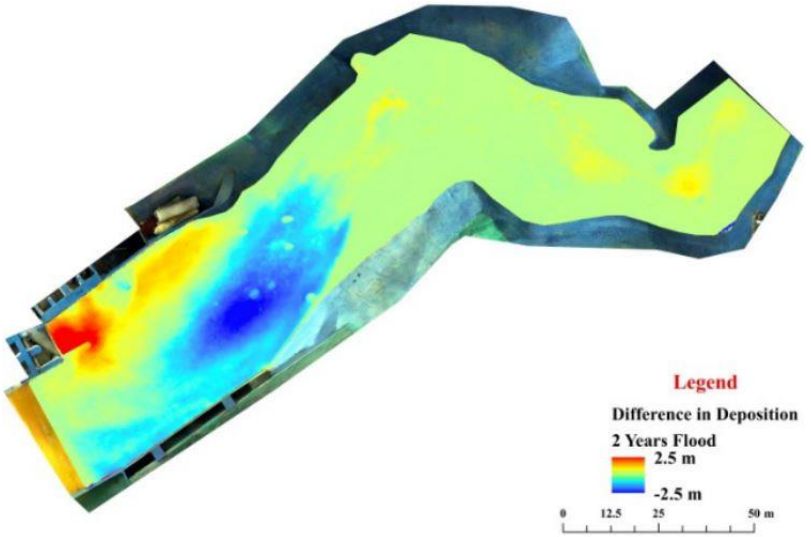


Sediment dipostion pattern observed during 5 years flood

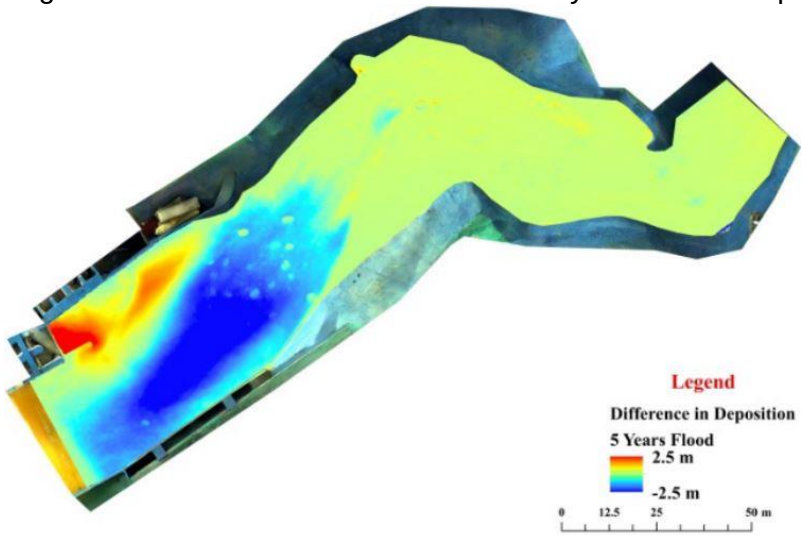
Photograph D- 75: Sediment dipostion pattern observed on Final Recommended Hydraulic Design bed structure, (FRHD)



Changes in Sediment dipostion pattern observed during annual flood compared to avgerage monsson flow



Changes in Sediment dipostion pattern observed during 2 years flood compared to avgerage monsson flow



Changes in Sediment dipostion pattern observed during 5 years flood compared to avg monsson flow

Photograph D- 76: Changes in Sediment dipostion pattern observed compared to Average monsoon flow bed, (FRHD)

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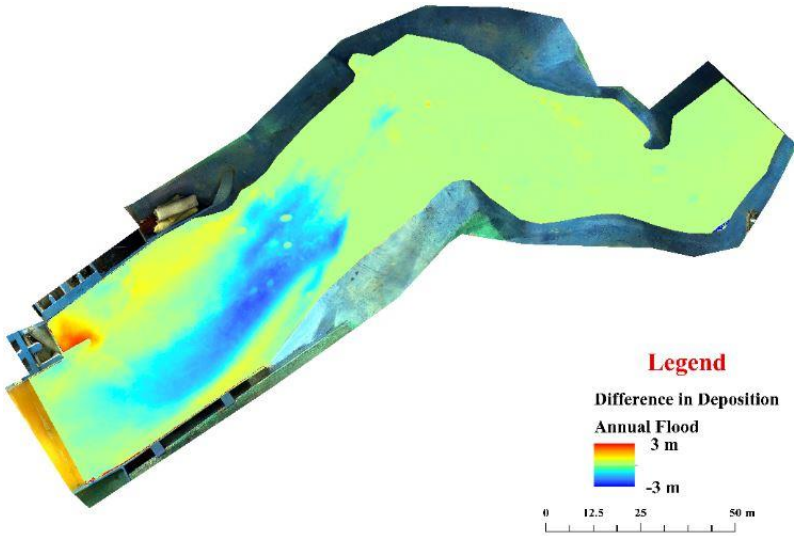
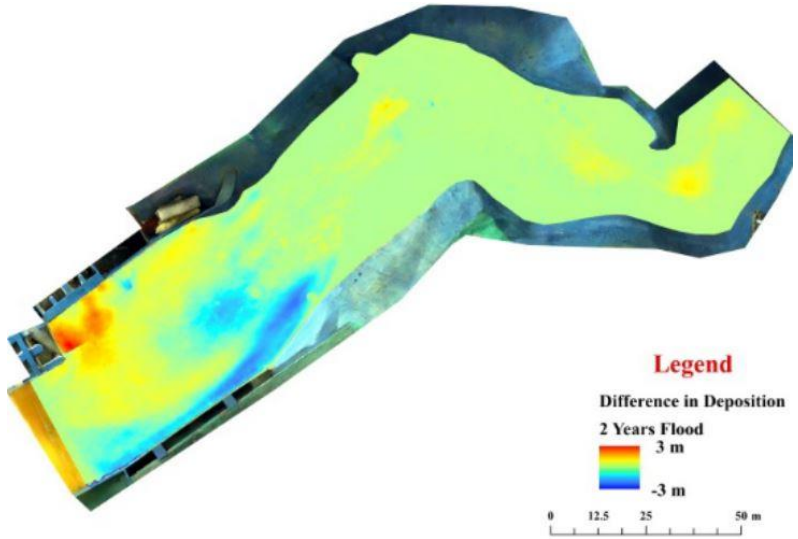
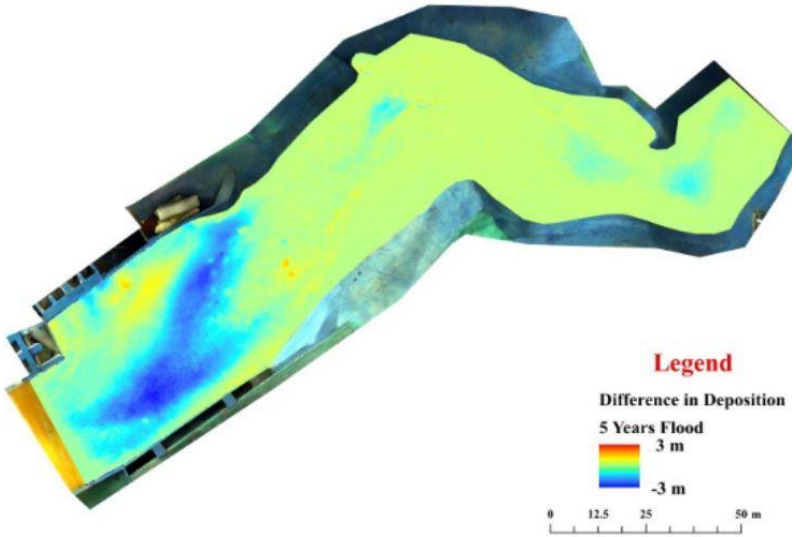


Bed level difference between Annual Flood Vs Avg. monsoon flow bed



Bed level difference between 2 yrs Flood Vs Annual Flood bed



Bed level difference between 5 yrs Flood Vs 2 yrs Flood bed

Photograph D- 77: Gradual change in Deposition Bed level with incremental flood scenario, (FRHD)