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Tamakoshi V Hydroelectric Project

Detailed Engineering Design and Tender Document Preparation

Nepal Electricity Authority

July 2019

Detailed Design Report

Main Report

20-31-00053

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Detailed Design Report

Main Report

Project number	20-31-00053
Subject	
Project manager	Dr. Klaus Moeller
Country	Nepal

Project title: **Tamakoshi V Hydroelectric Project**

Detailed Engineering Design and Tender Document Preparation

Client: Nepal Electricity Authority
Durbarmarg, Kathmandu
Nepal

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EXECUTIVE SUMMARY

Foreword

This Executive Summary Report is conceived as an umbrella document for the Report on the Detailed Engineering Design of the Tamakoshi V Hydroelectric Project. It is aimed at two sets of readers, namely:

- the executive decision makers who require key information without the background detail, and
- the technical readers who require an overview of the design studies and their main conclusions before entering into the detail.

The summary draws its information from the main report components which in turn draw information from the Album of Drawings and the Reports on Design Criteria, Topographic Surveying, Geology, Hydrology, Sediment as well as Cost and Quantity Estimating, which have been prepared within the framework of the present Consulting Contract.

The Tamakoshi V Hydroelectric Project is located in the Dolakha District of the Janakpur Zone along the Tamakoshi River, about 90 (direct) km east of the national capital Kathmandu. By road, the distance from Kathmandu to the project area is some 170 km. The location is shown in Figure 1-1.

Figure 1: Location of the Project

The Tamakoshi V Hydroelectric Project is designed as a tandem (slave) project to the Upper Tamakoshi Hydroelectric Project (UTK HEP) which at the time of release of this report had been under construction since year 2011 and was nearing completion. The principal concept adopted for the slave project was that Tamakoshi V would take the turbine water directly from the tailrace of UTK HEP; because of this concept it was not required to design the Tamakoshi V HEP with a separate dam or river water desanding facilities.

On the other hand this concept imposes certain restrictions on the design of the Tamakoshi V HEP. The major restriction to be observed are:

- The Tamakoshi V HEP has to be designed for the same rated plant discharge as UTK HEP.
- Operating limits earlier defined for the UTK HEP operation will similarly apply also to Tamakoshi V HEP. This concerns in particular the acceptable river discharge which, when surpassed, will cause UTK HEP to shut down its operation.
- The tailrace tunnel of UTK HEP is designed as channel with free surface flow. The inflow to Tamakoshi V HEP, however, has to be provided as pressurized flow, and this requires to provide a small intermediate pond in front of the Headrace Tunnel intake. Such Headpond was designed for the Tamakoshi V HEP as underground structure.
- A small spillway has to be foreseen at the Headpond for the load case that a plant stoppage occurs at Tamakoshi V while UTK HEP continues to release turbine water. Assuming a sudden load rejection at Tamakoshi V HEP, the Headpond will experience a peak inflow when the release from UTK HEP meets a simultaneous peak reverse flow from a downsurge of the Tamakoshi V Surge Tank.
- The peak inflow into the Headpond will partly be released via the spillway and partly create a backwater in the UTK HEP tailrace tunnel which will propagate upstream. The Headpond and Spillway have to be designed such that the backwater in the tailrace tunnel is limited and does not affect the operation of the UTK HEP turbines.
- The turbine water released from UTK HEP is considered essentially free from sediments. Nevertheless the desander basins of UTK HEP are designed to trap a preselected portion of sediments of defined grain diameter, which was determined as suitable for the Pelton turbines installed at UTK HEP. Tamakoshi V HEP will be equipped with Francis turbines, which have to be selected with design parameters that suit the operation with turbine water of the same sediment content.

The design of the Tamakoshi V HEP had to take all of the above restrictions into account.

Scope of Services

The overall objectives of the consulting services, as defined by NEA in the contractual Terms of Reference (ToR) are to carry the detailed engineering design of the Tamakoshi V Hydroelectric Project and, concurrently with the elaboration of that design, to prepare Tender Documents for international competitive tendering in accordance with the Government of Nepal's Standard Bidding Procedures.

This report relates to the detailed engineering design and in this particular regard, the scope of services covered the following:

- To conduct topographic surveys, hydrological studies and geological investigations to serve as the basis for the design of the hydropower scheme taking into account all relevant previous planning studies,

- To carry out the optimization of selected project components (like the optimization of the alignment and diameters of the power waterways, the number and sizing of generating units, etc.) to support NEA in identifying the preferred project configuration,
- To conduct a full detailed design, including an update of the Environmental Impact Assessment prepared for the project at feasibility design level,
- To prepare a Construction Schedule and an Engineer's Cost Estimate for the optimized project configuration, and
- To conduct a financial analysis of the optimized project configuration.

Optimized Project Configuration

The optimized project configuration was determined in several consecutive steps focusing on (a) the alignment of the Headrace Tunnel, (b) the alignment of the Tailrace Tunnel and location of the Outlet Structure, (c) the selection of the number and sizes of the generating units, and (d) the verification of the diameters of the power waterways. These optimizations were principally carried out for Tamakoshi V HEP operating at rated condition, i.e. at a plant (turbine) discharge of 66 m³/s, which corresponds to the discharge released from UTK HEP during peak operation.

Especially the selection of the number and sizes of the generating units, however, required extensive clarifications in regard to the intended operation of the upstream project Upper Tamakoshi HEP. From these clarifications it became obvious that UTK HEP is foreseen to operate during the dry season over a substantial part of the time at a minimum plant discharge of only 1.5 m³/s. This discharge is considered as a spinning reserve discharge which shall allow the turbines to ramp up within shortest time if power demand in the grid requires such action.

Whereas the above minimum discharge can be turbined at UTK HEP while operating one of the six Pelton turbines at minimum flow condition, this was found unfeasible for Tamakoshi V HEP. For the downstream project the equipment configuration comprising three Francis turbines of equal size was determined to match the unit installation at UTK HEP best and to feature the most favourable operation conditions over the expected range of partial plant discharges. Since, however, the Francis turbines designed at this size do not allow to be operated at the mentioned UTK minimum release, the equipment configuration for Tamakoshi V principally comprising three main turbine units was supplemented by a small unit which is designed to operate at specifically small discharges. The overall equipment configuration was scrutinized in detailed energy generation simulations, and the provision of the small hydro unit was verified as feasible.

Given the above findings derived from the equipment combination considerations the optimized project configuration was determined to comprise

- 3 Francis type generating units for a rated discharge of 22 m³/s, with a rated capacity of 31.6 MW, and
- 1 Francis type generating unit for a rated discharge of 3.3 m³/s, with a rated capacity of 5.0 MW.

Above installed capacities refer to the power available at the main transformer high voltage terminals. The total installed plant capacity is determined as 99.8 MW at the main transformer high voltage side.

Physical Project Setting

The Tamakoshi V HEP will be constructed and operated in rather remote area in the central part of the Dolakha district, with its Power Station located some 20 km north east of the district capital Charikot and about 4 km away from the next larger village Singati Bazar. In the project area the Tamakoshi valley experienced some development during the (still ongoing) construction of the UTK HEP further upstream, predominantly marked out by the construction of the access road from Charikot via Singati to Gongar village near the UTK powerhouse. This road, which directly passes by the Tamakoshi V Power Station and Headpond areas, improved the access conditions to the site significantly.

The project area is characterised by a narrow valley with steep, sometimes vertical, rock slopes, narrow and deeply incised side valleys, alpine forests as well as farm land and grazing meadows at higher altitude. Much of the Tamakoshi V structures are constructed underground, and no direct influence on the landscape is expected. Indirect impacts are, nevertheless, caused by tunnel portals and associated access roads, quarry and deposit areas, on-surface buildings and temporary structures.

The physical conditions at the sites of the individual project structures and in the surrounding area within which the project will be constructed and operated have been assessed by walk-trough surveys. The main conclusions of that assessment are:

- In the surrounding of the Headworks the Spillway Terminal Structure (STS) will be the only permanent structure at surface. Its construction will require the temporary deviation of the public road Singati-Gongar; once construction of the STS is complete the road will be relocated back to its original alignment and pass over the deck slab of the STS.
- Two access roads will be constructed along the route of the Headrace Tunnel (HRT). They require bridges to be constructed across Tamakoshi River, since the public road follows the left river bank where the right bank accesses are planned. The access routes are generally foreseen as temporary routes.
- Some land take is foreseen on the right bank of Tamakoshi River between the confluence of Oran Khola and the Jamune bridge. This land will be used for the construction of the Employer’s and Engineer’s Permanent Camp.
- The most prominent need for land at surface is at Suritar, a right bank terrace area along Tamakoshi River directly downstream from the confluence with Khare Khola. This land will be used as Service Area to accommodate the Power Station outdoor structures, i.e. the Terminal & Ventilation Building, the portal of the Main Access Tunnel, the Operation Building and the Workshop Building (to mention the most important structures). In addition, some area will be occupied by the deck slab of the Outlet Structure, which is located directly adjacent on the mountain side of the public road. Service roads and parking areas will need further land for their arrangement. All areas will be located at flood safe level or, where this is not possible, protected by respective flood protection walls.
- Some land will be required for temporary use opposite of the Outlet Structure on the left bank of Tamakoshi River. The Outlet Structure will have to be protected by a cofferdam during its construction, and this dam will likely narrow down the river cross section over a short (estimated 200 m long) stretch. The affected area on the left bank is presently used as a gravel quarry and meadow, however, generally uncultivated; it can be reinstated to original condition upon removal of the cofferdam.
- Further land is required for temporary use by site installations, or permanently influenced since foreseen as spoil tip or stone quarry areas. The respective land plots are almost exclusively located in direct vicinity of the course of Tamakoshi River and are not expected to significantly impair the availability of land used by the local communities for living or farming.

It is considered as a major advantage associated with the design concept adopted for the Tamakoshi V Hydroelectric Project that this project does not require a separate dam structure in the river course. Consequentyl, all side impacts associated with such dam structure, like the creation of a reservoir, the provision of a spillway dimensioned for a design flood of noteworthy peak discharge, or the provision of flushing facilities which may be required for reservoir sedimentation management are obsolete.

The river channel of the Tamakoshi River between the Headworks and the Outlet Structure are thereby widely left untouched by the implementation of the Tamakoshi V HEP.

Project Layout

The main components of the Tamakoshi V HEP when proceeding from up- to downstream are listed in the following short description together with some key data. A respective complete set of data and salient features of these components is compiled in Chapter 4 further below.

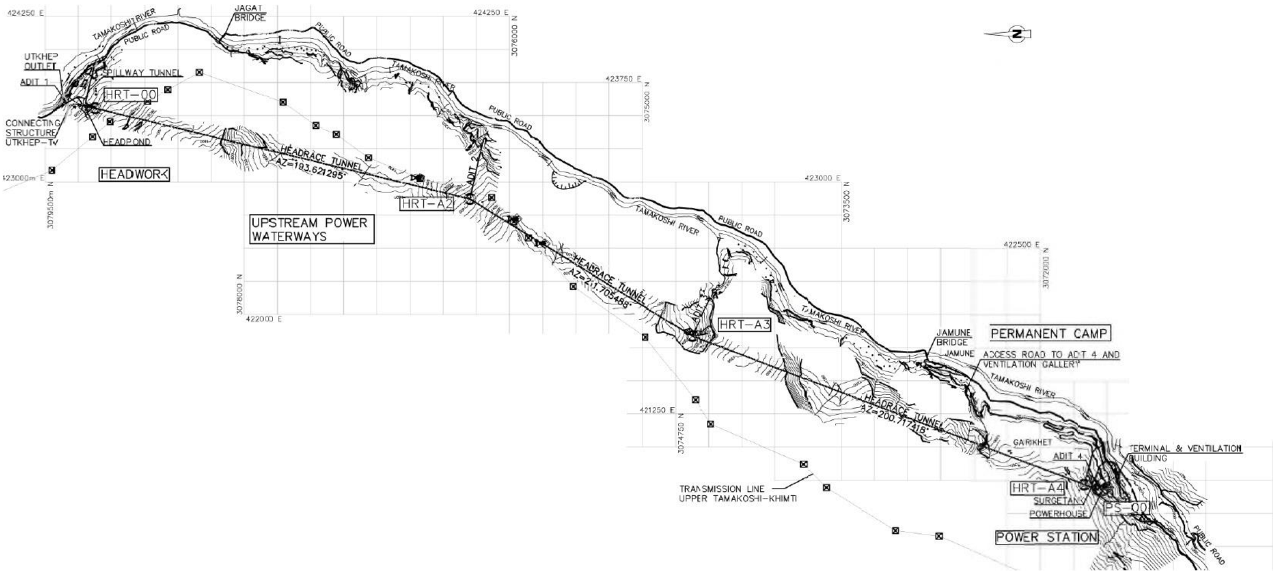

Figure 2: Project Layout, Overview

- Headworks

  The Headworks comprise the Connecting Tunnel, the Headpond and the Spillway as hydraulic structures. The Spillway itself can be separated into the lateral Spillway Weir, the Spillway Tunnel and the Spillway Terminal Structure. All these structures except the Spillway Terminal Structure are arranged underground.

  For construction and servicing purposes two tunnels are provided in the Headworks area: These are (1) the Access Tunnel to Connecting Tunnel and (2) the Spillway Aeration Tunnel. The Access Tunnel to Connecting Tunnel (Adit 1 during construction) branches off from the Access Tunnel to the Connecting Structure, a gated underground structure which is foreseen to direct the UTK tailrace flow either into the Connecting Tunnel of Tamakoshi V HEP or back into the Tamakoshi River. Due to this design, the Access Tunnel to Connecting Tunnel does not feature a separate tunnel portal at surface.

- Upstream Power Waterways

The Upstream Power Waterways include the about 8.1 km long Headrace Tunnel (HRT), the Upstream Surge Tank, the Upstream Valve Chamber, the Pressure Shaft (PS), the High Pressure Tunnel (HPT) and the Upstream Manifolds. The HRT is designed as concrete lined tunnel with 5.60 m diameter, the Surge Tank as concrete lined shaft structure with 15 m diameter and almost 50 m effective height. Downstream from the Surge Tank the steel lined part of the waterway begins, which is designed along the PS and HPT with 4.20 m inner diameter. The steel lined manifolds comprise three main branches to the main generating units and one small branch pipe to the small hydro unit.

The Upstream Valve Chamber is located immediately downstream from the start of the steel lined tunnel. The main butterfly valve which can be used to isolate the HRT from the PS will be installed here; it can be used for emergency closure and allows to keep the HRT filled with water when the PS shall be accessed for inspection.

For construction purposes four access tunnels are foreseen to be constructed to the different points along the waterway. Three of them are at high elevation, denominated as Adits 2, 3 and 4, where Adits 2 and 3 are temporary structures. Adit 4 will during the operation phase provide a downstream access to the HRT close to the Surge Tank and through a branch tunnel access to the Upstream Valve Chamber. The fourth access is at low level, leading to the HPT which it meets directly upstream from the first bifurcator. This latter access will be used to bring in the liner cans for the lower part of the steel lined waterway.

- Downstream Power Waterways

The Upstream Power Waterways consist of the Downstream Manifolds and the about 440 m long Tailrace Tunnel (TRT). The TRT is designed as concrete lined tunnel with the same diameter as the HRT; the concrete lined manifolds feature diameters which are associated with similar flow velocities as computed for the TRT under plant rated conditions.

One access tunnels to the TRT is foreseen for construction purposes. It will allow to excavate the TRT towards the Powerhouse Cavern as well as in direction to the Outlet Structure from underground, leaving a short rock plug in the Outlet Tunnel section in place over most of the construction period. The safety against flooding of the underground structures in the Power Station area is thereby significantly improved.

- Outlet Structure

This structure can be distinguished into the Gate & Access Shaft, the Outlet Structure Tunnel and the Tailbay. The deck of the Gate & Access Shaft is located directly adjacent to the public road and thus easily accessible also by heavy weight vehicles. The Outlet Structure Tunnel is aligned at low elevation and exits into the Tailbay at an elevation that secures sufficient water cover above the tunnel soffit at all operation conditions. The water level in the Tailbay will be controlled by the Tailbay end sill and the river water level.

Access to the construction pit of the Tailbay and most downstream section of the Outlet Structure Tunnel has been foreseen along a construction road placed on the downstream slope of the coffer-dam.

- Power Station with Service Tunnels and Transmission Line Corridors

The Power Station comprises all major structure required for the generation of energy, i.e. the Powerhouse Cavern, the Transformer Cavern and the Bus Duct Galleries underground, and the Terminal & Ventilation Building with Take-off Yards, the Operation Building and the Workshop Building above ground in the Service Area. The Powerhouse Cavern accommodates the four generating units and all auxiliary equipment required to operate the units, whereas the Transformer Cavern provides space for the main transformers and Gas Insulated Switchgear (GIS). The energy is transferred from the generating units to the main transformers at medium voltage level through the bus ducts, and from the main transformers via the GIS to the outdoor Take-off Yards at high voltage level via XLPE cables installed in the Cable & Ventilation Tunnel (CVT).

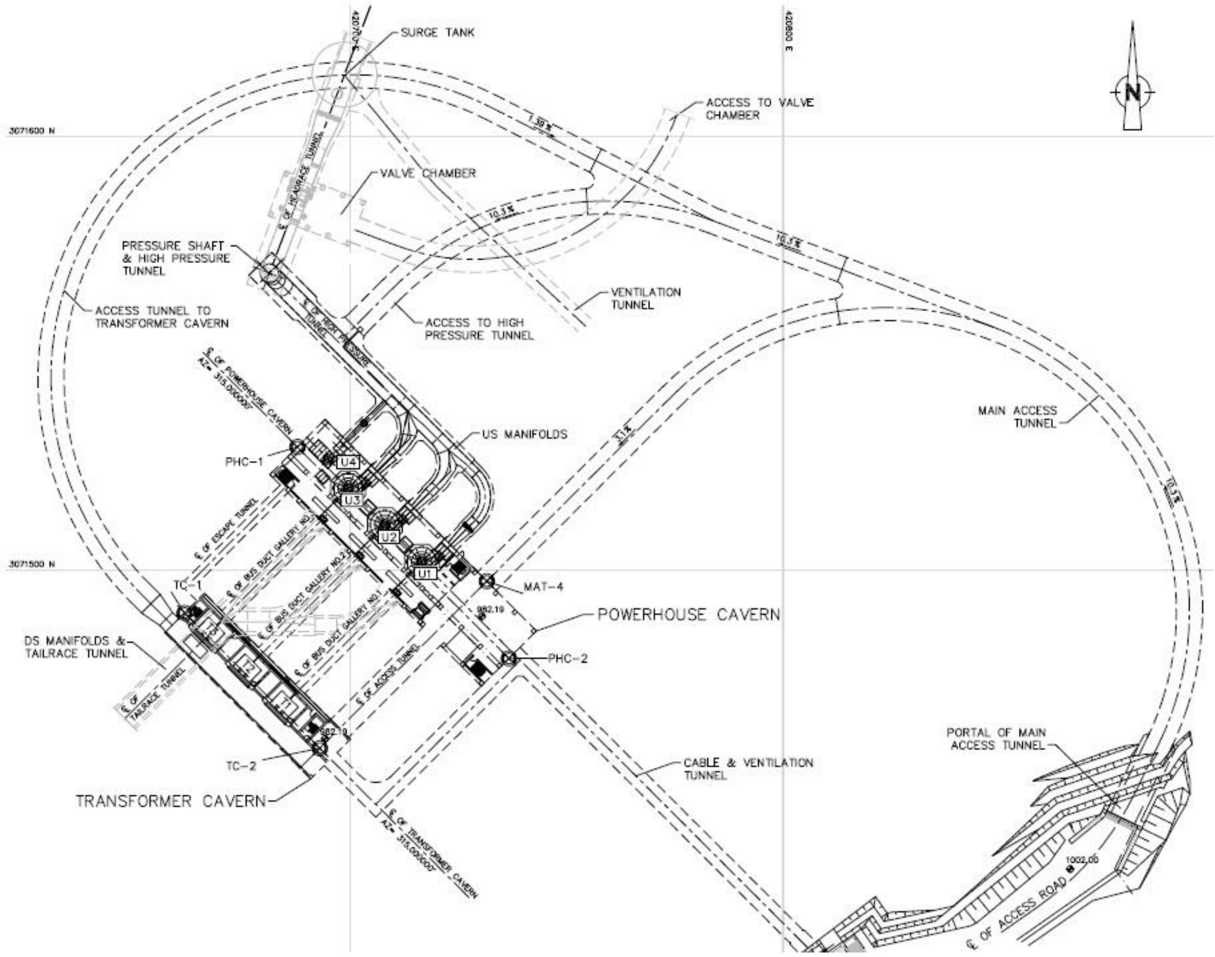

Figure 3: Structures in the Power Station Area, Layout

Several Service Tunnels are foreseen in the Power Station Area. The Main Access Tunnel (MAT) to the Powerhouse and Transformer Caverns, together with branches to the High Pressure Tunnel and rear end of the Transformer Cavern, provides paved driveways to the underground locations to which access for vehicles needs to be established (whether for construction or operation purposes). The CVT is a multi-purpose tunnel serving for cable and service pipe routing, ventilation, access and escape way provision. The Escape Tunnel and Bus Duct Galleries are foreseen in the direct vicinity of the two caverns as functional tunnel routes.

All the above tunnels have also been foreseen to provide during construction early access to the various locations of underground construction activities. In this phase, redundant access to the locations is especially important since the tunnels may be temporarily blocked when the tunnel linings and finishings are installed.

The link to the existing transmission line from UTK HEP to Khimti SS was designed as loop in / loop out (LILO) arrangement. The transmission line coming from UTK HEP will be aligned downslope to Take-off Yard no. 1, from where XLPE cables lead to the GIS in the Transformer Cavern. In similar manner, XLPE cables will lead from the GIS to Take-off Yard no. 2, where the starting point of the outgoing line to Khimti SS is located; the transmission line will from this point be routed uphill to join the existing line route at high elevation. A Switching Station provided in the Service Area will allow to bypass the Tamakoshi V scheme and connect the UTK HEP and Khimti line sections directly.

- Permanent Camp

The Employer’s and Engineer’s Permanent Camp was decided to be located in an area on the right bank of the Tamakoshi River between the side valley of Oran Khola and the Jamune bridge. The camp location can easily be reached via a project road which will branch of from the public road close to Jamune bridge and be constructed along the right river bank. This road will be implemented under the same contract which will concluded for the construction of the camp itself.

Energy Generated by the Project

The energy which will be generated by the Tamakoshi V HEP has been estimated for four different scenarios, which principally take into account whether water will also be available from the partial diversion of the Rolwaling River into the Upper Tamakoshi reservoir or not, and whether planned outages are included in the simulation of the generation scenarios or not. The total annual energy amounts which will be produced by Tamakoshi V (referring to the high voltage terminals of the main transformers) were derived as follows:

- Rolwaling not considered, Outages not simulated 464.77 GWh/a
- Rolwaling not considered, Outages simulated 461.66 GWh/a
- Rolwaling considered, Outages not simulated 516.55 GWh/a
- Rolwaling considered, Outages simulated 512.33 GWh/a

From these data it is obvious that the consideration of planned outages has only a marginal effect on the generation capacity; the supply of addition water through the Rolwaling diversion is, to the contrary, of significant impact.

Project Cost

The table on the following page provides an overview over the estimates of the total project cost of the Tamakoshi V Hydroelectric Project.

The individual cost components were calculated using a conversion rate of 1.00 USD = 112.12 NPR which was determined as average conversion rate for the 2018-19 (April-March). Miscellaneous costs were generally assumed to be 3% of the estimated base costs; The EPC Cost includes contingency and 3% Miscellaneous Expenses. The Contingency is assumed at 5% on Tunnel & camp works, 4.5% on Surface works, 1% on Hydro-Mechanical, Mechanical and Electrical works, 5% on Transmission line towards any exigencies. The Project Base Cost (excluding taxes& duties, financing charges and IDC) is Summarized in Equivalent US Dollars in the below Table:

Table 1: Project Base Cost is Summarized in Equivalent US Dollars

Item No.	Description	Foreign USD	Local NPR	Total USD
1	Civil Works	39,291,329	4,032,598,206	75,258,134
2	Hydromechanical Equipment	8,340,800	-	8,340,800
3	Mechanical Equipment	10,826,400	-	10,826,400
4	Electrical	23,831,600	-	23,831,600
5	Camp Works	-	467,977,213	4,173,896
6	Transmission Line	1,900,800	-	1,900,800
7	Owner’s Costs	5,577,807	292,922,469	8,190,387
	Total Base Cost			**132,522,017**

Financing Scenarios

Different financing scenarios were investigated to check for the financial viability of the Tamakoshi V HEP. In these scenarios the energy sales conditions were adopted in line with communicated NEA sales tariffs for dry and wet season energy, with all generated wet season energy being sold to NEA at its respective tariff. The interest rate was assumed at 9%, the repayment period as 15 years. The economic life of the project was considered with 30 years.

Based on these assumptions the following financial indicators were obtained which show the project to be financially viable.

Table 2: Financial Viability Indicators (Base Case)

Parameters	Unit	Values
Project IRR	%	10.81%
Project NPV	Million NRs	883.66
Benefit to Cost Ratio		1.06
Payback Period	Years	7.19

The investigated sensitivity scenarios showed that the project's financially viability varies depending on the assumptions adopted for the individual sensitivity cases. The investigated sensitivities included the payment of additional charges for transmission line losses to Upper Tamakoshi HPP, the increase in project costs, the prolongation of the construction time, the increase in the interest rate for project financing and increase in outages.

Project Implementation

The project implementation was studied by assuming construction methods which will most likely be chosen in order to meet the target of an overall construction period of estimated four years. It was strictly adhered to that the construction methods foreseen for individual activities match the assumptions made in relation to these operations when preparing the cost estimate.

The studies were based on an assumed start date of 1st July 2020. They revealed that the critical path for construction will pass along the excavation of the Tailrace Tunnel for early mucking access to the powerhouse cavern, the excavation & concreting of the powerhouse cavern, the installation of the generating units, and the sequential commissioning of the generating units. For the study it was assumed that the commissioning of the small hydro unit will be carried out in parallel with the main units and will not control the project completion date. With this construction program the completion date was determined as of 3rd May 2024.

In order to facilitate the swift implementation of the Tamakoshi V Hydroelectric Project, the Employer should complete the following tasks at the earliest time possible:

- Land acquisition for all permanent surface components of the project, including access roads, transmission line, the camp for the Employer and the Engineer, and all adit and tunnel portals and spoil areas.
- Conclude lease agreements for lands required for temporary facilities such as camps and yards for the contractor(s).
- Conclude the contract for the implementation of the camp for the Employer and the Engineer including the access road to and within the camp area.

1 INTRODUCTION

1.1 Components of the Detailed Design Report

Considering the very complex nature of the Tamakoshi V Hydroelectric Project it had to be anticipated that the Report on the Detailed Engineering Design would turn out as a correspondingly voluminous document in the presentation of the individual topics associated with the detailing of the project. In foresight of this anticipation it was therefore considered mandatory that a clear report structure was defined in the very early stage of the report preparation.

The report structure was subsequently coordinated with the Employer to pave the way for the elaboration of the individual report components to follow. The agreed structuring of the report and the overview over the individual report components are presented for reference in the listing provided below.

The main structuring foresees six components for the Detailed Design Report as listed below following the Main Report:

- Part A Project Design Report
- Part B Design Criteria Report
- Part C Construction Planning & Quantities
- Part D Costs & Project Economics
- Part E Album of Drawings
- Part F Field Investigations Report

Especially the Parts A and F of the report proved to require the presentation of a comprehensive amount of information. Therefore, subdivisions were implemented for these parts as shown here below.

Structuring of the Detailed Design Report under Part A:

A1	-	Final Layout Report
A2	-	Descriptions of Structures
A3	-	Design Criteria, Methods used in Design, Calculations & Results
Chapter	01	Headworks
Chapter	02	Water conveying Tunnels & Surge Tank
Chapter	03	Outlet Structure
Chapter	04	Power Station
Chapter	05	Service Tunnels
Chapter	06	Hydromechanical Equipment
Chapter	07	Power Station Mechanical Equipment
Chapter	08	Power Station Electrical Equipment
Chapter	09	Transmission Lines
Chapter	10	Permanent Camps & Roads
Chapter	11	Temporary Project Structures

Part D of the report was subdivided into two parts as follows:

D1 - Cost Estimate
D2 - Economic/financial Analysis

In a similar manner, Part F of the report was separated into six sub-parts listed here below:

F1 - Topographic Survey
F2 - Hydrological, Meteorological and Sedimentological Investigation
F3 - Geological, Geotechnical and Construction Material Investigation
F4 - Seismological Investigation
F5 - Investigation related to Glacier Lake Outburst Flood (GLOF)
F6 - Environmental Investigation

With this concept of structuring the overall report it was accomplished that project specific information is presented separately from information of more general nature. Depending on the complexity of the information to be presented, the bound volumes of the Detailed Design Report were organized in line with the above presented structuring.

1.2 Structures and Components of the Project

The Tamakoshi V HEP can be divided into the following structures and components:

1. Headworks, comprising:
   - Connecting Tunnel;
   - Headpond;
   - Spillway (with Spillway Weir, Spillway Tunnel and Spillway Terminal Structure);
2. Water Conveying Tunnels & Surge Tank, comprising:
   - Headrace Tunnel;
   - Surge Tank;
   - U/S Valve Chamber;
   - Pressure Shaft;
   - High Pressure Tunnel & U/S Manifolds;
   - D/S Manifolds & Tailrace Tunnel;
3. Outlet Structure comprising:
   - Gate & Access Shaft
   - Outlet Structure Tunnel
   - Tailbay
4. Power Station, comprising:
   - Powerhouse Cavern
   - Transformer Cavern
   - Bust Duct Galleries

- Terminal & Ventilation Building
- Operation Building
- Workshop Building

5. Service Tunnels, comprising:
    - Main Access Tunnel & Access Tunnel to Transformer Cavern
    - Cable & Ventilation Tunnel
    - Escape Tunnel
    - Access Tunnel to Connection Tunnel (Adit 1 during construction)
    - Aeration Tunnel to Spillway Tunnel (Access to Spillway Tunnel during construction)
    - Access Tunnel to U/S Valve Chamber & Adit 4 Plug (Adit 4 during construction)
    - Ventilation Tunnel Surge Tank
    - Access Tunnel to ~~High Pressure Tunnel~~ U/S Manifold (a Branch of MAT)
6. Hydromechanical Equipment
7. Power Station Mechanical Equipment
8. Power Station Electrical Equipment
9. Transmission Lines
10. Permanent Camps & Roads
11. Temporary Structures, comprising:
    - Adit 2 to Headrace Tunnel
    - Adit 3 to Headrace Tunnel
    - Adit to Tailrace Tunnel
    - Cofferdam at Outlet Structure

The above structures and components are described in detail in the chapters of Part A3 of the Detailed Design Report. In addition, Part A2 of the Detailed Design Report provides descriptions of the civil works structures together with their salient features; this has been dispensed of for the project's equipment components because of the high degree of detailing included in the respective chapters of Part A3.

1.3 Salient Features

1.3.1 Outline of the Project

The Tamakoshi V HEP is located in the Central Development Region of Dolakha District of Janakpur Zone. The project area is situated within Longitude 86°13'11" to 86°13'56" East and Latitude 27°47'30" to 27°50'00" North. The whole project area lies in Khare, Orang and Lamabagar Village Development Committees. The project is conceptualized to develop as a tandem operation project with Upper Tamakoshi HEP. The intake site / underground interconnection system with Upper Tamakoshi tailrace outlet is located in Lamabagar VDC whereas the underground powerhouse lies at the right bank of Tamakoshi River just downstream of the Suri River confluence with Tamakoshi River.

In the following, relevant key data of the project have been compiled based on the results derived from the Detailed Project Design.

1.3.2 Plant Operating Conditions

1.3.2.1 Plant Rated Conditions

• Rated Gross Head	173.45	m
• Rated Net Head	162.35	m
• Rated Tailwater Level	984.55	masl
• Rated Discharge	66	m³/s
• Rated Head Loss (d/s waterway)	0.80	m
• Rated Capacity	94.8	MW

1.3.2.2 Installed Generating Equipment

• No. of Units	4	
• Rated Discharge of Main Units	22	m³/s
• Rated Discharge of Small Hydro Unit	3.3	m³/s
• Total Turbine Capacity	99.8	MW
• Generator Capacity (Main Units)	38	MVA
• Generator Capacity (Small Hydro Unit, approx.)	6	MVA
• Transformer Capacity	2 x 40 / 1 x 44	MVA

1.3.2.3 Project Hydrology

• Average Annual Flow	69.51	m³/s
• Design Flood at Headworks	6,000	m³/s
• Design Flood at Power Station	6,000	m³/s

1.3.3 Headworks

1.3.3.1 Connecting Tunnel

• length	103.20	m
• width	6.80	m
• maximum depth	3.40	m
• invert slope	0.1258	%
• invert elevation at start of horizontal bend	1,155.02	m asl.
• invert elevation at entrance to Headpond	1,154.89	m asl.

1.3.3.2 Headpond

• length	74.01	m
• width	6.80 … 12.77	m
• water level at plant rated condition	1,158.00	m asl.
• maximum depth	14.00	m
• invert elevation at end wall	1,144.00	m asl.

1.3.3.3 Spillway

<u>Spillway Weir and Collecting Channel</u>

• type	free overflow concrete weir	
• length	55.50	m
• crest level	1,158.20	m asl.
• freeboard at plant rated condition	0.20	m
• maximum surcharge	0.90	m
• design discharge	103	m^3/s
• weir shape	WES standard	

<u>Spillway Tunnel</u>

• type	free surface flow channel	
• length	311.50	m
• channel width	4.40	m
• channel depth	3.55	m
• invert level at start section	1,153.11	m asl.
• invert level at exit section	1,150.21	m asl.
• channel cross section	rectangular	

<u>Spillway Terminal Structure</u>

• type	gated box type r.c. structure	
• length	31.51	m
• width (box frame)	7.40	m
• maximum height	11.35	m
• invert level of apron	1,150.00	m asl.

1.3.3.4 Access Tunnel to Connecting Tunnel (Adit 1)

- length 185.92 m
- width 4.20 m
- maximum invert slope 10.72 %
- invert elevation at start of horizontal bend 1,169.18 m asl.
- invert elevation at entrance to Headpond 1,154.89 m asl.

1.3.3.5 Construction Adit to Spillway Tunnel

- length 90.34 m
- width 4.20 m
- maximum invert slope 13.69 %
- invert elevation at start of horizontal bend 1,163.25 m asl.
- invert elevation at entrance to Headpond 1,152.90 m asl.

1.3.4 Water Conveying Tunnels & Surge Tank

1.3.4.1 Headrace Tunnel

- Total length of headace tunnel (HRT) 8098.0 m
- Length of shotcrete lined section of HRT 1000.0 m
- length of concrete lined section 7,098.0 m
- length of steel lined section (incl. valve) 41.90 m
- inner diameter (concrete lining) 5.60 m
- inner diameter (steel lining) 4.20 m
- thickness of concrete lining 0.40 m
- longitudinal slope 0.4209 %
- center line elevation at start section 1,149.80 m asl.
- center line elevation at end section 1,114.96 m asl.

1.3.4.2 Surge Tank

- type circular shaft surge tank
- top of concrete lining 1,180.00 m asl.
- surge tank bottom 1,129.40 m asl.
- highest upsurge 1,179.30 m asl.
- lowest downsurge 1,130.50 m asl.
- inner diameter 15.00 m asl.

• diameter of connecting shaft	2.50	m asl.

1.3.4.3 U/S Valve Chamber

• length	20.00	m
• width	11.00	m
• height	16.90	m
• invert elevation	1,110.60	m asl.
• span betwenn crane rails	9.60	m

1.3.4.4 Pressure Shaft

• type	steel lined shaft	
• length including bends	152.72	m
• center line at start section	1,114.96	m asl.
• center line at end section	976.03	m asl.
• Inner diameter	4.20	m
• excavation diameter	5.00	m
• bend radii	12.60	m

1.3.4.5 High Pressure Tunnel & Upstream Manifolds

• center line elevation at start section	976.03	m asl.
• diameter at start section	4.20	m
• center line elevation at end section (U1 - U3)	974.48	m asl.
• center line elevation at end section (U4)	975.61	m asl.
• diameter at end section (U1 - U3)	2.425	m
• diameter at end section (U4)	0.95	m
• deflection of manifolds	90 deg.	

1.3.4.6 Downstream Manifolds

• length of manifolds	31.64 … 36.09	m
• manifold diameter	3.30 … 5.60	m
• longitudinal slope	1.1	%
• end section elevation	968.74	m asl.
• deflection of branches	45 deg.	

1.3.4.7 Tailrace Tunnel

• length	404.36	m
• diameter	5.60	m
• center line elevation at start section	971.90	m asl.
• center line elevation at end section	976.80	m asl.
• longitudinal slope	1.13	%
• deflection of horizontal bend	45 deg.	

1.3.5 Outlet Structure

1.3.5.1 Gate & Access Shaft

• height above waterway liner	32.46	m
• outer diameter	8.40	m
• operation platform level	1,014.00	m asl.
• hoist floor level	1,003.00	m asl.
• rod storage floor level	998.00	m asl.
• shaft cross section	circular	

1.3.5.2 Outlet Structure Tunnel

• length of tunnel	53.00	m
• clear height	5.60	m
• clear width	4.40 … 5.60	m
• centerline elevation	976.80	m asl.
• tunnel cross section	rectangular	

1.3.5.3 Tailbay

• average length of tailbay	27.28	m
• tailbay floor below tailrace tunnel invert	-1.00	m
• slope of tailbay floor towards end sill	1 : 2.35	
• end sill elevation	982.80	m asl.
• deflection of side walls	15 deg.	

1.3.6 Power Station

1.3.6.1 Powerhouse Cavern

- The main dimensions are length x width x height = 69.00 m x 18.00 m x 33.14 m.
- The cavern houses 3 Francis turbines 31.6 MW and 1 Francis turbine 5 MW for a total installed capacity of 99.8 MW.
- A 80/15 t overhead crane extends over the entire length of the Powerhouse Cavern.
- An Elevator serves the Auxiliary Floor, Turbine Floor, Generator Floor and the Machine Hall Floor.
- Two staircases are located diagonally at the opposite ends of the Powerhouse Cavern. The staircase adjacent to the MAT and the Erection Bay reaches down to the Drainage Gallery.
- The Erection Bay and the Main Access Tunnel are arranged at Machine Hall Floor level.
- The MAT passes through the Powerhouse Cavern and extends up to the Transformer Cavern.
- Two Dewatering / Pump Sumps, one between Units 1 and 2 and one between Units 2 and 3.
- A movement joint separates the Units Bay and the Erection Bay.

The floors in the Powerhouse Cavern are (from top to bottom)

- Crane Runway Floor 989.24 m asl.
- Administration Floor 985.64 m asl.
- Machine Hall Floor 982.19 m asl.
- Generator Floor 978.51 m asl.
- Turbine Floor 975.15 m asl.
- Valve Floor 971.25 m asl.
- Drainage Gallery 967.90 m asl.

1.3.6.2 Transformer Cavern

- The main dimensions are length x width x height = 47.6 m x 13.00 m x 17.95 m.
- The upper floor is served by a 10 t bridge crane.
- The upper floor is accessible by two staircases at either end of the Transformer Cavern.

1.3.6.3 Bus Duct Galleries

- 3 nos. provided between Powerhouse & Transformer Caverns
- Dimensions length x width x height 23.79 m x 3.00 x 4.00 m

1.3.6.4 Terminal & Ventilation Building

- Dimensions length x width x height 16.30 m x 15.40 x 15.00 m

- The cable shaft which connects vertically from the Terminal & Ventilation Building extends an additional 32.70 m to either of the two Take-off Yards.
- Three different floor levels were planned in the building.

1.3.6.5 Operation Building

- Dimensions length x width x height 44.50 m x 14.00 x 9.6 m (including the Water Treatment Plant)

1.3.6.6 Workshop Building

- Dimensions length x width 28.32 m x 31.35 m
- Reinforced concrete structure
- 10 t crane

1.3.7 Service Tunnels

For expediency rock support analysis and design of the Service Tunnels were divided into Type A or Type B.

1.3.7.1 Main Access Tunnel & Access Tunnel to Transformer Cavern (Type B)

- The Main Access Tunnel has a horseshoe section.
- It has the dimensions width x height 6.00 m x 6.00 m
- A concrete invert is foreseen for the MAT

1.3.7.2 Cable & Ventilation Tunnel (Type A)

The Cable & Ventilation Tunnel has a horseshoe section with dimensions width x height = 4.20 m x 5.80 m. The entire length (consisting of three legs) is about 138 m.

1.3.7.3 Escape Tunnel

The Escape Tunnel has a D-shaped section with the dimension width x height of 2.50 m x 3.00 m. The length is 30 m.

1.3.7.4 Access Tunnel to U/S Valve Chamber & Adit 4 Plug (Adit 4 during construction) (Type B)

- Dimensions width x height x length = 5.00 m x 5.80 x 311 m.
- A gated plug will be installed at the Valve Chamber end.

1.3.7.5 Ventilation Tunnel to Surge Tank (Type A)

- Dimensions width x height x length = 4.20 m x 5.60 x 72 m.

1.3.7.6 Access Tunnel to High Pressure Tunnel (Type B)

- Dimensions width x height x length as for Main Access Tunnel

1.3.8 Transmission Lines

- Voltage 220 kV, double circuit
- Total Length of LILO Arrangement 3.4 km

1.3.9 Adits to Headrace Tunnel

- Adit 2 (temporary) 513.43 m
- Adit 3 (temporary) 312.69 m
- Adit 4 (permanent) 311.69 m

1.3.10 Generated Annual Energy

1.3.10.1 Data for Tamakoshi V High Voltage Terminals

- Rolwaling not considered, scheduled outages not simulated 464.77 GWh/a
- Rolwaling not considered, scheduled outages simulated 461.66 GWh/a
- Rolwaling considered, scheduled outages not simulated 516.55 GWh/a
- Rolwaling considered, scheduled outages simulated 512.33 GWh/a

1.3.10.2 Data for Khimti S/S incoming Terminals

- Rolwaling not considered, scheduled outages not simulated 450.01 GWh/a
- Rolwaling not considered, scheduled outages simulated 447.25 GWh/a
- Rolwaling considered, scheduled outages not simulated 500.04 GWh/a
- Rolwaling considered, scheduled outages simulated 496.34 GWh/a

Detailed breakdowns are available in Subchapter 3.7.

1.3.11 Estimated Project Costs

	Foreign [USD]	Local [NPR]	Total [USD] *)
• Civil Works	39,291,329	4,032,598,206	75,258,134
• Hydromechanical Works	8,340,800		8,340,800
• Mechanical Equipment	10,826,400		10,826,400
• Electrical Equipment	23,831,600		23,831,600

•	Permanent Camp		467,977,213	4,173,896
•	Transmission Line	1,900,800		1,900,800
•	Owner's Expenses	5,577,807	292,922,468	8,190,387
•	Total Estimated Cost			132,522,017

*) at currency exchange rate of 1.00 USD = 112.12 NPR

1.3.12 Financial Indicators

Indicators for Base Case Analysis

•	Interest during Construction (IDC)	9.0	%
•	Discount Rate	10.1	%
•	Project IRR	10.81	%
•	Project NPV	883.66	mio. NRs
•	Benefit/Cost Ratio	1.06	
•	Payback Period	7.19	yrs

2 TOPOGRAPHIC SURVEY AND MAPPING

2.1 Introduction

The scope of the field investigation works for the topographic survey is specified in the "Terms of Reference (ToR)", Chapter 4 under Task 2 a) "Topographical Survey". In this place, all relevant areas of the Tamakoshi V HEP are compiled, described with respect to their intended use and associated with specific survey requirements in terms of the accuracy of the field survey.

This present report contains more specifically the following information:

- project background and information on data from earlier surveys available for the current survey;
- a description on the method and equipment used for the terrestrial survey;
- a listing of major points staked during the survey of dedicated locations;
- a listing of the dedicated areas surveyed together with the area sizes;
- a documentation of the river cross section survey in the area of the Outlet Structure.

For the surveyed areas of the Tamakoshi V HEP topographic layout maps were prepared; they are included in the relevant drawing volume attached to Part F1 of the Detailed Design Report.

2.2 Terminology and Abbreviations used in this Chapter

- GPS: Global Positioning System
- GNSS: Global Navigation Satellite System.
- PDOP: Position Dilution of Precision.
- CAD: Computer Added Design.
- DTM: Digital Terrain Model.

2.3 Objectives and Scope of the Survey

2.3.1 Objectives of the Survey

The objectives of the survey campaign are summarized below:

- conduct GNSS survey with major control tied with Upper Tamakoshi Project,
- conduct checks on the National Geodetic Trig Point using GNSS control for relative accuracy,
- conduct topographical survey of components of the Tamakoshi V HEP at desired scale,
- conduct river cross section survey at tailrace of the project,
- carry out stakeout of major components of the Tamakoshi V HEP,
- compile all results in, and submit a Survey Report.

2.3.2 Scope of the Survey

The scope of work for topographical survey includes the following:

a. Establishment of a survey control network within the project area connecting the national grid for underground works and other topographic mappings.

b. Detailed mapping of the project sites including the areas of the interconnection system, powerhouse, surge tank, at the MCT zone along the headrace tunnel alignment and portal areas in scale of 1:500 and 1 m contour interval

c. Detailed mapping of foundation areas in scale of 1:500 and 1 m level contour interval for the detail design of surface structures.

d. Detailed mapping of the headrace tunnel alignment strip in scale of 1:5000 and 5 m contour interval.

e. River cross section survey at the interval of 50 m around the tailrace site from 200 m upstream to 200 m downstream of the tailrace outlet.

f. Detailed mapping of project road alignments, camp facilities, spoil disposal areas and so on in scale of 1:500.

g. Detailed mapping of transmission line route alignments in scale of 1:1000.

2.4 Survey Planning

2.4.1 Planning Considerations

2.4.1.1 Monumentation of Control Points

Control points were established either on a concrete pillar of L x B x H size 0.15 x 0.15 x 0.60 m (for major GNSS control points) or cross chiselled on a boulder or concrete nail cemented on boulder. Every control point was marked by red enamel paint on the site. Description cards of all the major control points were prepared. The description cards are presented in Part F1 of the Detailed Design Report. The description cards of the control points depict the following information:

- ID of the control point,
- location description of the control point,
- Monumentation Mark,
- dimension to the references to the point,
- coordinates (E, N and Z) of the points.

2.4.1.2 List of Survey Equipment used

Table 2-1: List of Survey Instruments

Survey Equipment and Model	Nos.	Angle Accuracy	Distance Accuracy
TOPCON GPT 7501	1	± 1”	± (2mm + 2ppm x D)
TOPCON ES 105	1	± 1”	± (2mm + 2ppm x D)

GOWIN 8E-0688	1	± 2”	± (2mm + 2ppm x D)
GOWIN 8E-0991	1	± 2”	± (2mm + 2ppm x D)
SYGNUS-KY-1127	1	± 2”	± (2mm + 2ppm x D)

2.4.1.3 List of GNSS Receivers used

Table 2-2: List of GNSS Receivers

S. No	Survey Equipment and Model	Serial Number	Receiver ID
1	TOPCON GNSS RECEIVERS (GR-5)	1118-21457	U0CJ2QNIEBS
2	TOPCON GNSS RECEIVERS (GR-5)	1118-20942	U002SRVIBYG
3	TOPCON GNSS RECEIVERS (GR-3)	442-3175	P8OUDW85U68
4	TOPCON GNSS RECEIVERS (GR-3)	442-3150	R82SQM1YMM8

2.4.1.4 Specifications of the Static Survey

The following general specifications were used for the static survey method and post-processing of the field data to achieve the desired accuracy of the survey.

Table 2-3: Specifications of the Static Survey

Specification	Static
General Network Design	
Minimum number of reference station to control the project.	1 previous control point.
Maximum distance between the survey project boundary and network reference control stations	50 km
Minimum number of all baselines contained in a loop	100%
Field	
Maximum PDOP during station occupation	5 (75% of time)
Minimum observation time on station	1 hrs
Maximum epoch interval for data sampling	5 sec
Minimum satellite mask angle above the horizon	15 degrees
Office	
Fixed integer solution for all baselines	Yes
Ephemeris	Broadcast
Maximum loop length	50 km
Maximum misclosure per loop, in terms of loop length	15 ppm

Maximum misclosure per loop in any one component (x,y,z) not to exceed	5 cm
Repeat baseline difference in any component (x,y,z) not to exceed	10 ppm
Maximum length misclosure allowed for a baseline in a properly-weighted, least squares network adjustment	50ppm
Maximum allowable residual in any one component (x,y,z) in a properly weighted, least squares network adjustment	5 cm

2.4.2 Planning Data and Information

2.4.2.1 Old Control Point Coordinates

The available data provided were the old control point coordinates of the project, topographical maps and description cards.

Table 2-4: Coordinates of National Geodetic Trig Point

Grid Sheet No. Alignment No.	Trig B.M. No.	Description Card	Co-ordinates		Height Above Mean Sea Level	Remarks
			Easting	Northing		
158	23		423115.88	3084879.99	-	-

Table 2-5: Control Point Coordinates of Upper Tamakoshi Hydroelectric Project

Point ID.	Ground Easting (m)	Ground Northing (m)	Elevation (m)	Code
UTKHP20	423124.450	3080744.692	1288.358	UTKHP20
UTP-14	423185.048	3080862.370	1277.159	UTP-14
UTPKHP21	423485.624	3079580.206	1173.911	UTPKHP21

2.4.2.2 Post-Processing and Adjustments of GNSS Observation

The post-processing was completed in two separate jobs. The first job was based on the observation between UTPKHP-21, UTP-14, GPS-11 and GPS-12. UTPKHP-21 was fixed as base station, and the coordinates of the remaining points were calculated. In the second job the coordinates of UTPKHP-21 along with GPS-11 and GPS-12 whose coordinates were obtained from the first job were also used as major control points. UTP-14 was left from the group to check the difference from two separate post-processings. The Grid coordinates then were converted to ground coordinates using an average scale factor from the scale factor of the points at the top and bottom ends of the project. The details of the parameters are presented in Part F1 of this Detailed Design Report.

2.5 Control Survey

2.5.1 Connection to National Geodetic Network

The coordinates of the national geodetic trig point and the control point coordinates of Upper Tamakoshi HEP used for the connection are presented above in the section on old control point coordinates.

2.5.2 Horizontal and Vertical Control Survey

2.5.2.1 GNSS Field Survey

The Standard Static GNSS survey method was adopted and simultaneous measurements of 4 receivers were made for duration of 30 minutes to 2 hours giving 6 correlated vectors. The GNSS network was designed as a closed network with triangles connected from a double base to other points. The location of GNSS stations were carefully selected on site in order to have a maximum visibility of the sky and minimum multipath and obstruction for the satellite signals. Continuous observation of no of satellites tracked was also observed during occupation of the points. A field book was maintained containing information such as project name, station name, date, occupation start time, occupation end time, receiver height at start as well as end of the occupation, receiver model, receiver serial number, and observer name for every GNSS station.

GNSS survey was carried out throughout the hydropower project. Main control point used was UTPKHP-21 of Upper Tamakoshi Hydroelectric Project. A total of 14 major control points were established for the hydropower project itself. The major points are GPS-1, GPS-2, GPS-3, GPS-4, GPS-5, GPS-6, GPS-7, GPS-13, GPS-8, GPS-9, GPS-11, GPS-12, GPS-10 and GPS-20. A general sketch of major GNSS connections is given in the figure below. Other points were established as secondary control points for other facilities of the hydropower project such as roads, camps, quarry areas, soil disposal areas, etc. A total of 22 numbers of such points were established.

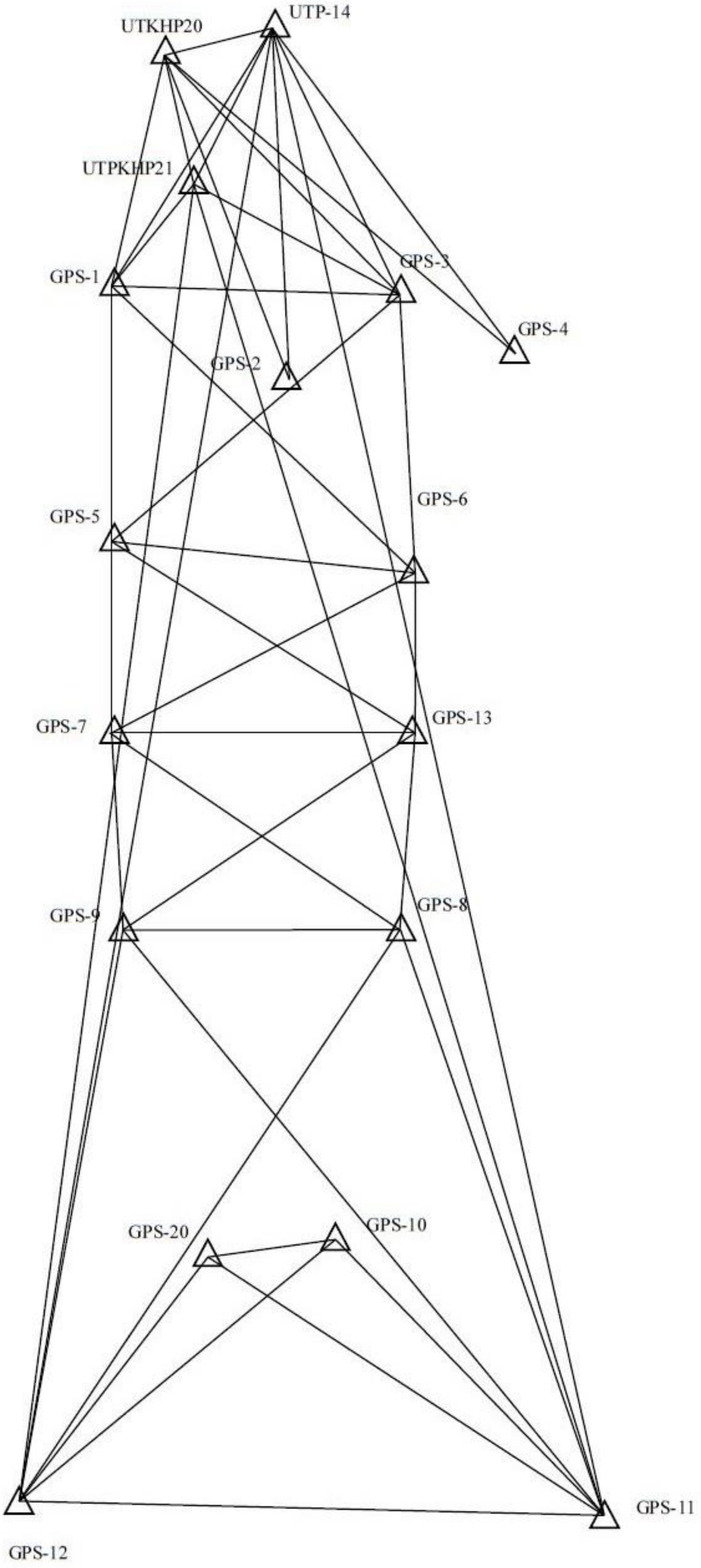

Figure 2-1: General sketch of major GNSS network

2.5.2.2 GNSS Control Points & Ground Coordinates

The complete listings of the GNSS control points and ground coordinates are included in Part F1 of this Detailed Design Report.

2.5.3 Monumentation

The major components of the preliminary design of Tamakoshi V Hydroelectric Project were staked out on the site. The staked locations were marked with a respective monumentation. A total of 14 points were staked for the HRT, Surge Tank, Adit 1, Adit 2, Adit 3, Adit 4, Main Access Tunnel & Test Adit, Spillway and Powerhouse Cavern left & right corners. The list of staked points is presented below.

Staked points were established either on a concrete pillar of L x B x H size 0.15 x 0.15 x 0.40 m or cross chiselled on a boulder or concrete nail cemented on a boulder. Every staked point was marked by red enamel paint on the site. Description cards of all staked points were prepared; the cards are presented in Part F1 of this report.

Table 2-6: List of Staked Points

STN	EASTING (m)	Northing (m)	RL (m)	REMARKS
1	422192.194	3074463.012	1109.062	Adit-3
2	422169.001	3074501.020	1124.204	Adit_3_01
3	423768.870	3079238.056	1177.622	Adit-1
4	423374.991	3076297.488	1169.110	Adit-2
5	423222.002	3076167.504	1205.660	Adit-2-01
6	422871.845	3076299.841	1389.243	HRT-Adit-2
7	420719.381	3071668.709	1313.959	HRT-Adit-4
8	420772.530	3071619.280	1234.450	Adit-4-01
9	420697.070	3071609.870	1272.789	HRT-Surge Tank
10	420773.176	3071539.490	1184.234	Surge Tank Adit
11	420729.290	3071489.173	1162.200	PH Corner Left
12	420693.809	3071524.744	1187.060	PH Corner Right
13	420819.250	3071399.240	1000.250	Access Tunnel Test Adit
14	423943.006	3079065.536	1147.500	Spillway

2.5.4 Closed Traverse Loop Survey

Closed traverse loops were carried out between the GNSS pairs to establish additional horizontal control points for the project. Double sets of data were observed. Both horizontal angles and vertical angles were observed with face right and face left, and the mean angle was used as final angles. Distances were also measured from both right and left faces at back and fore stations and mean distances were used. Traverse were conducted using high accuracy, and recently calibrated survey instruments with a least count of 1” in angles and ± (2mm + 2ppm x D) in distance were employed. Closing errors (ΔE and ΔN) in each traverse loop were calculated and distributed according to Compass (Bowditch) Rule. The accuracy of the traverse loop is defined by 1/(√(ΔE2+ΔN2)/D). Some control points were observed as offset points from traverse station. A total of 1 closed traverse loop were conducted.

Mean height differences from fore and back readings of points were used for calculation of level loops. This process almost eliminates all the errors due to curvature of earth and refraction. Errors of the loops were distributed evenly among the points to calculate the level of all the traverse points.

Only one closed traverse loop was carried out for an access road part between GNSS pair. Other necessary control on the rest of the sites such as other remaining road alignments, Headworks, HRT, Surge Tank, Adit 1, Adit 2, Adit 3, Adit 4, Main Access Tunnel & Test Adit, Spillway & Powerhouse site control points were established by GNSS static method. The accuracy summary of closed loop is shown in the table below. The traverse calculation summary is given in Part F1 of this report.

Table 2-7: Accuracy Summary of Closed Loops

Loop No.	Angular Error	ΔE	ΔN	Traverse Length	Accuracy
Loop - 1	-0.0084	-0.158	0.071	2490.771	14395.604

2.5.5 Documentation

2.5.5.1 Detailed Topographical Survey

Detailed topographical survey was carried out from Shrawan 02, 2074 to Bhadra 17, 2074. The survey was carried out on key components of the hydropower project (intake, headpond, powerhouse, HRT alignment, tailrace alignment, surge tank).

The survey areas were marked and finalized on topographical maps provided by the Government of Nepal in scale 1:25000 prior to the field survey. The detail topographical survey covered a strip of 150 meters on the headrace tunnel alignment and 100m strip of adit tunnel alignments and necessary coverage on the other components of the hydropower project. The list of detail work completed is given in the table below.

Peculiar features of the terrain were surveyed by means of spot surveying. Spot positions were taken by the total station from different GNSS, traverse and offset points. The survey adopted the break line method of survey. Points were taken on the original ground surfaces where the break of slop seemed to appear. Sufficient points were surveyed to represent the existing ground surface to the fullest.

The detail topographical survey depicted following information on ground:

- High Flood Level, Water Level, River Center, River Bank, etc.
- Kulo, Gully, Kholsi, etc.
- boulders, rock, cliffs, land slides, etc.
- trees, etc.

- houses, sheds, temples, Mane, Gumba, etc.
- taps, water tanks, etc.
- roads, bridges, tracks, etc.
- agricultural land, forest, village boundaries.
- transmission line towers, electric poles.

Table 2-8: List of Detail Topographical Survey Works Completed

Details of Topographical Survey	Units	Scale	Survey Qty
Proposed headworks site	ha	1:500	19.7
Proposed Adit 4, Surge Tank, Powerhouse	ha	1:500	8.20
Proposed Switchyard Area	ha	1:500	9.4
Proposed Tunnel Alignment	ha	1:5000	162.50
Proposed Main Access Tunnel & Test Adit	ha	1:500	22.09
Proposed Camp Area	ha	1:500	13.4
Proposed Access Road (Jagat Bridge to Adit 2)	ha	1:500	22.91
Proposed Access Road (Jamune Bridge to Adit 3 & Jamune Bridge to Surge Tank)	ha	1:500	41.80
Transmission Line Alignment	ha	1:5000	41.32
Transmission Line Alignment	km	1:1000	3.347

2.5.5.2 Transmission Line Survey

A preliminary design of the transmission line which was designed in 1:5000 scale was staked out on the site while conducting the 1:1000 scale topographical survey. A total of 30m corridor (15m on each side, left and right) was surveyed along the transmission line alignment. Some points could not be staked out due to inaccessibility of the points in the site, so the inaccessibility point were shifted; they are AP-17-8, AP-17-8A and AP-17-13. The lists of staked points are provided in Part F1 of this report.

2.5.5.3 Camp Area Survey

The detail survey was carried out for a camp area (in the terrace at Suritar) as marked and finalized by Total Management Services Pvt Ltd. (TMS). The detailed topographical survey was covering a respective area. Sufficient points were surveyed to represent the existing ground surface. The survey area quantity of the camp area is provided in the following table.

Table 2-9: List of Detail Topographical Survey Works Completed

Details of Topographical Survey	Units	Scale	Survey Q'ty.
Proposed Adit 1 Portal and Spillway	ha	1:500	9.65
Proposed Headrace Tunnel & Interconnection area	ha	1:5000	132.17

Details of Topographical Survey	Units	Scale	Survey Q'ty.
Proposed Adit 2, Adit 3 & Outlet Structure area	ha	1:500	3.7
Proposed Main Access Tunnel & Test Adit area, Switchyard including camp facilities, Surge Tank and Powerhouse area	ha	1:500	22.09
Proposed Outdoor Area near Outlet Structure	ha	1:5000	14.86
Proposed Access Road (Jagat Bridge to Adit 2)	ha	1:500	22.91
Proposed Access Road (Jamune Bridge to Adit 3 & Jamune Bridge to Surge Tank)	ha	1:500	41.80
Transmission Line Alignment	ha	1:5000	44.69
Transmission Line Alignment	km	1:1000	3.347
Soil Disposal Area	ha	1:500	12.3
Road Alignments	ha	1:500	9.20

2.5.5.4 Spoil Disposal Area Survey

The detail survey was carried out on soil disposal areas as marked and finalized by the Total Management Ser-vices Pvt Ltd. (TMS). The detail topographical survey was covered a respective area. Sufficient points were surveyed to represent the existing ground surface. The survey area quantity of soil disposal areas was provided in the above table.

2.6 Engineering Site Plan Survey

2.6.1 Map Scales and Contour Intervals

The detail mapping of the project sites which includes interconnection system, powerhouse area, surge tank area, MCT zone along the Headrace Tunnel alignment and portal areas were prepared in the scale of 1:500 and 1 m contour interval. Similarly, the mapping of foundation areas is in scale of 1:500 and 1 m level con-tour interval for detail design of surface structures. The mapping of the tunnel alignment strip is done in the scale of 1:5000 and 5 m contour interval.

2.6.2 Mapping Standard

The data acquired by the surveyors were plotted in site itself to check for gaps in survey works. The data was then brought to Kathmandu for final plotting and map preparation. The topographical map was prepared in Auto CAD using SW DTM software. DTM (Digital Terrain Model) was also prepared using SW DTM. All the natural and manmade features like houses, rivers, roads, tracks, kulo etc., were drawn and appropriate symbols were used to show rivers, trees, boulders, cultivated fields, rock, landslides etc. The map was pre-pared with a contour interval of 1 m and major contour at interval of every 5 m. Major contours were marked in appropriate places with heights. Spot heights were also placed in appropriate places for further height information. Further map information such as place names, river names, direction of flow of rivers, way to places were placed on the maps. Legends of the symbols were also prepared. The topographical maps are provided in a separate volume.

2.6.3 Survey Techniques

The detail survey was conducted using advanced total station which measures the angle and distance precisely. Total station instruments measure angles by means of electro-optical scanning of extremely precise digital bar-codes etched on rotating glass cylinders or discs within the instrument and measurement of distance is accomplished with the help Electronic Distance Measurement (EDM) devices fitted inside the telescope to measure the distance accurately.

2.6.4 Map Compilation and Drafting Specifications

The topographical map was prepared in Auto CAD using SW DTM software. DTM (Digital Terrain Model) was also prepared using SW DTM.

2.7 River Cross Section Surveys

River cross sections were surveyed along the axis of Tailrace and upstream and downstream of the Tamakoshi River. The depth of river was measured with rope tied on a weight. The weight used was of 10 kilograms and the rope was carefully marked at a meter interval. For the cross section of the river distance was measured along the water level of the river and chainages were fixed. Every cross section chainage was marked with red enamel paint at the bank of the river. Cross sections were done at 50 m intervals if the location allowed accurate measuring of the cross section at full width of the river properly. Otherwise a suitable location was searched and rope was tied on both banks of the river and rafting boat was moved along the rope and weight was dropped and the height was carefully measured. The location of the actual drop of the weight was measured with a total station setup at the bank of the river.

On the upstream river cross section survey was extend up to 190 m and on the downstream the river cross section was extended up to 755 m from the required 200 m upstream and downstream of tailrace axis due to unavailability of proper cross section sites. The list of cross sections completed is given in the table below.

Table 2-10: List of River Cross Sections - Completed Survey

S. No.	Chainage
1	0+390/US
2	0+335/US
3	0+230/US
4	0+200/US
5	0+150/US
6	0+100/US
7	0+050/US
8	0+000/Tailrace Axis
9	0+050/DS
10	0+100/DS
11	0+150/DS
12	0+200/DS

S. No.	Chainage
13	0+293/DS
14	0+333/DS
15	0+408/DS
16	0+700/DS
17	0+975/DS

2.8 Survey Report

2.8.1 Maps and Description Cards

A complete set of the produced topographical maps and the description cards for the control points is given in Part F1 of this Detailed Engineering Design Report.

2.8.2 Final Control Points

A complete listing of the final coordinates of control points with their locations is given in Part F1 of this Detailed Engineering Design Report.

2.8.3 Accuracy of Survey

The project area is mainly located in between steep cliffs, so reflector less methods were used for the survey.

The following data document the accuracy achieved during the reported survey work.

Table 2-11: Summary of Standard Deviation of Points

POINTS	
Lowest Value of Standard Deviation in Easting (m)	0.002
Lowest Value of Standard Deviation in Northing (m)	0.001
Highest Value of Standard Deviation in Easting (m)	0.015
Highest Value of Standard Deviation in Northing (m)	0.012

Table 2-12: Summary of Precision of Baselines

BASELINE	
Highest Horizontal Precision (m)	0.001
Highest Vertical Precision (m)	0.001
Lowest Horizontal Precision (m)	0.019
Lowest Vertical Precision (m)	0.056
Average Horizontal Precision (m)	0.0043
Average Vertical Precision (m)	0.0096

Further details about the performed survey work can be taken from Part F1 of this Detailed Design Report.

3 HYDROLOGY

3.1 Introduction

The TK-V HEP is a cascade of Upper Tamakoshi Hydroelectric Project (UTK HEP); the TK-V HEP taps the tailrace outlet from UTK HEP. The hydrological analysis of the Tamakoshi-V Hydroelectric Project (TK-V HEP) consisted of collection of relevant documents and secondary data from different sources and conducting basic data analysis related to estimation of values of various hydrological parameters needed for the design of different civil, hydro-mechanical and hydroelectrical components of the Project. This analysis has used the outputs of the hydrological analysis of UTK HEP to the extent considered to be appropriate.

For the design of various components of the TK-V HEP, the long term monthly flow of the Tamakoshi River at the proposed intake (headpond) site and the tailrace site of the Project was analyzed. The flood flow and the construction period flows were estimated based on the instantaneous annual flow values at the reference hydrological station. Further analyses dealt with the generation of the flow duration curve (FDC) at the same sites from the daily and average monthly flows, possible impacts on the estimated flows which are accountable to climate change and the sediment load to be expected. Finally, a forecast was prepared on the energy which will be generated by the TK-V HEP.

The following specific tasks related to the hydrological analysis of the TK-V HEP, specifically mentioned in the Terms of Reference of the engineering consultancy contract.

- Review relating hydrological, meteorological and sedimentological studies by UTK HEP.
- Assess the adequacy of available data and identify the gaps, if any, in the available data.
- Assessment and estimation of long term mean flow of Tamakoshi River as well as other tributaries that will be tapped by UTK HEP for power generation using appropriate methods in appropriate locations.
- Flood frequency analysis for determination of floods at different return periods and calculation of probable maximum flood (PMF) of Tamakoshi River in appropriate locations particularly at the powerhouse site.
- Assessment and estimation of sediment yield of Tamakoshi River using appropriate methods and identification of the needs of sediment management measurements.
- Assessment of possible impact of climate change on hydrological characteristics using different scenarios (without climate change, low climate change and high climate change) drawing from existing literature and data.
- Assessment of meteorological aspects relevant during construction phase, such as length of the rainy season, rainfall characteristics, number and duration of rainfall events, dry interval between rainfall events, temperature etc.
- Assessment and estimation of energy that can be generated by the Project under the assumption of different scenarios concerning the availability of water and scheduled unit outages.

3.2 Description of Tamakoshi V HEP Catchment

3.2.1 Catchment Location

The TK-V HEP catchment lies in the Khare, Orang and Lamabagar rural municipalities (RM) of Dolakha District, just downstream of the Upper Tamakoshi HEP (456 MW), with a significant part of the catchment located in the Tibetan Autonomous Region of the Peoples' Republic of China (PRC), as shown in below figure. The latitude, longitude and altitude of TK-V HEP intake/head-pond site are 27°49'45.7" N, 86°13'15.8" E and 1154 m respectively, located between Purano Jagat and Gongar. The latitude, longitude and altitude of powerhouse site are 27°45'32" N, 86°11'28" E and 982.2 m respectively, located between Jamune and Suridobhan. The headwaters of Tamakoshi originate from an elevation of about 7311 m. The length of main channel from highest point to the headpond is 62.1 km and to tailrace site is about 72.6 km. Average river bed slope of the main channel is 8.7%.

The study of available maps, aerial photographs and Google Earth indicate the presence of permanent and temporary snow covering about 1,148.86 km² (headpond) and within the upper area of the watershed; the area above 5000 m elevation is considered as the area of permanent snow cover. There are 57 glacial lakes in Tamakoshi catchment, including 13 in Rolwaling Catchment. The total catchment area delineated by GIS is 2,139 km² at the intake site and 2,460 km² at the powerhouse site. The catchment area computed by GIS has been adopted for these hydrology and sedimentation studies.

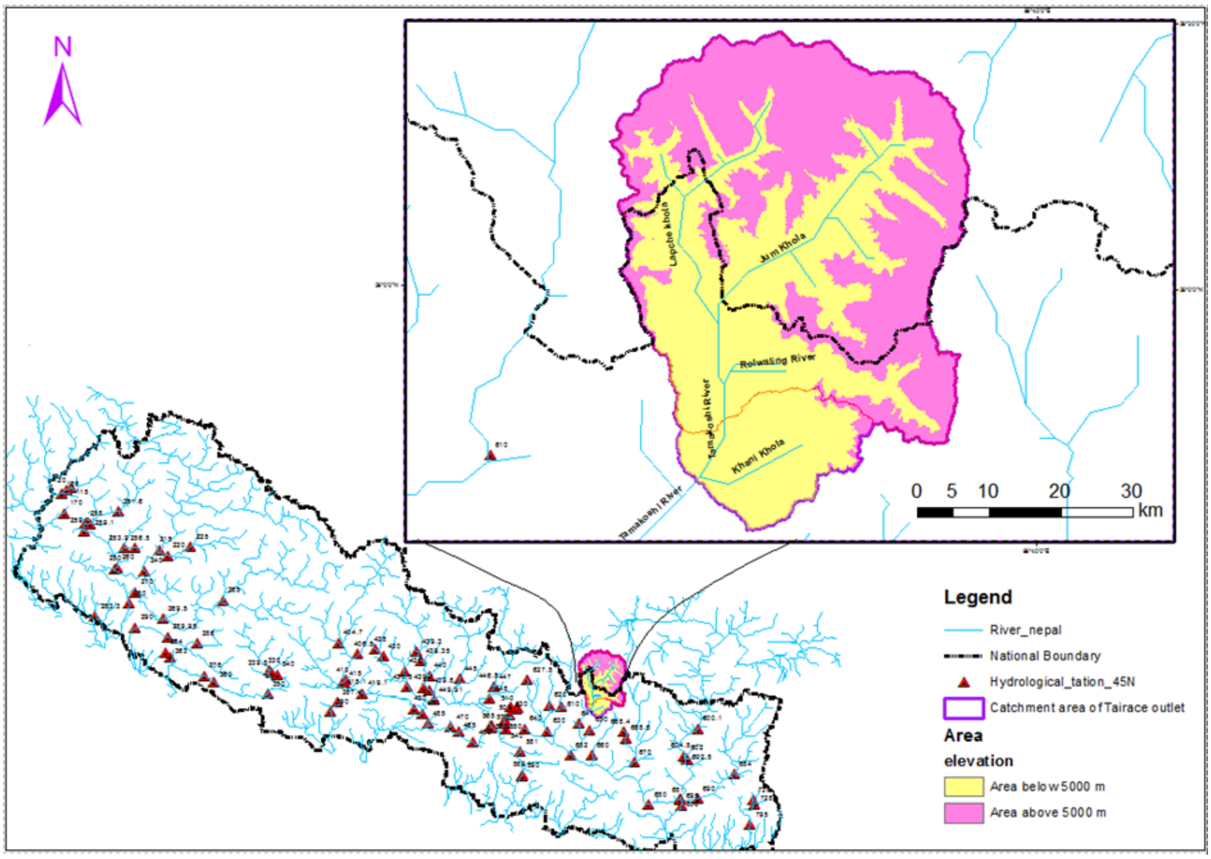

Figure 3-1: Location of the Tamakoshi V Hydroelectric Project catchment

3.2.2 Catchment Delineation and Estimation of Basin Characteristics

The catchment delineation and estimation of the basin characteristics consisted of the downloading and procuring the related digital maps of the basin and the GIS analysis for specific purposes. The comparison of different basin characteristics is used to select the reference station.

A digital elevation model (DEM) of the catchment at the proposed head pond and tailrace was developed from the digital topographic map, procured from the Department of Survey. The comparison of the hypsometric data between a specific catchment and the catchment of the potential reference station provides a basis in determining the best reference station for the hydrological study.

3.2.3 Reference Hydrological Station

Based on the location of different hydrological stations operated by the Department of Hydrology and Meteorology (DHM), the reference hydrological station for the hydrological analysis of TK-V HEP was considered as the Tamakoshi River at Busti (Station Number 647) (The NEA's UTK HEP used Bhotekoshi at Barabise (St. No 610) and Tamakoshi at Busti (St. No. 647) as reference hydrological stations to generate monthly mean flow; ref. NEA Study (Rep 2001), as mentioned in [1]). The St. No. 647 is located very close to the proposed intake site and tailrace outlet site of the Project and the catchment characteristics of St. No. 647 are similar to the catchment of the Tamakoshi River at the intake (head pond) site; the Rolwaling Diversion Scheme of UTK HEP found the daily flow data at Station 647 to be closely related to its data at Lamabagar Station (Station GS2), attesting quality of the data. The total catchment area of the proposed intake site is approximately 2,153 km² and 2,453 km² at the powerhouse site ([3]) whereas the catchment area of St. No. 647 is 2,753 km² (DHM 2004). The Consultant found the catchment area at the head-pond and tailrace sites to be 2,139 km² and 2,460 km² respectively in a recent GIS analysis, which is very close to the area values adopted in previous studies, given the accuracy of the maps; the slight difference between the catchment area values is due to shift of the tailrace site slightly to the downstream location. The catchment area ratio (CAR) of the intake (head-pond) site and powerhouse site, compared to St. No. 647 is 0.78 and 0.89 respectively; these CAR values and the flow ratio values are used as references to transpose the data of St. No. 647 to the proposed intake (head-pond) and powerhouse/tailrace site of the TK-V HEP. The Upper Tamakoshi Hydroelectric Project has established and conducted hydrological and sediment analyses in the areas close to the TK-V HEP; the outputs of these analyses were also considered when determining the final values of the various hydrological parameters of this Project.

3.3 Long-term Flows and Average Monthly Flow (AMF)

The daily flow data at Tamakoshi at Busti (St. No. 647) was collected (1971-2015) from the DHM and the average monthly flow was calculated for each year. The plots of the average monthly flow at St. No. 647 show a consistently decreasing trend in 9 out of 12 months; the average monthly river flow shows an increasing trend only in August and September (Figure 3-2 below), when there is surplus water for power generation. From October to January the slope of the best fit line is slightly negative, indicating a gradual decreasing trend. In February there is no clear trend; the best fit linear trend line is almost flat. There is a gradual decreasing trend in the average monthly flow in the months of March to July. Hence for the medium and long-term prospect a net decrease in the river flow in the dry season can be expected, while the river flow will increase in the wet months.

The analysis of the daily discharge at Tamakoshi at Busti was used to generate average monthly flow at UTK HEP for assessing the water availability at UTK HEP for energy generation, using the simple catchment area ratio (CAR) method; the output of the analysis showed higher values compared to the observed values. The discrepancy between the generated flow and observed flow resulted from the difference in ratio of the snow covered area (considered as area above 5000 m elevation) in the total catchment area. Hence the flow ratio (ratio of flow between two stations in a river for the same time duration) was used to obtain average monthly flow at the Lamabagar dam site of UTK HEP. Similarly, the average monthly flow at the Bhainse Khola and Rolwaling River were generated using the flow ratios.

The below table presents the Summary of the average monthly flow (in m³/s) at the St. No. 647, the Tamakoshi at Lamabagar (UTK HEP Damsite), Bhainse Khola, Rolwaling River and the proposed Intake Site of TK-V HEP Project.

Table 3-1: Long-term Average Monthly Flow at different locations related to the Tamakoshi V HEP (based on DHM St. 647 data of 1971-2015 and flow ratios of rivers at different locations)

	Average Monthly Flow (in m^3/s)							
Month	St. No. 647	Tamakoshi at Lama-bagar	Bhainse Khola	Tamakoshi at Lamabagar + Bhainse	Rolwaling Khola	Tamakoshi at Lamabagar + Bhainse + Rolwaling	TK-V Head-pond Site	TK-V Tail-race Site
Jan	29.5	14.6	0.41	15.04	3.2	18.28	17.9	20.6
Feb	25.3	12.9	0.32	13.23	3.1	16.36	15.8	18.2
Mar	24.4	12.5	0.28	12.80	3.2	16.03	15.3	17.7
Apr	28.2	15.6	0.25	15.83	5.3	21.09	19.1	22.0
May	52.5	31.1	0.33	31.41	12.2	43.61	38.1	43.8
Jun	164.1	79.6	1.11	80.72	34.1	114.80	97.5	112.2
Jul	427.7	166.2	3.56	169.72	61.7	231.46	203.6	234.2
Aug	496.0	177.4	4.09	181.46	57.7	239.17	217.3	250.0
Sep	311.2	117.2	3.38	120.57	39.1	159.67	143.6	165.2
Oct	123.6	54.0	1.79	55.82	16.9	72.76	66.2	76.2
Nov	59.0	26.3	0.79	27.11	6.4	33.50	32.3	37.1
Dec	39.6	18.1	0.49	18.55	4.2	22.72	22.1	25.5

The calculation of the average monthly river flow at the head-pond site of the Tamakoshi V HEP does not include the flow diversion to UTK HEP from the Bhaise Khola and Rolwaling Khola. Once the flows from these two Kholas are diverted to UTK HEP dam site for energy generation, the river flow at the head-pond site will decrease accordingly, especially in the dry season.

As shown in above table, the monsoon flow is dominant in the Tamakoshi River. Out of an annual flow of 1,778.7 m³/s; the sum of the monsoon flow (June-September) is 1,395.8 m³/s, which is close to 78.5%. The lowest average monthly flow occurs in the month of March, which is normal in Nepalese rivers with a significant portion of the catchment area located in the higher Himalayas and covered by snow most of the time. As the temperature begins to increase in April, the snowmelt component of the river flow begins to increase and river flow increases as well.

However, due to the declining trend in the average monthly flow in Tamakoshi River, the long-term flow for the duration of the project design life should be evaluated accordingly. A recent NEA study of Tamakoshi River flow for the Rolwaling Diversion Tunnel Scheme estimated monthly average flow available for power generation at lower values.

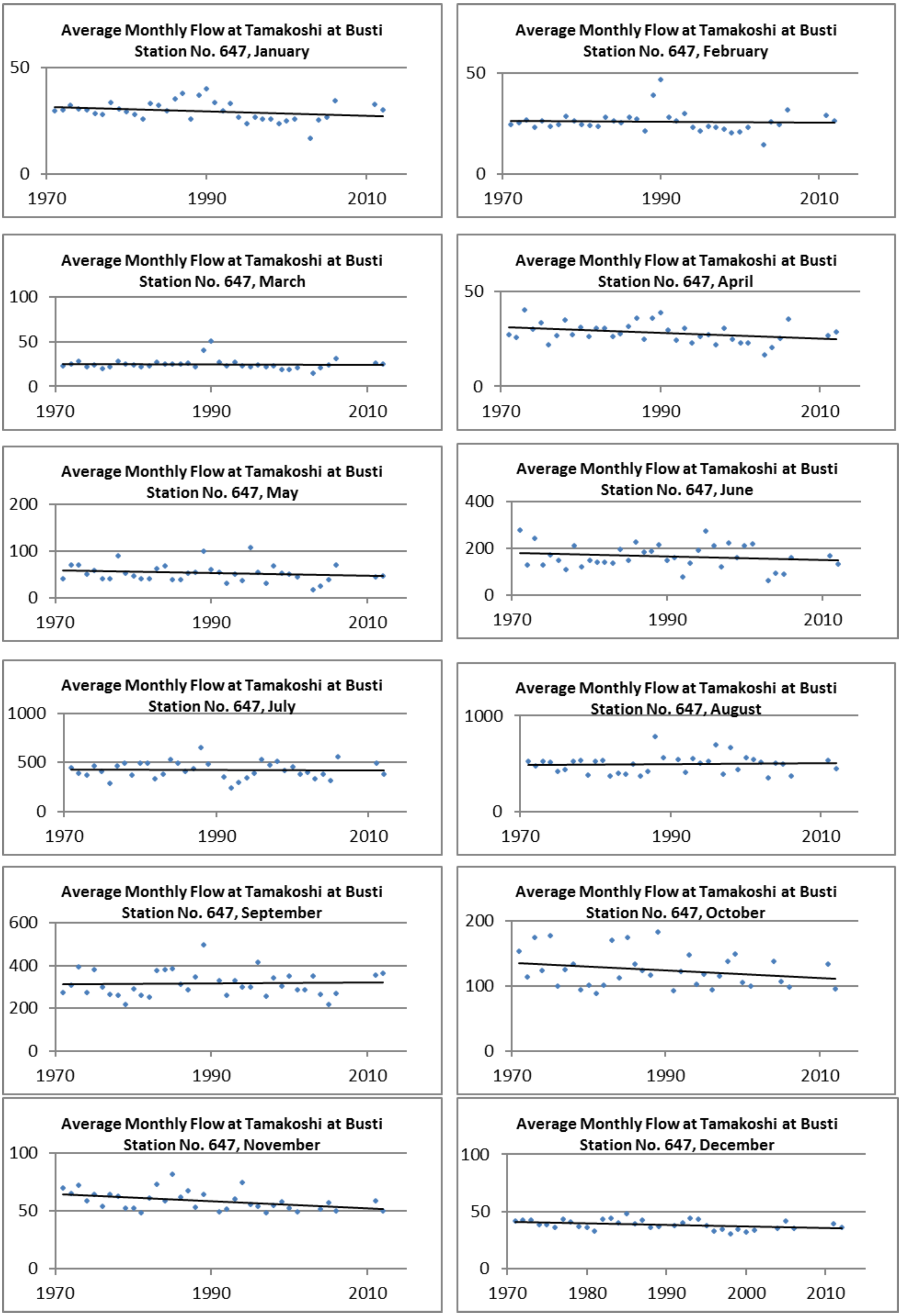

Figure 3-2: Average Monthly Discharge (m^3/s) at Tamakoshi River at Busti

3.4 Flood Flow Analysis

3.4.1 Statistical Analysis

The flood flow analysis at the proposed headworks site of TK-V Project is carried out based on 38 years of annual instantaneous annual flood discharge value at Tamakoshi River at Busti (St. No. 647), collected from the DHM.

Five different flood frequency analysis methods, namely the Gumbel, Extreme Value I, Log-Pearson Type III, Log-Normal, and Weibull Plotting Position, were used to estimate the flood flow value of different return periods at St. No. 647 of Tamakoshi River at Busti; the Weibull Plotting Position method plot of reduced variate (yT) versus flood discharge is given in below figure, where yT = - [ln {ln (T/(T-a))}] and T = return periods (in years). The results from the five different methods were averaged, after eliminating the highest and lowest value for each return period, and transposed to the proposed tailrace site of TK-V HEP, using CAR method with exponent value of 0.5.

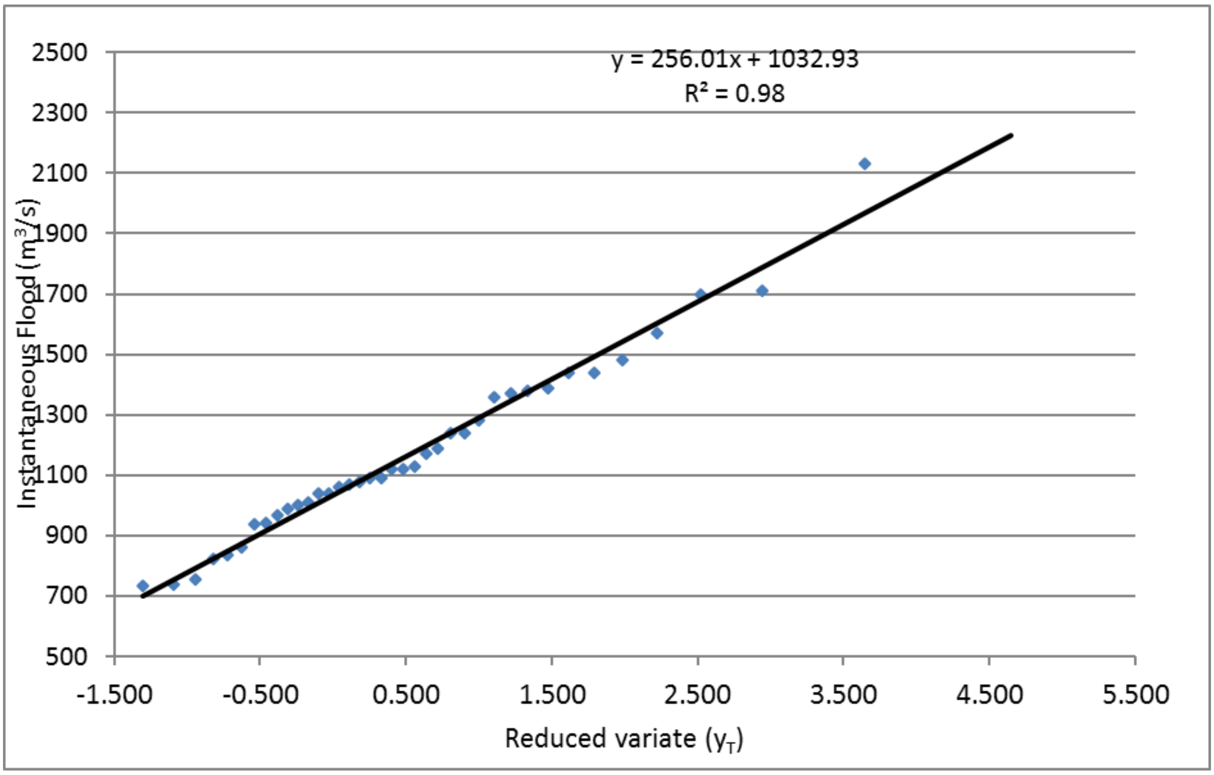

Figure 3-3: Weibull plot of instantaneous flood versus reduced variate yT, for Tamakoshi at Busti

The above figure indicates a reasonably stable relation between reduced variate (yT), which is related to the return period (T) by the relation yT = - { ln (ln [T / (T – 1])}, and the instantaneous annual flood (Qp).

The flood value at a hydrological station can be transposed to a different location, depending on the hydrological similarity of the two catchments. Since the catchment of UTK HEP and the Tamakoshi at Busti is basically the same catchment with more than 60% area shared by two catchments, the flood values of different return periods at the St. No. 647 was transposed to the flood values at UTK HEP dam site and Tamakoshi V HEP headpond and tailrace sites (see table below), using the catchment area ratio (CAR), with the ratio of the catchment area located below 3000 m elevation; the Water and Energy Commission Secretariat (WECS, 1990) found that the most relevant parameter associated with flood values in Nepalese rivers is the area located below 3000 m elevation. The precipitation in the area located above the elevation of 3000 meter did not show significant correlation with the flood values; the precipitation above the elevation of 5000 m is mostly in the form of snow and hence does not contribute directly to the flood values in the downstream stretch of the river. The total areas under 3000 m elevation at the St. No. 647, UTK HEP dam site, TK-V HEP Headpond and TK-V Tailrace are 169.556 km², 17.372 km², 49.18 km² and 96.88 km², respectively.

Table 3-2: Instantaneous flood values (in m³/s) of different return periods at UTK HEP dam site, TK-V head-pond and TK-V tailrace site

	Tamakoshi River at UTK HEP Dam Site, TK-V Head Pond and TK-V Tailrace		
Return Period, years	**UTK HEP Dam Site**	**TK-V Head Pond**	**TK-V Tailrace**
2	360	606	851
5	448	755	1059
10	505	850	1192
20	560	942	1323
50	631	1062	1491
100	685	1153	1618
500	810	1363	1914
1000	865	1455	2042
5000	993	1671	2345
10000	1049	1766	2478

As shown in above table, the instantaneous annual flood discharge for a return period of 1000 years at UTK HEP dam site is 865 m³/s. The Feasibility Study of UTK HEP [4] listed the 1000 years return period at the dam site as 875 m³/s. The difference between these two values, which is approximately 1%, is due to the use of additional data of later years (2007-2012) in the current analysis and the use of averaged flood value from the five methods against the use of a single method.

3.4.2 Probable Maximum Flood

Theoretically, a probable maximum flood (PMF) is associated with the frequency of the occurrence of a probable maximum precipitation (PMP), which does not have any specific return period. Review of literatures indicates a very wide range of return periods used to estimate PMP, anywhere from 1000 years to 100000 years. A recent study [5] indicates that in order to arrive at a rational decision on the frequency of PMF, the various components associated with the PMF needs to be decomposed, the frequency of each components needs to be evaluated, and the final frequency for the PMF needs to be determined based on combined frequency of each component. The assignment of the frequency of each of the component itself is subjective, depending, in part, on fund availability and an acceptable degree of socio-economic risk. Since the site specific data associated with each component of the PMF for the TK-V Project is unavailable, depending on the available data, a plot of the flood magnitude versus return period was made (see the following figure) for the Station 647, and a return period of 10000 year was chosen as a basis to determine the magnitude of the PMF. As noted in the Feasibility Study Report of UTK HEP, the WECS/DHM 1990 study recommended using twice the value of 10000 year return period as the PMF. As shown in the figure below, the 10,000 years return period value for the Station 647 is 3,278 m³/s and the transposed value of the same return period at UTK HEP dam site, and the head-pond and tailrace site are 1,049 m³/s, 1,766 m³/s and 2,478 m³/s, respectively. Therefore the recommended PMF value at UTK HEP dam site, and the head-pond and tailrace site are 2,098 m³/s, 3,532 m³/s, and 4,956 m³/s, respectively. The Feasibility Study Report of UTK HEP recommended the PMF at UTK HEP dam site to be 2,300 m³/s; given the breaks in the data, the difference between these two values are within the range of errors.

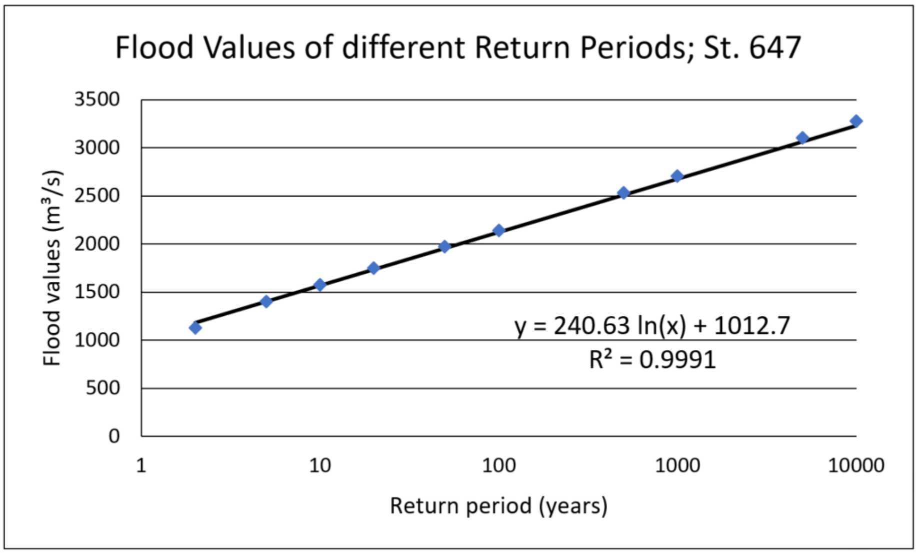

Figure 3-4: Plot of Return Period versus Flood Values to determine PMF

In the event of a worst case scenario of the PMF occurring simultaneously with the GLOF from the Tsho Rolpa Glacier, the instantaneous flood value magnitude can increase by around 3,500 m³/s, which will result in a combined flood value of around 7,000 m³/s at the head-pond section of the Tamakoshi V HEP. The financially burden to design and construct all the structures of a hydropower project for such a hypothetical event will be too high, and hence not recommended.

3.4.3 Construction Period Flood

The construction period extreme hydro-meteorological events, like a high flood, were assessed by identifying the specific months when the record of maximum daily river flow was highest. The averaged maximum daily river flow data of Tamakoshi River at Busti (St. No. 647) showed that the river flow is appreciably high in the months of June to September. Considering June to September as non-construction period, the maximum daily river flow of each year in the months between October to May, inclusive, was collated, and the frequency analysis methods were used to find the probable high flood values during the construction period.

Just like for the annual maximum flood flow analysis, five different flood frequency analysis methods, namely the Gumbel, Extreme Value I, Log-Pearson Type III, Log-Normal, and Weibull Plotting Position, were used to estimate the construction period flood flow value of different return periods at St. No. 647 of Tamakoshi River at Busti; the Weibull Plotting Position method plot of reduced variate (yT) versus flood discharge is given in the figure below. The results from the five different methods were averaged, after eliminating the highest and lowest value for each return period, and transposed to the proposed tailrace site of TK-V HEP, using CAR method with exponent value of 0.5.

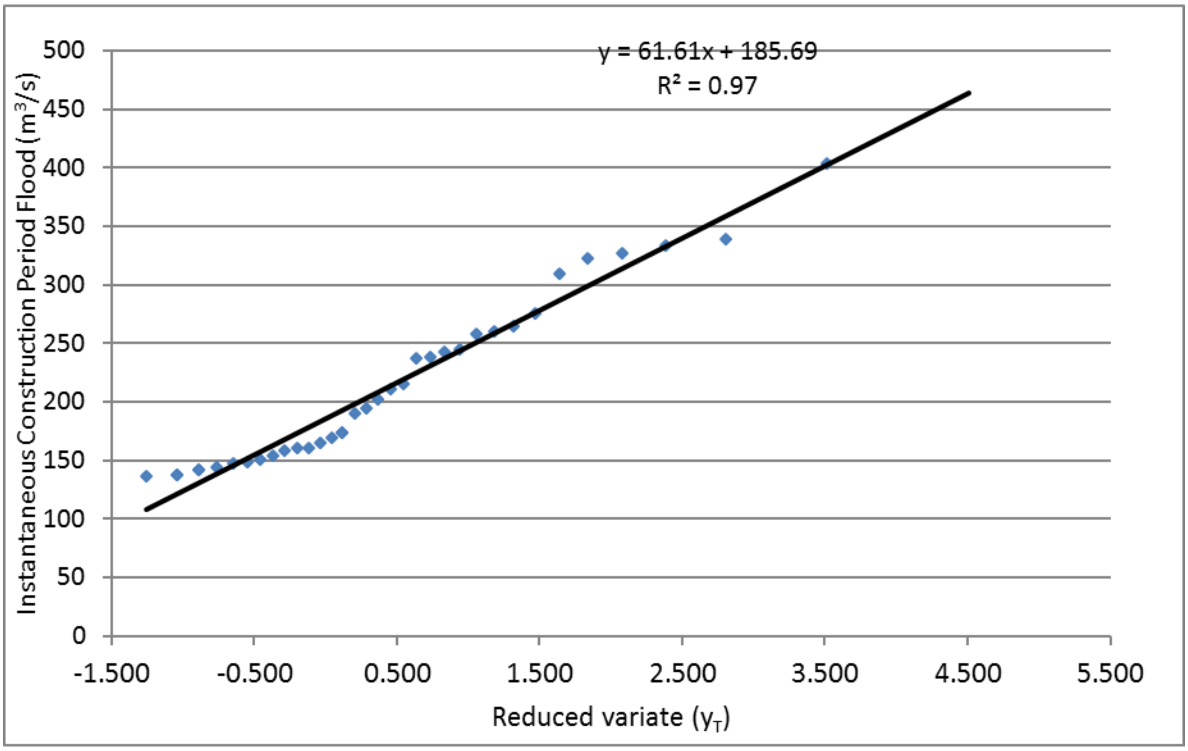

Figure 3-5: Weibull Method plot of construction period flood at Tamakoshi at Busti (St. No. 647)

The construction period flood at the Tamakoshi at Busti was used to transpose the values at UTK HEP dam site and at the head-pond and tailrace sites at the Tamakoshi V HEP (see table below).

Table 3-3: Construction period flood values (in m³/s) of different return periods at UTK HEP dam site, TK-V head-pond and TK-V tailrace site

	Tamakoshi River at UTK HEP Dam Site, TK-V Head Pond and TK-V Tailrace		
Return Period, years	**UTK HEP Dam Site**	**TK-V Head Pond**	**TK-V Tailrace**
2	66.6	112.0	157.2
5	87.4	147.0	206.4
10	101.5	170.7	239.6
20	115.6	194.4	272.9
50	120.0	202.0	283.5

Based on the analysis of the construction period flood data, and considering construction period to be 5 years, the risk of occurrence of the flood of that magnitude occurring within the construction period associated with using 50 years return period as construction period flood is 9.6%, which is considered acceptable.

3.5 Potential Impact of Climate Change

3.5.1 General

The potential impact of climate change in various hydro-meteorological parameters related to the design of different components of the TK-V HEP is assessed based on secondary information. Since the TK-V HEP is located in the Dolakha District, the results of the climate change analysis conducted as a part of two previous studies ([6], [7]) are utilized to assess the potential climate change impacts.

3.5.2 Temperature

Based on the time series data of air temperature of the Charikot Station (St. No. 1102) of Dolakha District, a climate change model was run to assess potential changes in the air temperature. Both the minimum and maximum air temperature in the catchment of the TK-V HEP is expected to increase in summer and winter months, as given in the following table and illustrated graphically in the figure further below.

Table 3-4: Expected changes in temperature (2015-2060 AD)

Month	Change in Min^m Monthly Temp °C (2060)	Change in Max^m Monthly Temp °C (2060)
Jan	+2.1	+2.6
Feb	+3.5	+4.0
Mar	+2.8	+3.2
Apr	+2.1	+0.7
May	+1.0	+1.6
Jun	+1.5	+1.5
Jul	+1.5	+1.5
Aug	+1.7	+1.8
Sep	+1.9	+1.8
Oct	+2.7	+1.5
Nov	+3.1	+1.2
Dec	+2.7	+1.9

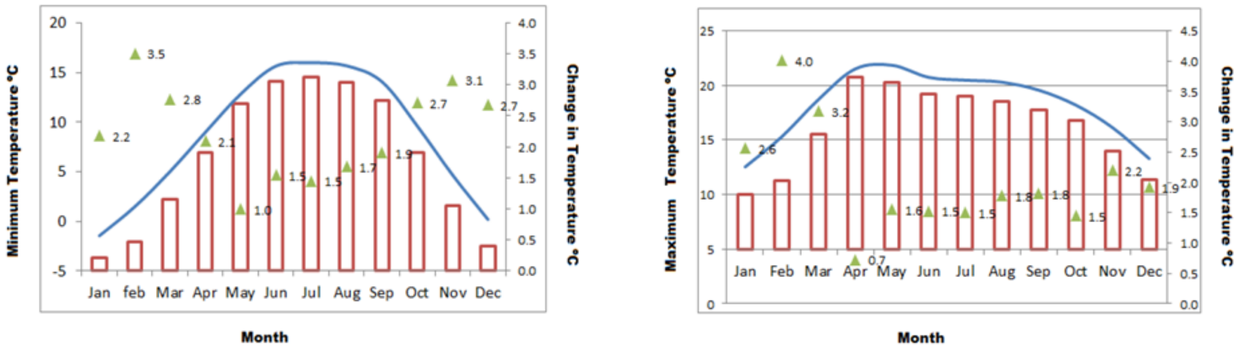

Figure 3-6: Expected changes in minimum and maximum air temperature from 2015 to 2060 AD

3.5.3 Climate Change Impact on GLOF

Based on two previous studies ([8], [9]), the peak flow at the Tamakoshi River upstream of UTK HEP dam during a hypothetical case of Glacial Lake Outburst Flood (GLOF) resulting from a breach of Tsho Rolpa Glacial Lake moraine dam is approximately 7,000 m³/s.; the flood wave resulting from such an incidence of a GLOF is expected to attenuate more than 20 km upstream of the Rolwaling-Tamakoshi confluence, and will not affect UTK HEP dam. The location of the Tsho Rolpa Glacial Lake, the specific distances from the Lake towards UTK HEP dam at which the flood values are calculated is shown in Figure 3-7 below. The result of the dam break simulation of the Tsho Rolpa Glacial Lake is presented in Figures 3-8 and 3-9.

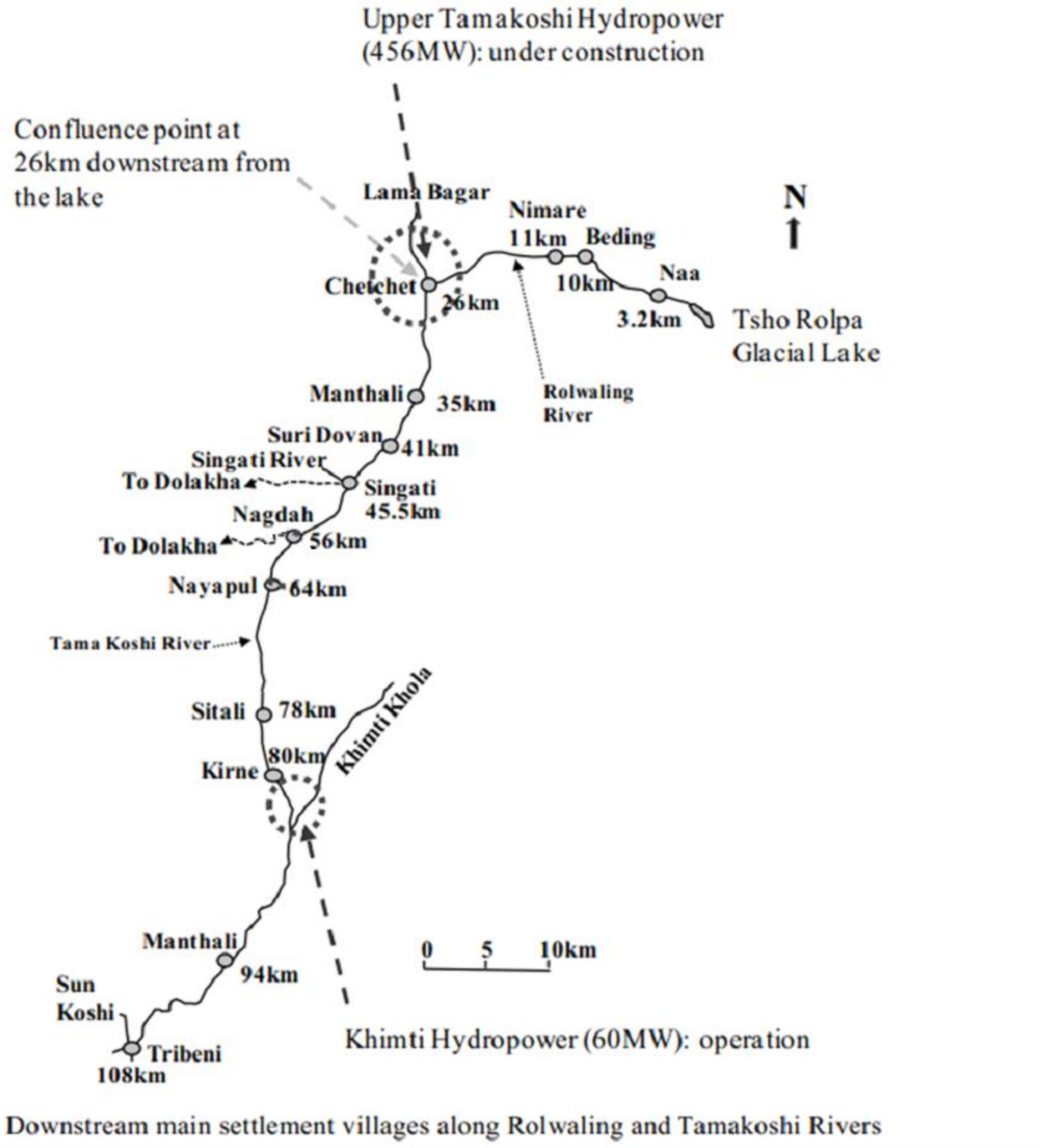

Figure 3-7: Distance from the Tsho Rolpa Lake towards UTK HEP Dam at which flood values for GLOF simulation is calculated (Figure source: Shrestha et al., 2011 [10])

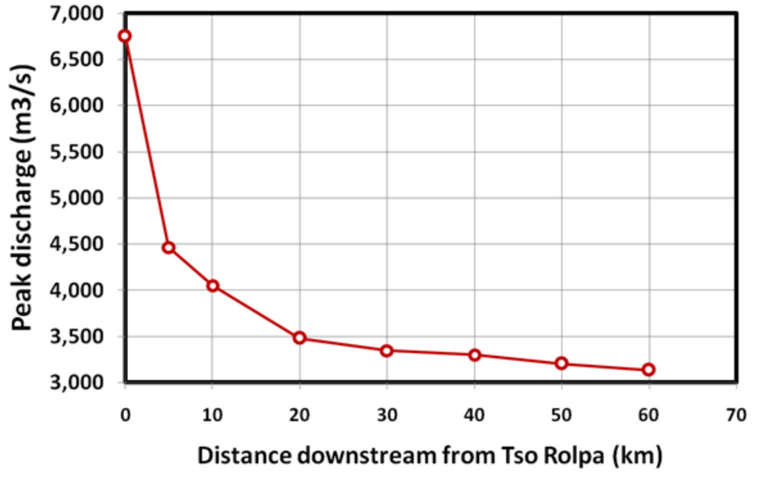

Figure 3-8: Distance versus flood value resulting from GLOF simulation of Tsho Rolpa Lake

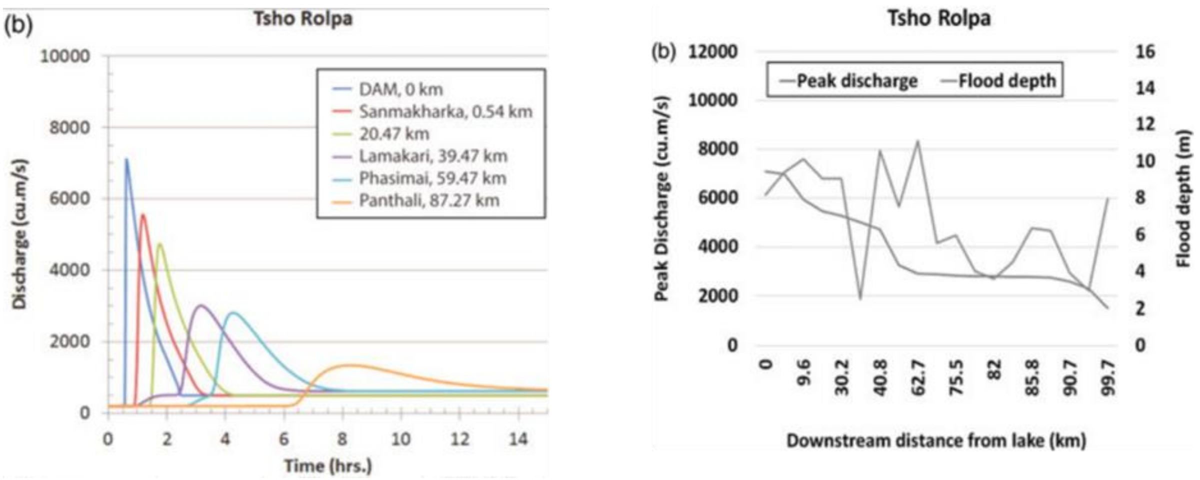

Figure 3-9: Time vs. Distance and Distance vs. Peak Discharge plots of Tsho Rolpa GLOF (Source: Khanal et al., 2015)

The results of another study (Khanal et al., 2015) on potential effect of Tsho Rolpa GLOF matches with the MoSTE study results; both show an initial peak flood of around 7,000 m³/s at the starting location of the GLOF, which recedes to about 3,100 m³/s by the time the flood wave reaches about 60 kms downstream (Figure 3-8). A study conducted by ICIMOD after the April-May 2015 earthquake ([11]) showed no evidence of additional risk of GLOF from the glacial lakes of Tamakoshi River Basin, including the Tsho Rolpa Glacial Lake. However, a regular vigilance is required for the safe operation of UTK HEP and the TK-V HEP; the implications of climate change for GLOFs are of particular concern in the high Himalaya districts of Dolakha, a detailed climate and hydrological modeling suggests that the high elevation areas of Dolakha District will experience major increases in temperature and precipitation by 2050 (MoSTE, 2014a; MoSTE 2014b [12]):

- An increase in the days of the year where the maximum temperature exceeds 0ºC from 50 to 70 days;
- An increase in the average maximum temperature in the dry season of up to 3.5 ºC;
- An increase in wet season mean monthly precipitation by up to 180 mm

The results of another study (Shrestha et al, 2011) of the potential effect of the Tsho Rolpa Glacier Lake Outburst Flood also indicated that the magnitude of the flood is under to 6,000 m³/s in the initial few seconds of the event and gradually decreases as the time passes. By the time the GLOF wave reaches the Rolwaling-Tamakoshi confluence, the flood magnitude will be much less. Hence all the three studies agree that the maximum flood values from the Tsho Rolpa GLOF is under 7,000 m³/s in the location immediately downstream of the lake and will attenuate to a value of alightly above 3,000 m³/s after flowing for 30 to 40 kilometers downstream. The maximum water volume in the Tsho Rolpa Glacial Lake is estimated to be about 80 million cubic meter in 2008; after the installation of the discharge mechanism in the lake the water level and declined and the stored water volume was estimated to be 70 million cubic meter in 2009 (Shrestha et al., 2011).

3.5.4 Climate Change Impact on Riverine Flood

The increase in rainfall intensity can result in an increase in flood magnitude due to decrease in infiltration. Similarly, the increase in air temperature can result in increased snow melt component in the flood magnitude. Moreover, the increase in air temperature can trigger GLOF events. If all the three events occur simultaneously, a much larger magnitude flood can occur in the Tamakoshi River and its tributaries. The below table shows the expected increase in flood value for same return period at Tamakoshi River at Busti. The instantaneous discharge at the Tamakoshi River at Busti is expected to rise due to the impact of climate change ([13]).

Table 3-5: Result of climate change impact study on instantaneous flood at Tamakoshi at Busti

Return Period	Design Flood Baseline (m^3/s)	Design Flood 2040(m^3/s)	Design Flood 2060 (m^3/s)	%Δ 2040	%Δ 2060
2	799	1125	1254	+41	+57
5	1080	1474	1697	+36	+57
10	1280	1711	1998	+34	+56
25	1589	2019	2388	+27	+50
50	1825	2254	2683	+24	+47
100	2090	2492	2983	+19	+43
200	2350	2735	3289	+16	+40

Source: PPCR 3 – Mainstreaming CC Risk Management in Development, Dolakha Threat Profile, MoPE, 2014

As shown in the above table, the instantaneous flood magnitude of 200 year return period can increase by up to 40% in 2060 compared to the baseline value in 2015. The table also shows that as the return period increases the percentage increase in the flood magnitude decreases. Any structures exposed to the flood events should be designed to withstand flood magnitude resulting from the effects of the climate change.

3.6 Sediment Sampling and Analysis

3.6.1 Background

Sedimentation study is an integral part of the overall hydrological study of any water resource related project. However, despite the failure of several water resource projects due to sediment related problems, the number of hydrometric stations that collects regular suspended sediment data in Nepalese rivers is still low; the data on bed load is rare. The oldest sediment data collected in Nepal for the purpose of hydropower development is from the Karnali River beginning 1963. The regular sediment monitoring at the Marsyangdi Hydropower project started in 1989. Other hydropower projects like the Jhimruk and Khimti are also monitoring the river sediment and its effect on wear and tear of various electro-mechanical components. However, the DHM monitors sediment data from only from a few rivers in Nepal.

The annual sediment loads carried by Nepalese rivers are considerably higher compared to the similar basins in other countries. Review of the sediment yield information available for the Himalayan catchments indicates an extremely wide range of erosion rates and sediment yields among basins. The results of the previous studies show that the sediment load is very high in the rivers in the Lesser Himalayas of Nepal. The specific sediment yield (SSY) rates in the High Himalayas, the upper part of Lesser Himalaya (Lesser Himalayas 1) and the lower part of Lesser Himalaya (Lesser Himalayas 2) are 1000, 3000 and 5000 t/km2/year respectively (WECS, 2003).

The parts of Tamakoshi River catchment are located in Tibet Autonomous Region of the People's Republic of China (PRC), High Himalaya, Lesser Himalaya 1 and Lesser Himalaya 2. Based on the average sediment production by Tamor River in eastern Nepal, the Chatara Research Center estimated the SSY of 8200 t/km2/year (Laban, 1978). Even these high SSY values are considered “too low” for Nepalese rivers; the actual yields are estimated to be 2 to 5 times higher than shown by the estimates of the previous studies (WECS, 1987).

Proper sediment management plays a vital role in the success of any hydropower project in Nepal. Since the TK-V HEP taps the water directly from the tailrace of the Upper Tamakoshi HEP (UTK HEP), the fluvial sediment characteristics of the Tamakoshi River do not directly affect the sediment concentration in the intake water of the TK-V HEP. However, the sediments at the tailrace can affect the design of the structures at this site, and as such, the river sediment in the Tamakoshi River at the tailrace was analyzed based on the outputs of the previous studies, result of the sediment monitoring conducted by the consultant as a part of this project, beginning from 1 June 2018, and linked with the river flow at the tailrace site. For the Tamakoshi River near the intake site of TK-V HEP the Consultant made the best effort to estimate the sediment load based on the analysis of available primary and secondary data and information.

3.6.2 Catchment Characteristics

The catchment area of the Tamakoshi River up to the intake site of the TK-V HEP is 2139 sq. km. The Tamakoshi River originates from the Tibetan platue and meets the Sun Koshi River just to the upstream of the DHM Station 652 (Sunkoshi at Khurkot) after traversing southwards through the Gaurishankar Conservation Area, from its origin. Catchments to its east and west drain into Dudh Koshi and Indrawati Rivers which, like the Tamakoshi River, are the major tributaries of the Sun Koshi River.

The Tamakoshi River is a typical Himalayan river with a gravel-boulder bed and relatively steep slope (more than 3% in the head-pond site of the TK-V HEP). The River has a potential to carry large amounts of suspended sediment and bed loads during floods. The fine sediments are derived from land erosion and the coarse material from erosion of river benches and riverbed. Small scale landslides, mostly on the risers of irrigated terraces, are very frequent in the Tamakoshi River Basin.

The Tamakoshi River catchment lies within the zones of the High Himalayas, and the Lesser Himalayas 1 and Lesser Himalayas 2. The hypsometric and land use details are given in the hydrology section of the report. Out of the total basin area of 2139 km2 at the intake, 1149 km2 lies at an elevation of more than 5000 m and hence is considered as permanent covered area. Only about 43 km2 of the catchment is below an elevation of 3000 m with barely any cultivation and human habitat but covered with forest and rock outcrops.

The High Mountain (approximately 3,500 to 5,000 m asl) physiographic zone is characterized by glaciated valleys and geologic formation of highly metamorphic phyllite, schists, gneiss and quartzite. These rocks are resistant to weathering so the soils here are generally shallow and coarse textured, and the suspended sediment rate comparatively low. The mineralogical composition analysis of the water samples from the Tamakoshi River collected in June-September 2018 consists mostly of quartz, feldspar and mica.

The intake dam site of UTK HEP is located in the middle and stable reach of the Tamakoshi River. Down cutting tributaries upstream bring sand and gravel into the Tamakoshi River which are easily transported downstream by high velocities due to the steep gradient of the River. However, the amount of finer materials seen in the riverbed in the vicinity of the Project site during the field visits is very low. The highly metamorphosed rocks (phyllite, schist, gneiss and quartzite) predominate the proposed head pond site of the TK-V HEP. These rocks are generally resistant to weathering. From the erosion point of view, this site has been stable in the past. With the recent expansion of agriculture in these areas, forest lands are degrading and which can result in increased erosion.

3.6.3 Sediment Yield

In Nepalese rivers, there is considerable seasonal variation in sediment transport rates, which is linked to the rainfall pattern, snow melting and weathering. The bulk of total transport, and the worst period occur during the monsoon months. The higher concentrations of the sediments in these months must be taken into account and there is a need to take measurements during high flows. However, at the current level of technology in Nepal, due to the extremely high velocity of flow in the rivers in the monsoon months, presence of floating debris on the water surface and rolling boulders in the river bed, it is very difficult to get accurate data of the river discharge and sediment concentration.

The suspended sediment concentration and the corresponding discharge of the Tamakoshi River were monitored by the NEA, starting from 1 July 2003 and ending on 30 September 2004 (15 months), as a part of the Upper Tamakoshi Hydro Electric Project feasibility study for one hydrological cycle, at G2 and G3 stations located in Lamabagar (The Feasibility Study of Tamakoshi Hydroelectric Project, Volume 1, Main Report, NEA, 2011, mentions the starting date of sediment monitoring at G2 as 16 July 2001. The same report also mentions, in Section 3.12.4, that “the data series from July 2001 through June 2004 is found to be vulnerable to error, the available data is not sufficient to make a reliable sediment yield estimate”.). The sediment concentration and mineralogical analyses were conducted at the SRCL of NEA and the HydroLab Pvt. Ltd. The Norconsult conducted an initial analysis of the NEA’s 2003-2004 data in 2005, which was further refined in another study for the Rolwaling Diversion project in 2016. The average monthly measured suspended sediment concentration at the monitoring site G2 is given in the following table and figure.

Table 3-6: Average monthly measured suspended sediment concentration (ppm) at Tamakoshi River (2003-2004)

Year	2003						2004								
Month	07	08	09	10	11	12	1	2	3	4	5	6	7	8	9
Sedi. conc. (ppm)	108.95	93.65	40.69	3.24	0.43	0.32	0.13	0.17	0.13	1.32	3.25	2.22	21.28	37.56	31.05

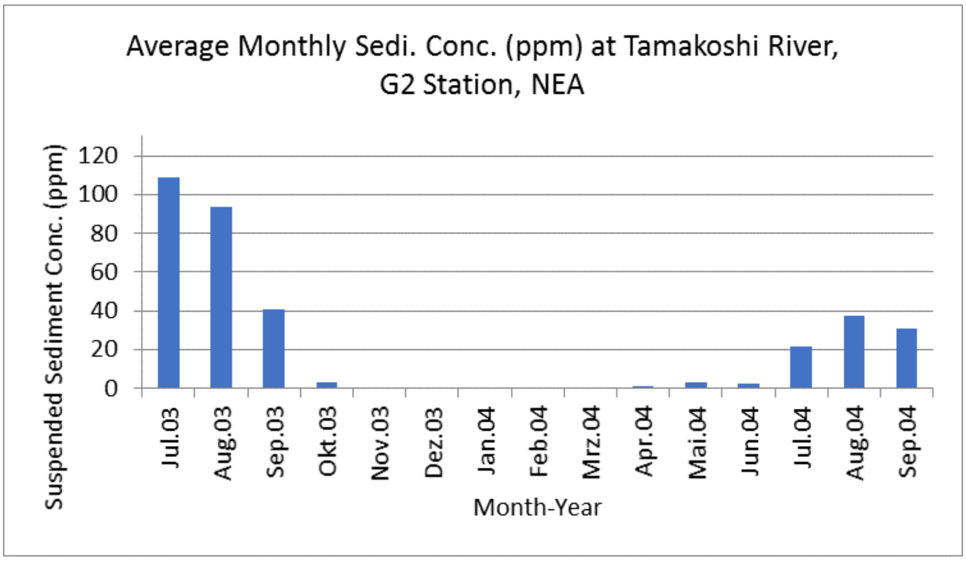

Figure 3-10: Average monthly sediment concentration of Tamakoshi River at G2 station, NEA (2003-2004)

3.6.4 Latest Sediment Monitoring at the Tamakoshi River

Due to the high degree of uncertainties in the available sediment data, as a part of its study, the Consultant sampled the river water of the Tamakoshi River on a daily basis for suspended sediments at the location just below the UTKHEP dam, from 1 June to 30 September 2018 to have a better quantitative and qualitative estimate of the fluvial sediment. The laboratory analyses of the collected water samples were conducted at the Hydromet Consultant Pvt. Ltd. for concentration and at the HydroLab for the particle size distribution, mineralogical composition and organic composition. Since the measurements were taken only in the monsoon season, it does not represent the overall annual cycle of the river sediment. Table 5 shows measured suspended sediment concentration in Tamakoshi River in the months of June to September 2018. The data show an unusually high sediment concentration from August 5 to 15 (11 days); however, the discharge data during these 11 days do not show any abnormality. A statistical test, using the outliers as the values beyond 1.5 times the interquartile range, i.e., the range between the first and third quartile, indicated these data as outliers. These extremely high sediment concentrations can result from anthropogenic activities, like construction activities at the UTKHEP or natural event like a landslide. Since no records of any landslide events were noted during 5-15 August 2018, the sudden and unexpected rise in the sediment concentration is attributed to anthropogenic activities, and hence does not represent natural phenomenon. Therefore, the utility of these data is limited. Nevertheless, a tentative notion of sediment concentration during high flows can be obtained from it. The results of the suspended sediment sampling analysis are presented in the table below.

Table 3-7: Suspended sediment concentration in the Tamakoshi River, June-September 2018

Sampling Date	Month and Concentration (ppm)			
	June	July	August	September
1	4.3	252.2	462.2	431.0
2	14.0	143.0	498.0	407.0
3	26.9	1,427.5	347.5	523.3
4	29.0	901.3	382.5	406.7
5	66.0	1,192.5	8,692.0	381.1
6	67.7	395.6	9,850.0	411.0
7	156.3	951.1	7,578.9	522.0
8	363.4	937.8	6,878.1	380.9
9	74.4	1,088.9	4,237.8	294.4
10	76.2	767.8	3,040.0	246.7
11	247.5	670.0	2,167.0	284.0
12	215.6	1,293.7	1,980.0	242.0
13	133.3	1,681.1	2,455.6	166.0
14	103.3	1,088.9	2,798.8	644.0
15	141.0	571.1	2,448.8	254.4
16	56.7	841.0	1,511.1	232.2
17	48.9	646.3	1,831.3	164.0
18	43.8	464.0	1,354.0	159.0
19	246.3	407.0	790.0	181.1
20	222.2	451.1	674.0	175.0
21	192.2	366.0	347.8	141.0
22	144.0	316.3	738.0	168.0
23	171.1	350.0	1,134.0	100.0
24	215.6	730.0	960.0	75.0
25	307.8	1,188.0	1,384.4	84.0
26	283.0	1,203.0	1,134.4	78.9
27	662.5	932.0	861.0	93.0
28	635.0	719.0	850.0	84.0
29	441.1	728.9	562.0	90.0
30	170.0	494.4	397.8	89.0
31		706.7	516.0	
Average	185.3	771.2	836.8 *)	250.3

*) The average concentration in the month of August is calculated by discarding the outlier values, i.e., the values during August 5 to 15. The average of the sediment concentration from June to September is 510.9 ppm, after discarding the concentration values during August 5 to 15. However, if these outliers are also considered, then the average of the sediment concentration from June to September will be 867.5 ppm.

These data are not sufficient for the estimation of quantity and quality of the sediment transport in the Tamakoshi River. However, if the average of the June-September 2018 concentration values (i.e., 510.9 ppm) given in Table 5 is considered as mean concentration for the mean annual flow (flow with 50% exceedance probability) of 33.67 m^3/s, the annual suspended sediment yield comes out to be 0.542 million tons and in terms of the specific yield it is 310 t/km^2/year (521 t/km^2/year if the outliers are considered as normal values), which is very low compared to the average value for the country of over 4000 t/km^2/year as given in Table 1. An addition of approximately 15% of the suspended sediment load as bed load still results in the total sediment load value in the Tamakoshi River much lower than the national average.

Since the available data of 2018 on suspended sediment were taken in the monsoon period of the year, the sediment concentrations are expected to represent the average sediment load in the River in the monsoon season. The water samples were analyzed for the grain size distribution, mineralogical and petrographic and organic content analysis by the Hydro Lab Pvt. Ltd. The results of the latest sample analysis are presented in Annexure 3.

3.6.5 Estimation of Sediment Yield

The estimation of the sediment in the Tamakoshi River is made based on the results of the previous studies conducted at different parts of Nepal and based on the measured suspended sediment data collected by the NEA in 2003-2004 and the data collected by the Consultant in June-September 2018, as a part of this study. The estimation of the sediment load based on previous studies is conducted for comparison purpose only.

A number of different approaches were applied to derive estimates for the sediment yields, Among them are the Himalayan Sediment Yield Technique (HYST) and approaches developed in relation to several sub-basins, like the method of Gerrard & Gardner, the Galay-WECS SSY Rates method or an approach developed by EDC for the estimation of sediment transport in the Likhu Khola. The applied approaches are presented in greater detail in Part F2 of this report.

The summary of the sediment yields at the headwork of UTK HEP and the tailrace of TK-V HEP from various methods is given in below table, sorted in descending order of the sediment yield values.

Table 3-8: Summary of the suspended sediment yield estimation at the head work of UTK HEP and tailrace of TK-V HEP

	Annual Sediment Yield (million ton/year)	
Basis/Method	**UTK HEP Head work**	**TK-V HEP Tailrace**
SSY rate of Sun Koshi	7.85	11.1
EDC	3.54	4.99
SSY rate of Arun	2.62	3.69
Himalayan Sediment Load Technique	1.17	1.78
Gerrard and Gardner, 2002	0.84	1.18
Galay-WECS, 2003	0.84	1.18
Dudh Koshi	0.616	0.671
UTK HEP	0.53	0.58

A sediment rating curve using the best fit linear regression method was developed for the Tamakoshi River at UTK HEP, based on the available discharge and corresponding suspended sediment data of the NEA (2003-2004 data), given in the following figure.

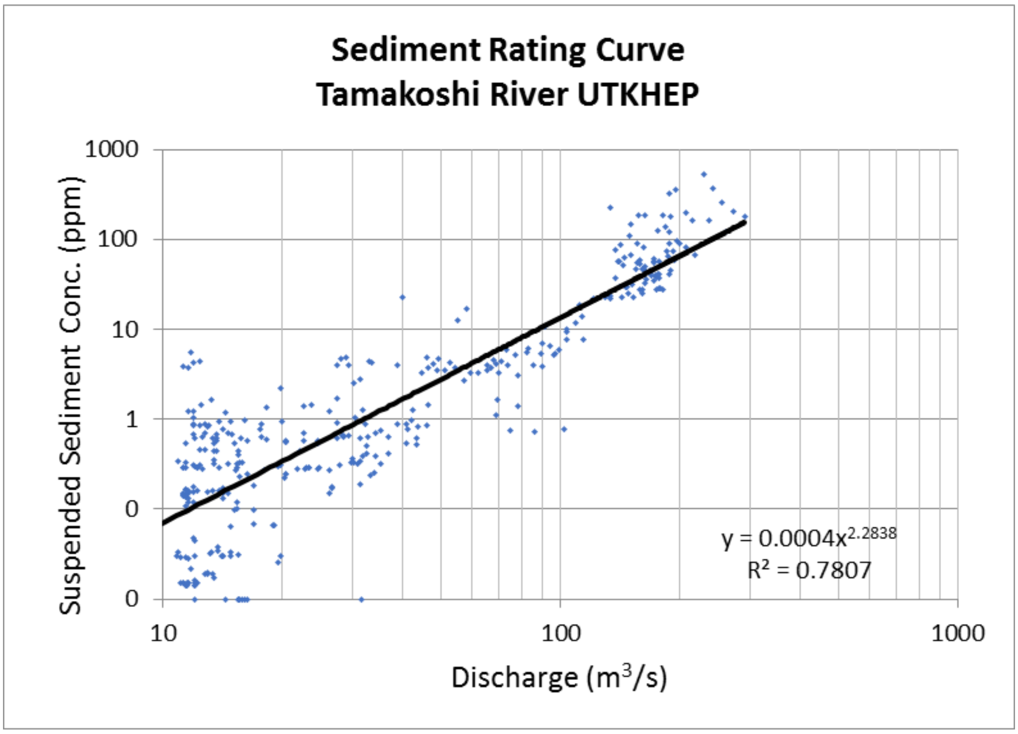

Figure 3-11: Sediment Rating Curve of Tamakoshi River at the headwork of UTK HEP

As shown in above figure, the best fit linear regression equation is $y = 0.0004\ x^{2.2838}$, where y represents the suspended sediment concentration (mm) and x represents the river discharge. However, when back calculation was used to check the difference between the calculated concentration and average value of the measured concentration was checked, a large variation was found. Following the Anandale et al (2016) method[1], the equation $y = 0.00106x^{2.2838}$, where y represents the suspended sediment concentration (ppm) and x represents the river discharge equation was used to generate suspended sediment concentration from the discharge data; with this equation the ratio between the measured and generated sediment concentration was found to be 1.0 for the NEA's sediment concentration data of the years 2003 to 2004 of Tamakoshi River at Lamabagar.

Based on the surveyed discharge and sediment concentration data two estimates were prepared for UTK HEP dam site and the Tamakoshi V outlet site which are presented in the following two tables.

[1] Annandale, George W., Gregory L. Morris, and Pravin Karki, 2016, *Extending the Life of Reservoirs: Sustainable Sediment Management for Dams and Run-of-River Hydropower,* Directions in Development, Washington DC: World Bank. doi: 10.1596/978-1-4648-0838-8. License: Creative Commons Attribution CC BY 3.0 IGO, Chapter 6, page 87

Table 3-9: Estimated monthly suspended sediment concentration and load in Tamakoshi River at UTK HEP dam site (catchment area: 1,745.5 km²)

Month	Month	Monthly Discharge (m^3/s)	Suspended Sediment Conc. Based on rating curve (ppm)	Accumulated flow (million m^3)	Suspended Sediment Load (m^3)	Suspended Sediment Load (tonne)	Monthly Suspended Sediment Yield (tonne/km^2)
Baisakh	Apr-May	32.35	2.975	86.65	258	377	0.22
Jestha	May-Jun	79.205	22.997	212.14	4,879	7,138	4.09
Asadh	Jun-Jul	173.1	137.183	463.71	63,613	93,066	53.32
Srawan	Jul-Aug	235.3	276.490	630.27	174,263	254,946	146.06
Bhadra	Aug-Sep	199.4	189.460	534.13	101,196	148,049	84.82
Ashwin	Sep-Oct	116.2	55.202	311.27	17,183	25,138	14.40
Kartik	Oct-Nov	53.1	9.239	142.30	1,315	1,924	1.10
Mangsir	Nov-Dec	28.1	2.159	75.29	163	238	0.14
Poush	Dec-Jan	20.5	1.050	54.91	58	84	0.05
Magh	Jan-Feb	17.3	0.714	46.39	33	48	0.03
Falgun	Feb-Mar	16.2	0.613	43.38	27	39	0.02
Chaitra	Mar-Apr	18.6	0.837	49.71	42	61	0.03
					Total	531,109	304.27

Table 3-10: Estimated monthly suspended sediment concentration and load in Tamakoshi River at TK-V HEP Tailrace site (catchment area: 2,460 km²)

Month	Month	Monthly Discharge (m^3/s)	Suspended Sediment Conc. Based on rating curve (ppm)	Accumulated flow (million m^3)	Suspended Sediment Load (m^3)	Suspended Sediment Load (tonne)	Monthly Suspended Sediment Yield (tonne/km^2)
Baisakh	Apr-May	32.9	3.092	88.12	272	399	0.16
Jestha	May-Jun	78	22.206	208.92	4,639	6,787	2.76
Asadh	Jun-Jul	173.2	137.310	463.90	63,698	93,190	37.88
Srawan	Jul-Aug	242.1	295.035	648.44	191,313	279,890	113.78
Bhadra	Aug-Sep	207.6	207.678	556.04	115,476	168,942	68.68
Ashwin	Sep-Oct	120.7	60.188	323.28	19,458	28,467	11.57
Kartik	Oct-Nov	56.7	10.697	151.73	1,623	2,375	0.97
Mangsir	Nov-Dec	31.3	2.759	83.83	231	338	0.14
Poush	Dec-Jan	23.1	1.372	61.74	85	124	0.05
Magh	Jan-Feb	19.4	0.926	51.96	48	70	0.03
Falgun	Feb-Mar	18.0	0.775	48.08	37	55	0.02
Chaitra	Mar-Apr	19.9	0.975	53.17	52	76	0.03
					Total	580,712	236.06

The above tables indicate that the sediment concentration and the sediment load in the Tamakoshi River at UTK HEP and TK-V HEP catchment are much less than the results of the other studies in Nepal. The communication with the NEA's senior experts who were involved in design and construction phases of UTK HEP also confirmed that the sediment level in the Tamakoshi is much less compared to the other rivers in the vicinity.

The main reason for this relatively low sediment concentration in the head work area of UTK HEP is anticipated to be related to the fact that a major portion of the catchment lies in Tibetan plateau, which has very low rainfall intensity and low specific sediment yield value. The WECS study also indicates that the specific sediment yield rate in Nepalese rivers reduces with the increase in elevation; only about 1% of the catchment area of TK-V HEP is located in area below 3000 m.

3.6.6 Bed Load

The estimation of bed transport rates is complicated because of the difficulty in obtaining accurate measurements of bed load quantities for either theoretical or laboratory analyses. Instruments designed to measure the bed movement disturb the natural movements of the bed material thus introducing a bias into the measurements, and it is almost impossible to determine the magnitude of this bias. Simultaneous measurements often provide estimates that differ by several hundred percent. The difficulty arises because of an array of factors associated with variations in flow patterns, bed surface characteristics, and particle shape. While these difficulties are recognized, a number of estimation methods have been developed from the theoretical and laboratory analyses. Empirical equations have also been derived from field measurements.

Bed load measurement data are almost nil in Nepal. The bed load is the function of the sediment carrying capacity, which in turn depends on the flow velocity. Therefore, for steeply sloping mountainous rivers, the bed load component in percentage of the total load may be quite high. It needs to be assessed based on the actual measurement of the bed load for at least for one monsoon period.

Two methods are used for the estimation of the bed load viz. the stream sampling method and analytical or empirical method. Various types of samplers such as the box type sampler and the slot type sampler are used for this purpose. These samplers, however, are not satisfactory.

In the absence of site specific data, the bed load is estimated as a percentage of the suspended sediment. In the upper Lesser Himalayas with steep slopes, the bed load is taken as 40 to 60 percentage of the suspended sediment load. Similarly, at the lower Lesser Himalayas, the bed load is taken as 5 to 15 percentage of the suspended sediment load (Schumm 1977, Galay et al 1995). The intake site of the Tamakoshi River is located in the Lesser Himalayas. The bed load was taken as 15% of the suspended sediment load during the design of the Kulekhani reservoir, which lies in the lower Lesser Himalayas (Nippon Koei, 1983). The bed load concentration of the Tamakoshi River may be estimated as 15 per cent of the suspended sediment load. If the sediment yield of 1.2 million ton/yr is to be taken as the suspended sediment load of the Tamakoshi River, then its bed load is about 0.18 million ton/year.

3.6.7 Particle Size Distribution, Mineralogy and Organic Content

The Consultant collected the water samples on a daily basis, from June 1 to September 30, 2018, from the Tamakoshi River, at the river section just below the dam site of UTK HEP, for the particle size distribution and mineralogical and organic content analysis. The collected samples were analyzed in the laboratory of the HydroLab. The analysis results show that the suspended sediments do not contain any particle of size greater than 2 mm, and the quart is the dominant form of the minerals, which has higher hardness compared to the other minerals found in the samples, namely the feldspar, mica, tourmaline and garnet. The following table is the summary of the laboratory analyses of the collected suspended sediment samples.

Table 3-11: Summary of the laboratory analyses of the suspended sediment samples of Tamakoshi

	Mineral content (in %)				
Mixed Sample of the Month	**Quartz**	**Feldspar**	**Mica**	**Organic Content (in %)**	**Analysis date**
June	72	3	14	2	4 August 2018
July	74	3	13	1	27 August 2018
August	73	3	13	1	20 September 2018
September	74	3	13	1	9 October 2018

3.7 Energy Generation Simulations

3.7.1 General Approach

The Tamakoshi V Hydroelectric Project (TV) is designed to divert the tailwater discharge from UTK HEP for generation of electric power at TV before returning the water into Tamakoshi River. Most generally speaking with respect to turbine discharge Tamakoshi V HEP is hence a slave project to UTK HEP and thus depends on the discharge allocated to the upstream project.

The best suitable configuration of the TV generating equipment was during earlier investigations determined to include 3 generating sets with 22 m³/s rated discharge and one small hydro unit designed for 3.3 m³/s rated discharge. All generating sets feature Francis type turbines. The respective rated parameters of the generating sets are compiled in the following table.

Table 3-12: Rated parameters for TV generating units

Parameter		Unit	Large Unit	Small Unit
Discharge	Q_{Rated}	m^3/s	22.00	3.34
Net Head	$H_{N,Rated}$	m	162.35	169.26
Power	$P_{T,Rated}$	MW	31.60	5.08

The investigations reported in this chapter present the results of the energy generation modelling covering the entire annual cycle, separated into peak and non-peak generation as well as dry and wet season generation in line with the NEA Rules for PPA of ROR/PROR/Storage Projects [14] and classification of TV as ROR project (as per information forwarded by the Client). They further cover the scenario of continuous equipment availability as well as the scenario that the equipment will be operated under consideration of scheduled inspection outages at UTK HEP and Tamakoshi V HEP.

The approach applied for the analyses makes principally use of the following subsequent steps:

- the determination of water availability and the processing of available data to suit the subsequent steps of evaluation;

- the allocation of available water for various purposes of use in the hydropower plants, which data will then be employed as inputs to the energy generation simulations;
- the modelling of peak energy generation and the determination of generation constraints;
- the definition of dispatch patterns for the power station equipment when operated under non-peak generation mode,
- the modelling of non-peak energy generation; and finally
- totalizing of the partial results to obtain energy generation indicators for the entire period of modelling.

3.7.2 Availability of Water

For the present study on the potential for energy generation recent flow data for the Tamakoshi River are available from the analysis of long term flows above. That section presents long term mean monthly river discharges for the Tamakoshi River at Lamabagar, Bhaise Khola and for the Rolwaling River.

Bhaise Khola is already connected to the UTK headrace tunnel and provides a firm inflow contribution. The connection of Rolwaling River is, to the contrary, still under development and has hence to be classified as “not yet confirmed”. The simulations consequently distinguish into scenarios with and without inflow contribution provided by the Rolwaling River via the projected transfer scheme to the Upper Tamakoshi reservoir.

The inflow contributions provided by the three water sources were considered in the simulations with the following proportions:

- The inflows from Tamakoshi River as stated in the analysis of long term flows were adjusted by the catchment correction factor 0.9837 to account for the difference in catchment sizes of (i) the Lamabagar gauge site and (ii) the UTK dam site [2, p. 4-7]. The resulting inflows were reduced for the portion of the legally prescribed environmental release, which is 10% of the long term average monthly discharge of the driest month.
- The inflows from Bhainse Khola were reduced for the portion of the legally prescribed environmental release (determined as above).
- The inflows from Rolwaling River were as well reduced for the portion of the legally prescribed environmental release (determined as above). The remaining available inflows were, after converting the data into daily flows, capped at a maximum contribution of 13.4 m^3/s in line with the recommended design of the Rolwaling Diversion Scheme.

The respective inflow data are compiled in the table below as taken from the analysis of long term flows and after adjustments in line with the afore-mentioned considerations. Also included are the flow data for the site of the TV Outlet Structure; these data were used for the determination of corresponding tailwater levels.

Table 3-13: Long term mean monthly Flow Data as used in the Simulations (values in [m³/s], lowest long term mean monthly discharges are marked bold-red, ER = Environm. Release)

Month	Lama-GS2	UTK Dam	UTK Dam excl. ER	Bhainse	Bhainse excl. ER	Rolw-GS8	Rolw-GS8 excl. ER	TK-V Outlet
Jan	14.63	14.39	13.16	0.41	0.39	3.24	2.93	20.60
Feb	12.92	12.71	11.47	0.32	0.29	**3.12**	2.81	18.20
Mar	12.52	**12.32**	11.09	0.28	0.25	3.23	2.92	17.70
Apr	15.58	15.32	14.09	**0.25**	0.23	5.26	4.95	22.00
May	31.08	30.57	29.34	0.33	0.30	12.20	11.89	43.80
Jun	79.61	78.32	77.09	1.11	1.08	34.07	33.76	112.20
Jul	166.16	163.46	162.22	3.56	3.53	61.74	61.43	234.20
Aug	177.37	174.48	173.25	4.09	4.06	57.71	57.40	250.00
Sep	117.19	115.28	114.04	3.38	3.35	39.10	38.79	165.20
Oct	54.03	53.15	51.92	1.79	1.77	16.94	16.62	76.20
Nov	26.32	25.89	24.66	0.79	0.76	6.39	6.08	37.10
Dec	18.06	17.76	16.53	0.49	0.47	4.18	3.86	25.50

3.7.3 Preparation of Data to fit the Analyses

The modelling of the energy generation by the Tamakoshi V HEP requires that daily discharge data can be fed into the numerical model. It consequently necessitates that the long term mean monthly river discharges are transformed into daily discharge data using suitable transformation methods. A possible approach is to approximate the series of monthly flow data (or parts thereof) by mathematical polynomials; this approach was applied for the present data processing.

In order to have the monthly flow data converted into daily data at satisfactory accuracy, the data covering the entire year were divided into two parts representing a dry and a wet period of the year (note that for the purpose of this analysis the periods are different from the seasons addressed in [14]; therefore, a careful distinction between periods and seasons has to be borne in mind). After analyzing the full annual cycle data series, the most suitable division into the dry and wet period series was found by allocating the months of November to May to the dry period and the months of July to September to the wet period. The months of June and October were classified as transition periods.

3.7.4 Allocation of Inflows to UTK HEP Reservoir

The requirements governing the allocation of inflows to UTK HEP Reservoir as well as the foreseen operation of UTK HEP contingent upon the water availability were clarified to be done in line with the rules summarized here below:

- The release of environmental discharge from the UTK dam into the Tamakoshi river bed will be served with highest priority.
- The second highest priority (after satisfying the provision of environmental release) is allocated to the concept that at least one unit at UTK HEP is always operated at spinning reserve. It was clarified that a minimum discharge of 1.5 m³/s will be required to follow this operation principle.
- Third priority is given to the generation of peak energy in line with the following operation principles:
  (i) During the wet season sufficient water will be available over the entire day to run UTK HEP at its maximum power discharge of 66 m³/s. This operation is equivalent to topping up the

spinning reserve discharge by another 64.5 m³/s. Excess water will be released through the spillway at UTK dam.

(ii) If water inflow is not sufficient to follow the operation principle of (i), UTK HEP shall be operated in peaking mode for at least 6 hours during the day. Since for the remainder of the day no specific UTK HEP operation rules were communicated, the simulations were set up under the assumption that flat operation at constant turbine discharge will be applied.

(iii) In the event that the inflow into UTK HEP reservoir reduces that much that peaking over 6 hours would not be possible without reducing the spinning reserve discharge, the peaking time will be shortened as required to maintain the specified spinning reserve operation.

- A discharge through the power waterways in excess of 66 m³/s (e.g. by over-opening the turbines) was generally ruled out for the simulations presented herein since the discharge of 66 m³/s is understood as concession limit for UTK HPL.

The numerical simulation of UTK HEP operation is principally performed by balancing in- and outflowing daily water volumes. For the numerical modelling it is not of relevance whether the daily peaking at UTK is scheduled for one or more periods. With a life storage capacity of 1.2 mio. m³ UTK HEP reservoir is big enough to supply water for a daily continuous 6 hours peaking operation as long as the reservoir inflow excl. environmental release remains above about 10.45 m³/s (which is supported by the hydrological data of Section 17.2.1).

3.7.5 Results of the Generation Simulations

The energy data computed for the high voltage terminals of the TV main transformers are derived under consideration of efficiencies of 98.0% for the generators and 99.5% for the step-up transformers. The respective results are compiled in the following table.

Table 3-14: Results for generated annual energy amounts in GWh/a, valid for high voltage terminals of TV main transformers

Scenario	Wet Season		Dry Season		Totals		
	Peak	Non-Peak	Peak	Non-Peak	Peak	Non-Peak	Total
Without Rolwaling Outages not considered	338.13	73.61	48.43	4.61	386.55	78.22	464.77
Without Rolwaling Outages considered	338.09	73.60	45.49	4.48	383.58	78.08	461.66
With Rolwaling Outages not considered	373.61	75.27	62.18	5.49	435.79	80.76	516.55
With Rolwaling Outages considered	373.61	75.27	55.54	7.91	429.15	83.18	512.33

From the data given in above table it is apparent that the consideration of outages has almost no effect on the energy generation during the wet season, for the scenarios without consideration of the Rolwaling Diversion Scheme the influence is negligible (reduction by 0.05 GWh/a), for the scenarios with consideration of Rolwaling not existing.

With respect to the energy generation during the dry season the consideration of outages results for the scenario without consideration of Rolwaling in the first place in a reduction of the peak energy production by almost 3 GWh/a, whereas the reduction in non-peak energy is computed very small at 0.13 GWh/a. For the scenario with consideration of Rolwaling the reduction of the peak energy production is obtained even more prominent at 6.64 GWh/a, whereas, however, the non-peak energy production increases by 2.42 GWh/a due to the same modification in the simulation. This latter effect is a result of an increase of small turbine flows released from UTK HEP during UTK HEP outages which provide more water for the small hydro unit at TV.

The following table presents the results again for the scenarios reported before, however, in this case including line losses occurring in the transmission line from TV to Khimti SS and attributable to the energy fed in at TV. For this analysis the losses in the transmission line from UTK HEP to Khimti were calculated for operation of UTK HEP alone at rated condition (456 MW) and for combined operation of UTK HEP and TV at rated condition (456 MW + 94.8 MW). In a simplified approach it was generally assumed that UTK HEP and TV will operate at all times at the same ratio of energy infeed, and that the line losses will change proportionally with the square of the current caused by the infeed.

For the rated condition the line losses were determined as 7.34 MW for UTK HEP operating alone and as 10.58 MW for the situation of combined operation; the additional line losses caused by adding TV to the transmission line hence amount to 3.24 MW. With this parameter for operation at rated condition the energy delivered by TV at Khimti SS were computed as compiled in the table below.

Table 3-15: Results for generated annual energy amounts in GWh/a, valid for delivery at Khimti SS

Scenario	Wet Season		Dry Season		Totals		
	Peak	Non-Peak	Peak	Non-Peak	Peak	Non-Peak	Total
Without Rolwaling Outages not considered	326.58	72.06	46.76	4.61	373.34	76.67	450.01
Without Rolwaling Outages considered	326.55	72.05	44.16	4.48	370.71	76.53	447.24
With Rolwaling Outages not considered	360.85	73.66	60.04	5.49	420.89	79.15	500.04
With Rolwaling Outages considered	360.85	73.66	53.93	7.90	414.78	81.56	496.34

The following tables provide an overview over the energy generation on monthly basis under the different simulation scenarios.

Table 3-16: Monthly Energy Generation for: Rolwaling not considered / Outages not considered / Data in MWh

	Energy at T V Outgoing Terminals		Energy at Khimti Incoming Terminals	
	Peak	Non-Peak	Peak	Non-Peak
Jan	13,916.38	1,174.18	13,439.13	1,173.34
Feb	11,007.35	1,092.87	10,629.85	1,092.10
Mar	10,590.88	1,243.00	10,227.65	1,242.12
Apr	13,492.35	1,295.22	13,029.65	1,294.03
May	17,685.74	16,977.44	17,079.48	16,718.42
Jun	52,938.33	12,944.85	51,127.68	12,567.78
Jul	70,325.13	0.00	67,928.63	0.00
Aug	70,278.49	0.00	67,885.17	0.00
Sep	68,225.67	0.00	65,894.94	0.00
Oct	24,519.29	32,413.03	23,679.20	31,589.98
Nov	17,118.74	9,629.95	16,531.79	9,545.90
Dec	16,455.55	1,445.25	15,891.25	1,443.69
Subtotals	**386,553.89**	**78,215.80**	**373,344.41**	**76,667.36**
TOTALS		**464,769.70**		**450,011.77**

Table 3-17: Monthly Energy Generation for: Rolwaling not considered / Outages considered / Data in MWh

	Energy at T V Outgoing Terminals		Energy at Khimti Incoming Terminals	
	Peak	Non-Peak	Peak	Non-Peak
Jan	13,981.72	1,157.71	13,523.40	1,156.89
Feb	9,790.75	1,046.14	9,542.82	1,045.39
Mar	9,366.55	1,198.03	9,130.90	1,197.18
Apr	12,892.84	1,268.97	12,498.37	1,267.81
May	17,685.74	16,977.44	17,079.48	16,718.42
Jun	52,938.33	12,944.85	51,127.68	12,567.78
Jul	70,325.13	0.00	67,928.63	0.00
Aug	70,278.49	0.00	67,885.17	0.00
Sep	68,225.67	0.00	65,894.94	0.00
Oct	24,519.29	32,413.03	23,679.20	31,589.98
Nov	17,118.74	9,629.95	16,531.79	9,545.90
Dec	16,455.55	1,445.25	15,891.25	1,443.69
Subtotals	**383,578.79**	**78,081.38**	**370,713.62**	**76,533.04**
TOTALS		**461,660.17**		**447,246.65**

Table 3-18: Monthly Energy Generation for: Rolwaling considered / Outages not considered / Data in MWh

	Energy at T V Outgoing Terminals		Energy at Khimti Incoming Terminals	
	Peak	Non-Peak	Peak	Non-Peak
Jan	17,553.37	1,388.00	16,951.28	1,386.68
Feb	14,427.79	1,022.62	13,932.89	1,021.90
Mar	14,264.14	1,167.57	13,774.85	1,166.75
Apr	16,503.58	3,171.87	15,937.45	3,159.73
May	17,696.48	30,468.92	17,089.48	29,784.16
Jun	66,629.62	1,676.82	64,349.66	1,620.50
Jul	70,325.13	0.00	67,928.63	0.00
Aug	70,278.49	0.00	67,885.17	0.00
Sep	68,225.67	0.00	65,894.94	0.00
Oct	45,063.95	21,304.54	43,519.38	20,686.60
Nov	17,124.64	17,377.72	16,537.29	17,143.49
Dec	17,695.91	3,184.44	17,088.95	3,177.16
Subtotals	**435,788.76**	**80,762.50**	**420,889.96**	**79,146.96**
TOTALS		**516,551.25**		**500,036.92**

Table 3-19: Monthly Energy Generation for: Rolwaling considered / Outages considered / Data in MWh

	Energy at T V Outgoing Terminals		Energy at Khimti Incoming Terminals	
	Peak	Non-Peak	Peak	Non-Peak
Jan	16,471.67	2,162.58	15,941.54	2,159.23
Feb	11,725.70	2,219.73	11,430.65	2,216.03
Mar	12,608.59	1,116.65	12,291.32	1,115.85
Apr	15,301.83	3,672.15	14,822.86	3,658.77
May	17,696.48	30,468.92	17,089.48	29,784.16
Jun	66,629.62	1,676.82	64,349.66	1,620.50
Jul	70,325.13	0.00	67,928.63	0.00
Aug	70,278.49	0.00	67,885.17	0.00
Sep	68,225.67	0.00	65,894.94	0.00
Oct	45,063.95	21,304.54	43,519.38	20,686.60
Nov	17,124.64	17,377.72	16,537.29	17,143.49
Dec	17,695.91	3,184.44	17,088.95	3,177.16
Subtotals	**429,147.68**	**83,183.54**	**414,779.86**	**81,561.80**
TOTALS		**512,331.22**		**496,341.65**

Finally, the operation characteristics of TV was determined for two characteristic discharges in Tamakoshi River, i.e. for the discharges Q32 and Q40 as per the data provided in the table for the Flow Duration Curve (see Part F2 of this report). The respective flow data for Tamakoshi River and Bhainse Khola were calculated under consideration of the environmental discharge to be released at both water intakes and otherwise without applying any further adjustments; the respective river discharges were derived as Q40 = 38.62 m³/s and Q32 = 62.34 m³/s. The respective operation conditions can be taken from discharge allocation tables for the scenarios not considering the Rolwaling Diversion Scheme and are found at calendar days nos. 142 (for Q40) and 158 (for Q32). The associated amounts of generated energies are determined as

- 1,330.78 MWh daily generated energy for Q40; and
- 2,169.93 MWh daily generated energy for Q32.

3.8 Conclusions related to the hydrological aspects of the Tamakoshi V HEP

The following are the basic conclusions for the Tamakoshi V HEP based on the analysis of the available pertinent data.

1. The Tamakoshi V HEP received water for energy generation from the tailrace of UTK HEP project. The analysis of the available flow data at the Tamakoshi River at Busti (St. No. 647), Tamakoshi at Lamabagar, Rolwaling, and Bhainse Khola shows that the dry month flow in the Tamakoshi at Lamabagar can be expected to decrease by around 8 percent. This decrease can be compensated by the diversion of Rolwaling flow in the dry months to ensure reliable power generation. Hence, the effect on the energy production from the Tamakoshi V project due to gradual decline in Tamakoshi River’s dry month flow is expected to be quite low.

2. The instantaneous flood in the Tamakoshi River at Lamabagar for the return periods of 100 years and 10,000 years are 685 m³/s and 1,049 m³/s, respectively; the respective values for the same return periods were listed as 638 m³/s and 1,153 m³/s in the Feasibility Study Report of UTK HEP (2005) and are thus in good agreement. The slight discrepancy between these values can result from additional data used in the recent study.

3. The structures of the Tamakoshi V HEP being exposed to riverine floods shall be designed to withstand the greater of the following two design flood discharges: (i) the discharge associated with the PMF as determined in line with the approach mentioned in Chapter 15, i.e. twice the magnitude of the 1 in 10,000 years peak discharge, and (ii) the peak discharge of the flood with 100 years return period escalated by 40% for the potential climate change effect by the year 2060 and superimposed to the GLOF peak discharge from Tsho Rolpa expected for the sites. This latter discharge is stated as approx. 3,500 m³/s for the site of the Spillway Terminal Structure and as approx. 3,000 m³/s for the sites of the Power Station and Outlet Structure.

   Owing to the uncertainty associated with the determination of the GLOF peak discharges it is further recommended to design these structures such that their designs can easily be modified to withstand also slightly greater peak flood discharges.

4. The months of May to September are the wet months in the Tamakoshi V catchment area. More than 60 percent of the days as rainfall days can be expected in these months. Hence outdoor con-

struction activities should be scheduled and prepared accordingly and, if possible, be avoided or at least minimized.

5. Monitoring the water level, rainfall and temperature at the available river gauging facilities and meteorological stations at the project site shall be continued, augmented by periodic (monthly) discharge measurements. The sediment sampling should also be continued so that the design of the structures which may be affected by the sediment concentration can be updated accordingly, before the construction begins.

6. The energy generation simulations revealed that the Tamakoshi V HEP can generate 450 GWh/a energy for the scenario without consideration of the Rolwaling Diversion Scheme, and 500 GWh/a with Rolwaling implemented (figures referring to the delivery at Khimti SS incoming terminals, no scheduled outages). If scheduled outages are simulated, the above energy amounts reduce by some 3 GWh/a. Out of the total energy, about 83% is peak energy; referring to the wet season alone this portion is 82%, referring to the dry season it is 91% (scenarios without Rolwaling, seasonality as per [14] for ROR projects, irrespective of outages).

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4 PROJECT GEOLOGY & SEISMICITY

4.1 Introduction

4.1.1 General

The Tamakoshi 'V' Hydroelectric Project is a cascade development scheme of Upper Tamakoshi HEP with installed capacity of 99.8 MW owned by Nepal Electricity Authority (NEA). Major civil components in the project are head pond, spillway, headrace tunnel, surge tank, valve chamber, powerhouse, switchyard, tailrace and four access tunnels. All structures are underground.

As an engineering geological investigation, a detailed geological mapping of the project area as well as a precise engineering geological study was carried out for several times taking traverses through different locations. The study helped to determine nature and behavior of structural features of the rock mass. The study provides desired information about possible challenges during construction stage. It was also helpful on the interpretation of sub-surface geology, on account of civil components design, determination of location for open and underground structures, selection of tunnel route and determination of suitable construction methodology.

Overbreak and water ingress challenges during underground excavations are primarily associated with geological structures. Taking them into account, a precise study has been carried out especially about fold, fault, shear zone, fractures etc.

Apart from geological / engineering geological mapping, Core Drilling, In-situ testing (Drill hole & Test Adit), Laboratory testing and Electrical Resistivity Tomography (ERT) survey have also been carried out for individual component as complementary tasks of exploratory geological investigation.

4.1.2 Objective

Main objectives of the geological/ geotechnical investigation in the project:

- To determine geologically appropriate location for civil components,
- To provide engineering geological information for selection of tunnel alignment,
- To provide engineering geological information to establish the construction methodology for tunnels, caverns, shaft & portals,
- To assess the possible challenges during construction of components, and
- To determine appropriate exploratory geotechnical tests, testing criteria and locations for detail investigation of individual component.

4.1.3 Topography and Geomorphic Features

Project area is located on the northern limb of an anticlinorium. The core of it lies near outlet structure. Rocks are gently dipping at the Outlet. In north direction, rocks are dipping towards north with gradually higher inclinations. The inclination reaches up to 70^0 at the Headwork site.

Higher Himalayan rock succession is forming high altitude mountains and the Lesser Himalayan succession is forming comparatively low altitude mountains (see Figure 4-1 & Figure 4-2).

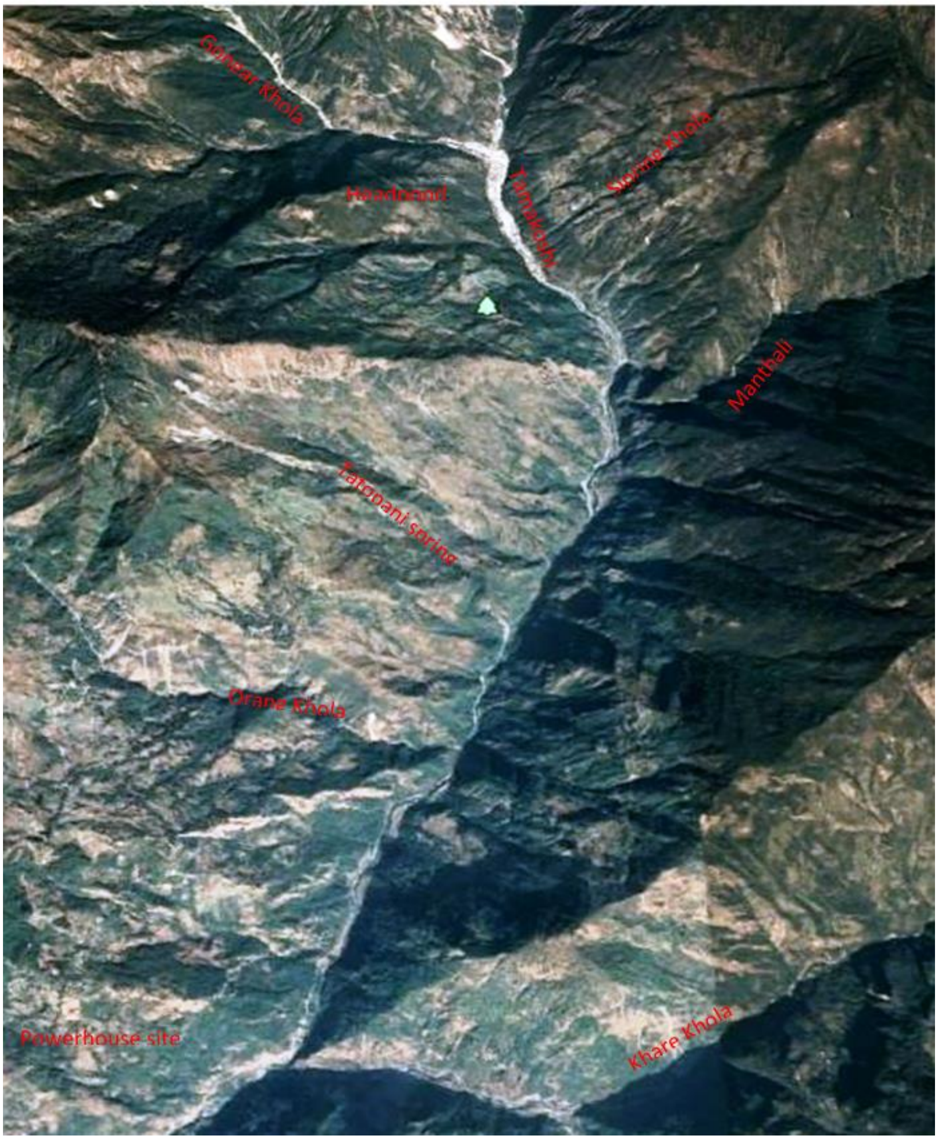

Figure 4-1: Satellite Image Showing Geomorphic Features of Mountains and River System within Project Area

Figure 4-2: Higher and Lesser Himalayan Range in Project Area

In general, mountain ridges are extending in the south – north direction. Slope of the mountain ridge is controlled by foliation joint. In the Powerhouse site, hill is composed of augen gneiss with sub-horizontal foliation into the hill and vertical slopes towards valley (see Figure 4-3).

Figure 4-3: Powerhouse Area Topography

Escarpment of ridges are basically controlled by joints. They are forming steep slopes and cliffs on the southern flanks. Formation of deep gorges, steep slopes, cliffs and active gullies in the project represent the erosional landforms. Some small rugged hills formed on the foot of the mountain are composed of talus and colluvial deposit.

Flat surfaces on the ridge where thickness of soil is more than 2m are normally used as cultivated land. Apart of flat agricultural land, rest of the land surfaces are full of vegetation and forests. Rocks are normally exposed on the steep slopes & cliffs.

The project area ranges in altitude from minimum of 983 masl on the river valley at Tailrace Outlet and maximum of 3000 masl at the peak of Philinge Dada. Likewise, along HRT alignment, the altitude ranges from 1180 masl to 1840 masl.

Main river system in the region is the Tamakoshi River. Its major tributaries in the project are Gongar Khola, Sipring Khola, Maure Khola, Khani Khola, Manthali Khola, Orang Khola, Tatopani Khola and Khare Khola (see Figure 4-1). Tributaries are flowing from either side of the Tamakoshi River nearly in perpendicular direction. These rivers with their catchment drains are flowing in the Dendritic and Angular pattern. In addition, there are some waterfalls and gullies which are seasonal and contains significant quantity of water during summer.

4.2 Methodology

Methodology followed to accomplish the stated tasks and objectives is described below:

4.2.1 Topography and Geomorphic Features Desk Study

During desk study, all the previous engineering geological reports of the project provided by Nepal Electricity Authority (NEA) have been studied. Reports are quite helpful in getting concept and information about the project, especially in terms of engineering geology, geological mapping & geological structure.

100m (approx.) of core drilling along with point load and uniaxial compression tests had been carried out during the feasibility study. The core logs, lab test results had been studied. Drill hole core samples were also checked. To avoid some confusion, in mutual understanding with NEA, few core samples were also sent for petrographic analyses outsourcing.

Seismic Refraction Survey at the powerhouse area and along expected Main Central Thrust (MCT) zone has been studied. Likewise, Electrical Resistivity Tomography (ERT) done along the expected MCT zone has also been studied.

4.2.2 Reconnaissance survey

Some reconnaissance surveys have been carried out at the project area after getting information from the previous reports provided by NEA. Main objectives of the reconnaissance were to update the geological / engineering geological map with sufficient field records. The reconnaissance was carried out through different locations across the project area. Geological field observations were done with geological data (Dip and strike of foliation surfaces & joints) at several rock outcrops. Then geological report and geological map were updated successively with the updated field information. During update of the map, field visits have been arranged for several times. Then the geological / engineering geological report was updated accordingly.

For all the civil components, appropriate portal locations of Adits, underground location of Powerhouse & Surge Tank, Headrace Tunnel alignment, Spillway and Tailrace tunnel alignment was determined by the updated geological map and the geological / engineering geological report.

4.2.3 Exploratory Boreholes

Five exploratory boreholes were carried out at different structure locations. The boreholes were TT-1, SW-1, HB-1', HB-2' and SP-1. Drill holes are completed at TRT Outlet structure, Tailrace Tunnel alignment, HRT / shear zone intersection/ Tatopani, HRT / Orang Khola intersection and at Spillway portal respectively (refer Table 4-1). Completed drill depth in total is about 277m.

Objectives of drilling is to assess characteristics of the sub-surface material, establishment of bedrock and groundwater level.

Table 4-1: Summary of Completed Drill Hole

S. No	Drill Hole No.	Location	Elevation (m)	Total Depth (m)	Overburden Depth (m)	Water Table (m)	Rock Type
1	TT 1	TRT Outlet	1012.30	32.00	2.50	29.80	Augen Gneiss
2	SW 1	TRT Align-ment	1043.20	40.10	32.50	29.80	Augen Gneiss
3	HB 1'	HRT Alignment/ Tatopani	1412.75	80.00	2.45	49.10	Meta-carbonate/ Carbonate
4	HB 2'	HRT Alignment/ Orang Kho-la	1185.21	65.50	0.00	3.20	Augen Gneiss
5	SP 1	Spillway Tunnel	1161.81	60	19.2	---	Banded Gneiss

In-situ tests like Standard Penetration Test (SPT), Dynamic Cone Penetration Test (DCPT), Permeability test and Water Pressure (Lugeon) test have also been completed in the boreholes.

4.2.4 Electrical Resistivity Tomography (ERT) Survey

ERT survey is conducted at four locations within the project. Locations are HRT / MCT intersection, HRT / shear zone area intersection, tailrace tunnel alignment and terminal & ventilation building take off yard. Completed linear length of the ERT test is 1788m. Objective of the ERT test is:

1. To infer the characteristics of sub-surface material along the MCT zone,
2. To know characteristics of sub-surface material in the shear zone,
3. To identify rock line below the ground at the tailrace tunnel alignment, and
4. To know the characteristics of sub-surface material for foundation of terminal buildings.

4.2.5 Evaluation of Rock Mass

Two well established evaluation methods are used to determine the rock mass characteristics for underground excavation namely i. Geomechanics Classification (RMR System), and ii. Rock Tunneling Quality Index (Q System).

4.2.5.1 Geomechanics Classification

Bieniawski (1976) published the details of a rock mass classification called the Geomechanics Classification or the Rock Mass Rating (RMR) system. Over the years, this system has been successively refined as more case records have been examined and the reader should be aware that Bieniawski has made significant changes in the ratings assigned to different parameters. The discussion which follows is based upon the 1989 version of the classification (Bieniawski, 1989). Both this version and the 1976 version deal with estimating the strength of rock masses. The following six parameters are used to classify a rock mass using the RMR system:

1. Uniaxial compressive strength of rock material.
2. Rock Quality Designation (RQD).
3. Spacing of discontinuities.
4. Condition of discontinuities.
5. Groundwater conditions.
6. Orientation of discontinuities.

In applying this classification system, the rock mass is divided into a number of structural regions and each region is classified separately. Boundaries of the structural regions usually coincide with a major structural feature such as a fault or with a change in rock type. In some cases, significant changes in discontinuity spacing or characteristics, within the same rock type, may necessitate the division of the rock mass into a number of small structural regions.

The Rock Mass Rating (RMR) system is presented in detail in Part F3 of this report. The method applied aims at determining ratings for each of the six parameters listed above. These ratings are summed up to obtain a value of RMR. Bieniawski (1989) published a set of guidelines for the selection of support in tunnels in rock for which the value of RMR had been determined.

4.2.5.2 Rock Tunneling Quality Index (Q System)

On the basis of an evaluation of a large number of case histories of underground excavations, Barton et al (1974) of the Norwegian Geotechnical Institute (NGI) proposed a tunneling quality index (Q) for the determination of rock mass characteristics and tunnel support requirements. Numerical value of the index Q varies on a logarithmic scale from 0.001 to a maximum of 1,000 and is defined by:

Q=RQD/Jn x Jr/Ja x Jw/SRF

Where, RQD is the Rock Quality Designation

Jn is the joint set number

Jr is the joint roughness number

Ja is the joint alteration number

Jw is the joint water reduction factor

SRF is the stress reduction factor

In explaining the meaning of the parameters used to determine the value of Q, Barton et al (1974) offer the following comments:

First quotient (RQD/Jn), representing the structure of the rock mass, is a crude measure of the block or particle size, with the two extreme values (100/0.5 and 10/20) differing by a factor of 400. If the quotient is interpreted in units of centimeters, the extreme 'particle sizes' of 200 to 0.5 cm are seen to be crude but fairly realistic approximations. Probably the largest blocks should be several times this size and the smallest fragments less than half the size. (Clay particles are of course excluded).

Second quotient (Jr/Ja) represents the roughness and frictional characteristics of the joint walls or filling materials. This quotient is weighted in favor of rough, unaltered joints in direct contact. It is to be expected that such surfaces will be close to peak strength, that they will dilate strongly when sheared, and they will therefore be especially favorable to tunnel stability.

When rock joints have thin clay mineral coatings and fillings, the strength is reduced significantly. Nevertheless, rock wall contact after small shear displacements have occurred may be a very important factor for preserving the excavation from ultimate failure.

Where no rock wall contact exists, the conditions are extremely unfavorable to tunnel stability.

Third quotient (Jw/SRF) consists of two stress parameters. SRF is a measure of: 1) loosening load in the case of an excavation through shear zones and clay bearing rock, 2) rock stress in competent rock, and 3) squeezing loads in plastic incompetent rocks. It can be regarded as a total stress parameter. The parameter Jw is a measure of water pressure, which has an adverse effect on the shear strength of joints due to a reduction in effective normal stress. Water may, in addition, cause softening and possible out-wash in the case of clay-filled joints. It has proved impossible to combine these two parameters in terms of inter-block effective stress, because paradoxically a high value of effective normal stress may sometimes signify less stable conditions than a low value, despite the higher shear strength. The quotient (Jw/SRF) is a complicated empirical factor describing the 'active stress'.

It appears that rock tunneling quality Q can now be considered to be a function of only three parameters which are crude measures of:

1. Block size: (RQD/Jn)
2. Inter-block shear strength: (Jr/ Ja)
3. Active stress: (Jw/SRF)

A table (after Barton et al 1974) giving the classification of individual parameters used to obtain a tunneling quality index (Q) for a rock mass is presented in Part F3 of the Detailed Design Report.

4.2.6 Rock Mass Classification Criteria

Generally, it is underlined that the use of two rock mass classification schemes side by side is advisable. In many cases, it is appropriate to give a range of values to each parameter in a rock mass classification and to evaluate the significance of the final result.

The rock mass classes in the tunnel alignments in Tamakoshi V HEP are determined in a combined way with the criteria based on the Rock Mass Rating (RMR) classification system by Bieniawski (1989) and the Tunneling Quality Index (Q) by Barton (1974).

The rock mass classes are then categorized more or less as per ranges of RMR and Q as shown in the table below:

Table 4-2: Generalized Criteria of Rock Mass Classification System

S.N.	Rock Classes	RMR Value	Q - Value
1	I	81 - 100	> 10.0
2	II	61 - 80	4.0 – 10.0
3	III	41 - 60	1.0 – 4.0
4	IV	21 - 40	0.5 – 1.0
5	V	0 - 20	< 0.5

4.3 Overall Geological Appraisal

4.3.1 Brief Account of the Nepal Himalaya

4.3.1.1 General Description

The Himalayan Range is a young mountain system in the world. It is a broad continuous arc along the northern fringes of the Indian subcontinent, from the bend of the Indus River in the northwest to the Brahmaputra River in the east. The Himalayan mountain chain extends in an east-west direction between the wide plains of the Indus and the Brahmaputra in the south and vast expanse of the high Tibetan Plateau in the north. The limit of the Himalayas in the east and west is marked by the eastern and western arc of Himalayan bends. Between these bends the Himalayan range is approximately 2400 km long and 200 km to 300 km wide. The Himalayas cover an area of approximately 600,000 sq. km in south Asia.

The Himalaya was formed by the collision of the Indian Plate with the Tibetan (Eurasian) Plate around 55 million years ago (see Figure 4-4). Many scientists believe that at that time the northward moving Indian plate first touched the southern edge of Tibetan (Eurasian) plate.

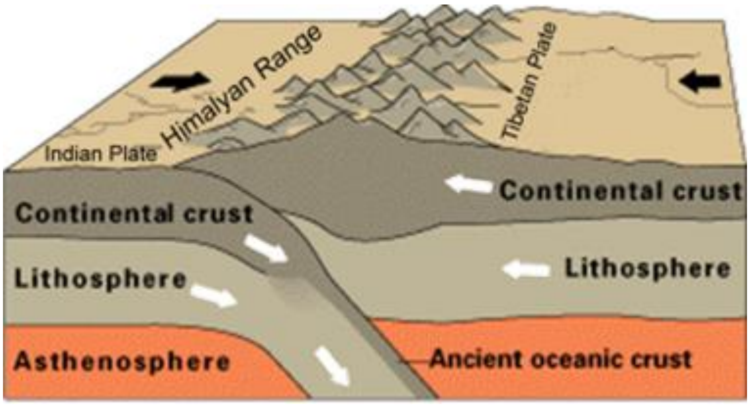

Figure 4-4: Collision of Indian Plate with Tibetan Plate and Formation of Himalaya (Modified after USGS, 1999)

The mountain building (Orogeny) process continues from the collision and the mountain is still on making process. This is noticeable by present day northward movement of India at the rate of 5 cm per year and the occurrences of frequent seismic shakes all along the Himalaya and its surroundings (Jackson and Bilhan, 1994, Pandey et al., 1995, Bilham et al., 1998). Most part of the drift is accommodated within the Himalaya by various thrusts as well as rising peaks. The Himalayan mountain system developed in a series of stages 30 to 50 million years ago and they are still active and continue to rise today. Himalaya is considered as a tectonically very active and vulnerable mountain system in the world.

4.3.1.2 Morpho-tectonic Division of Nepal Himalaya

In 1964, Augusto Gansser provided the first comprehensive picture of the Himalayas (see Figure 4-5) and he had transversely divided the whole Himalayan range into following five major groups:

1. Gangetic Plain
2. Sub-Himalayan Zone
3. Lesser Himalayan Zone
4. Higher Himalayan Zone
5. Tibetan-Tethys Himalayan Zone

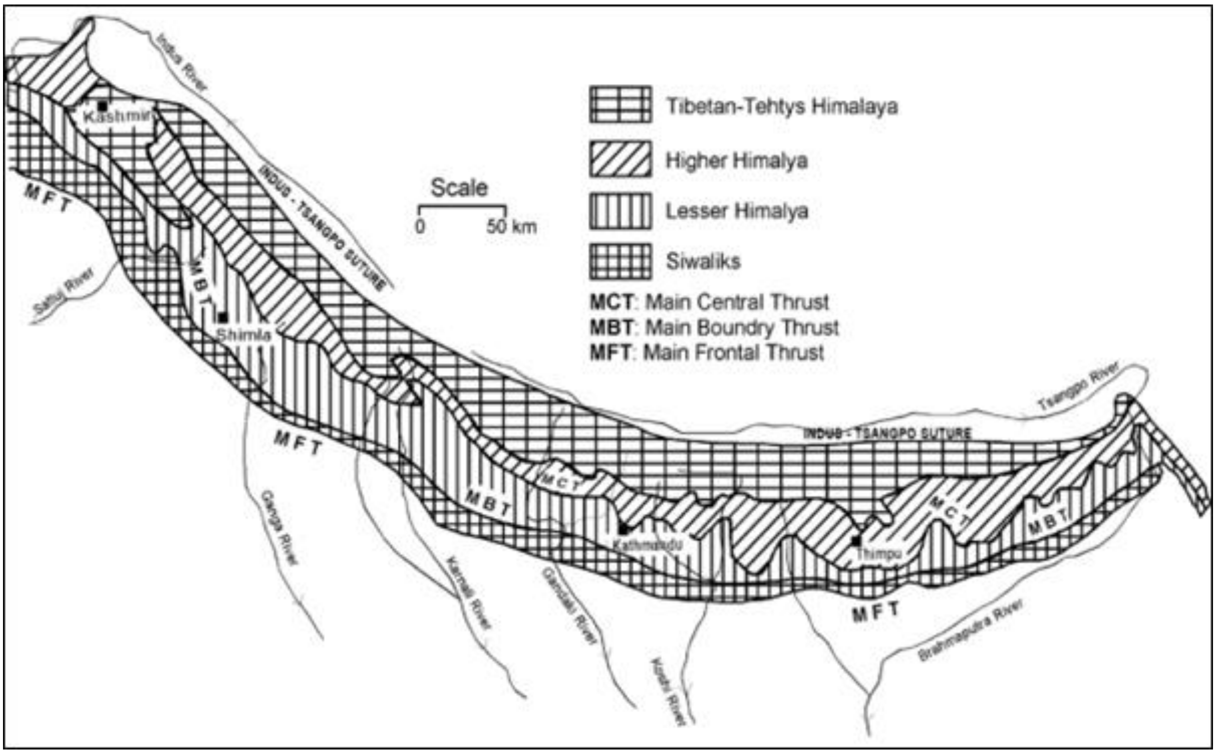

Figure 4-5: Longitudinal Sub-division of the Himalaya (Modified after Gansser 1964)

1. Gangetic Plain:

The Gangetic plain is also called the Terai which is a rich, fertile and ancient land in the southern parts of Nepal. It represents Holocene/Recent sedimentation belt where fluvial sedimentation is still in progress. This plain is less than 200 m above sea level and has thick (about 1500 m) alluvial deposit. The alluvial deposits mainly consist of boulders, gravel, sand, silt and clay. It is a foreland basin which consists of the sediments brought down from the northern part of Nepal.

2. Sub-Himalayan Zone

The Sub-Himalayan Zone is delineated by the Main Frontal thrust (MFT) and Main Boundary Thrust (MBT) in south and north respectively. The youngest sediments on the top are the conglomerates. The sandstone and mudstone are dominant in the lower portions. The upward coarsening sequence of the sediments obviously exhibit the time-history in the evolution and growth of the Himalaya during the early Tertiary time. Total width of the zone ranges from 10 Km to 25 Km.

The Siwalik Group in Nepal is composed of three units that are known as lower, middle and upper members. These units can be correlated with the Sub-Himalaya of Pakistan.

3. Lesser Himalayan Zone

The Lesser Himalayas lies in between the Sub-Himalayas and Higher Himalayas separated by the Main Boundary Thrust (MBT) and the Main Central Thrust (MCT) in south and north respectively. The total width of the zone ranges from 60 Km to 80 km. The Lesser Himalaya is made up mostly of the unfossiliferous sedimentary and meta-sedimentary rocks; such as shale, sandstone, conglomerate, slate, phyllite, schist, quartzite, limestone and dolomite. The rocks range in age from Precambrian to Miocene. The geology is complicated due to folding, faulting and thrusting and are largely unfossiliferous.

4. Higher Himalayan Zone

The Higher Himalayan Zone lies in between the Lesser Himalaya and the Tibetan – Tethys Himalaya separated by the Main Central Thrust (MCT) in the south and runs throughout the country. This zone consists of almost 10 km thick succession of the crystalline rocks. This sequence can be divided into four main units, as Kyanite-Sillimanite gneiss, pyroxenic marble and gneiss, banded gneiss, and augen gneiss in the ascending order (Bordet, Colchen & LeFort 1972).

5. Tibetan-Tethys Himalayan Zone

The Tibetan-Tethys Himalayas generally begins from the top of the Higher Himalayan Zone and extends to the north in Tibet. This zone is about 40 km wide and composed of fossiliferous sedimentary rocks such as shale, sandstone and limestone etc. The fossiliferous rocks are well developed in Thak Khola (Mustang), Manang and Dolpa area. The rocks of the Tibetan - Tethys Himalaya consist of a thick and nearly continuous marine sedimentary succession from lower Paleozoic to lower Tertiary age.

4.3.2 Regional Geology

In the upper reaches of the Tamakoshi River, the Lesser Himalayan meta-sediments constitute the footwall of the Main Central Thrust, whilst its hanging wall contains the medium- to high-grade metamorphic rocks and Miocene granites belonging to the Higher Himalaya **(**Figure 4-6).

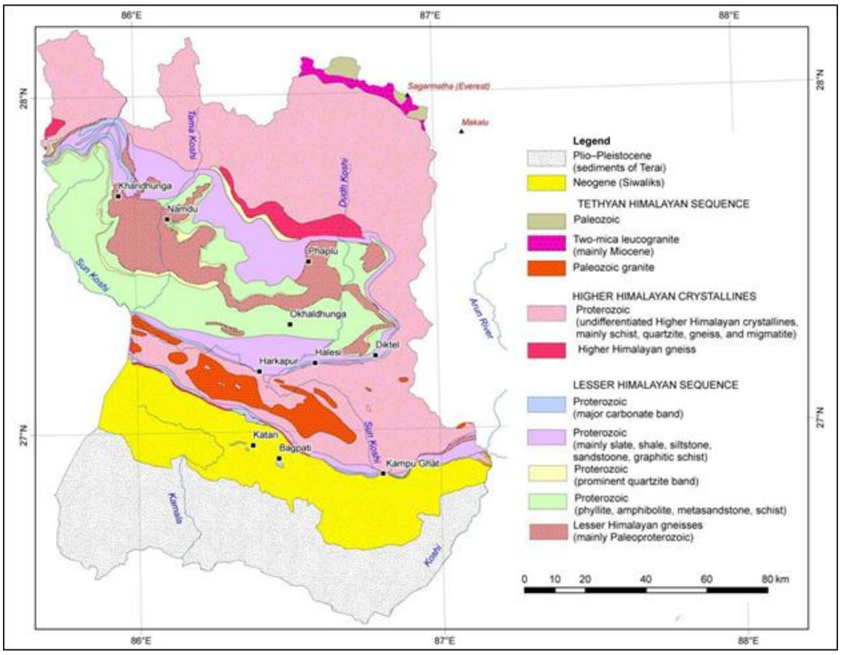

Figure 4-6: Regional Geological Map of Okhaldhunga District Including the Tamakoshi Region *Source*: *Dhital (2015)*

Schelling (1987) divided the Lesser and Higher Himalayan rocks of this area into the following units (Figure 4-7).

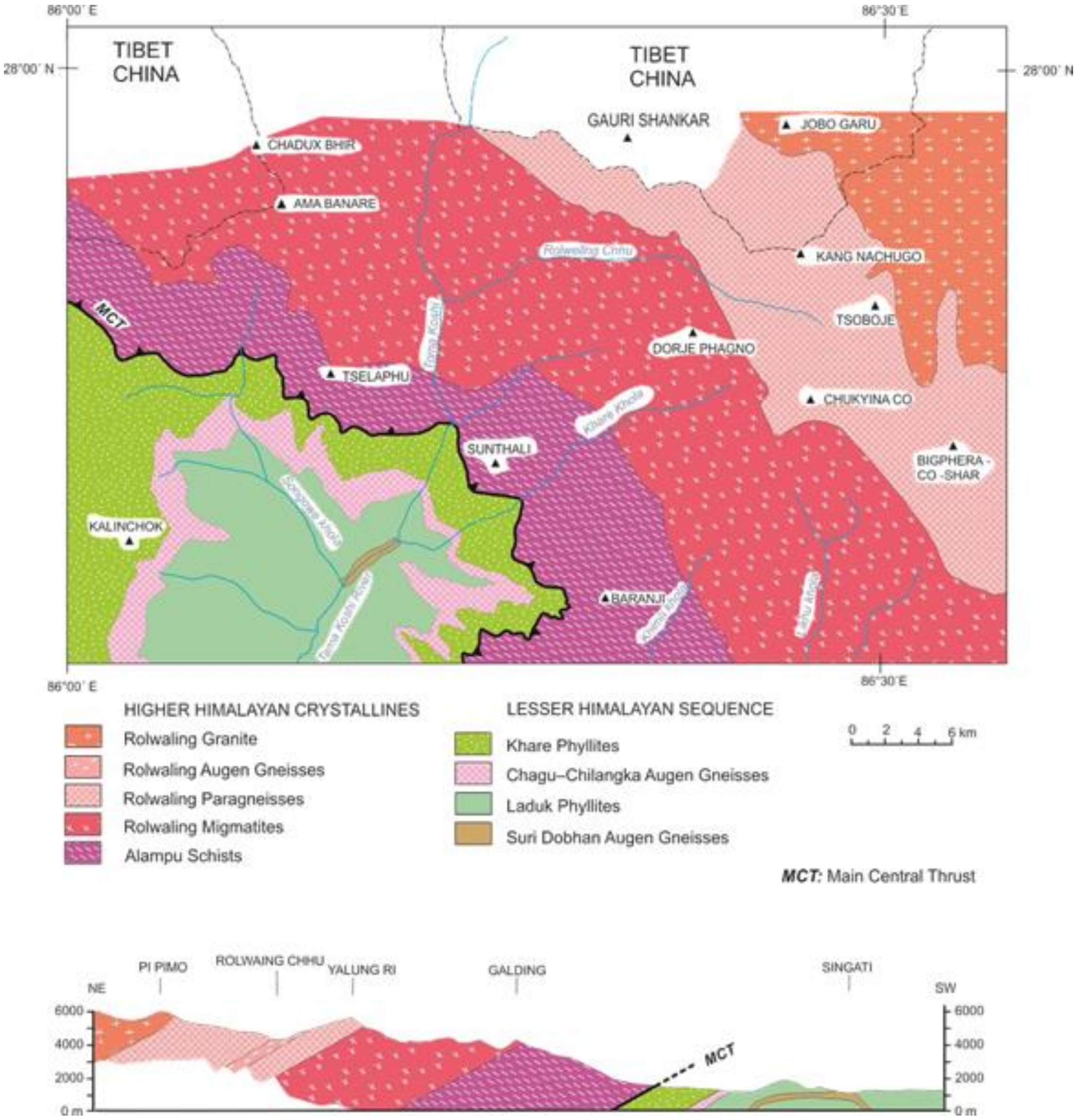

Figure 4-7: Geological Subdivisions of the Upper Tamakoshi Area
Source: Modified from Schelling (1987)

Further upwards, the migmatites give way to about 6,000 m thick Rolwaling Paragneisses. The paragneisses are delimited from above by a sharp intrusive contact with the Rolwaling Granites and below by a gradational contact with the underlying Rolwaling Migmatites (Schelling 1987). This unit also possesses some Biotite Schists, Calcareous Schists, Marbles, and Granitic Gneisses.

Lesser Himalayan Sequence

The Laduk Phyllites constitute a more than 2,000 m thick pile of grey–green pelitic phyllites, psammitic phyllites, chlorite schists, and graphite schists with garnet and actinolite in abundance, especially near the MCT.

These rocks are strongly lineated. Schelling (1987) reported a foliated dolerite dyke, entailing its emplacement prior to the development of foliation in the Laduk phyllites.

The Suri Dobhan augen gneisses crop out in the core of the Tamakoshi Dome, where their thickness exceeds 200 m. The augen gneisses contain muscovite, biotite, quartz, and alkali feldspar. Several centimeter-long feldspar laths are frequently fractured and rotated. A few sporadic bands of quartz-mylonite and sericite-schist are also present within these gneisses.

The Chagu–Chilangka augen gneisses range in thickness from 400 to 700 m and they consist of muscovite, biotite, alkali feldspar, quartz, and tourmaline. Quartz frequently shows bluish tinge. In the gneisses, large (up to 10 cm across) feldspar augen are rotated and fractured with trails of poligonised quartz (Schelling 1987). These augen gneisses have a conspicuous C/S fabric, in which the later formed ‘C’ surface is parallel to the foliation in the Laduk and Khare phyllites.

The Khare phyllites constitute about 2,500 m thick zone of Graphite Schist, Talc Schist, Chlorite– Biotite Schist, as well as Slate, Quartzite, Limestone, and Magnesite. In them, actinolite and tremolite needles define a lineation. Many small- to meso-scale isoclinal as well as open to close asymmetric folds are common in the Khare phyllites.

In the Khare phyllites, the carbonate and quartzite bands (Figure 4.6 & Figure 4.7) are rather discontinuous, and alternating with black graphitic slates and some infrequent amphibolites. The black slates, equivalent to the Khare phyllites, continue eastwards from Kharidhunga and Kalinchok to the Khimti Khola, and Likhu Khola. Thus, they form an uninterrupted belt representing the Benighat slates in Central Nepal, and are invariably present in the footwall of the MCT.

Higher Himalayan Crystallines

The Higher Himalayan rocks override the Khare phyllites rather abruptly, and they are constituted of the following rock types, respectively from bottom upwards.

The Alampu Schists are about 6,000 m thick and consist of well-foliated biotite–garnet schists, calcareous schists, quartzites, hornblende schists, feldspathic gneisses, and augen gneisses. The Alampu schists frequently contain large (up to a few centimeters) garnet porphyroblasts, exhibiting snowball structures, and several centimeter-long blue or green kyanite blades in biotite-rich bands (Schelling 1987).

The Alampu schists are followed upwards by the Rolwaling migmatites, which attain a thickness of 6,000 m. They are predominantly sillimanite-bearing migmatites with muscovite, biotite, garnet, quartz, feldspar, and tourmaline (Schelling 1987). The migmatites are crosscut by a swarm of granitic and pegmatitic veins.

Further upwards, the Migmatites give way to about 6,000 m thick Rolwaling para-gneisses. The para-gneisses are delimited from above by a sharp intrusive contact with the Rolwaling granites and below by a gradational contact with the underlying Rolwaling migmatites (Schelling 1987). This unit also possesses some biotite schists, calcareous schists, marbles, and granitic gneisses.

Sillimanite, garnet, and hornblende are regularly found in the Biotite Schist. There are also a few, up to 600 m thick, Augen Gneiss bands in the para-gneisses.

The Rolwaling granites represent a typical Higher Himalayan leucogranite body (Schelling 1987). The granites are typically medium-grained and they are composed of quartz, K-feldspar, plagioclase, biotite, muscovite, and tourmaline. Garnet is also infrequently found in the Granites, whereas some pegmatite veins contain beryl. The granites cover at least about 50 sq. km area and are from 2.5 to 3 km thick in the Rolwaling valley. The granite body is either undeformed or slightly deformed but the pegmatitic and aplitic veins are strongly folded. Since there are no major feeder dykes, Schelling (1987) inferred that the source of granite lies further north, under the Tibetan Plateau, and the granite moved to its present position along the north-east-dipping Tethyan sediment – para-gneiss contact.

4.3.3 Geology of the Project Area

Detailed geological field mapping of the project area as well as a precise engineering geological study has been carried out during the present stage of study. Study of fold, fault, shear zone, fractures etc. in the project area has also been carried out (refer drawing no. 31-00053-DD-4130-N-1111).

The field mapping was primarily concentrated around the proposed alignment of headrace tunnel, head pond, spillway, powerhouse, tailrace tunnel, surge shaft and four adits. Geological reconnaissance was carried out through different locations and geological map was updated accordingly.

The rock mass in the entire project area is lying on the northern limb of a regional anticlinorium. The tailrace outlet structure is located at the core part. Dip angle at the tailrace outlet is almost horizontal. Rockmass of augen gneiss in TRT outlet and PH hill, in general dips into the North-West i.e. into the hill. The dip angle begins with 5^0 to 10^0 at the tailrace tunnel. The dip angle gradually increases towards north. It is 65^0 to 70^0 at the head pond area.

The rock mass of the project area can be divided into two basic categories. They are medium-to high-grade Higher Himalayan crystalline rock sequence and low-grade metamorphic rocks of the Lesser Himalayan rock sequence. A detailed geological plan and profile and petrographic analysis of the rock samples are presented in Part F3 of this report. The rock sequence of Lesser & Higher Himalaya are delineated by the Main Central Thrust (MCT) near the Tallo Jagat village.

At the proposed powerhouse site, massive augen gneisses occur with sporadic phyllite or chlorite schist partings (Figure 4-8). The foliation is almost horizontal and the rock forms the Tamakoshi Dome. These rocks were called the Suri Dobhan augen gneiss by Schelling (1987).

Figure 4-8: Outcrop of Augen Gneiss in the Powerhouse Hill

The contact between the augen gneisses and chlorite schist is observed near the confluence of the Khare Khola and Tamakoshi River at Suri Dobhan (Figure 4-9).

Figure 4-9: Geological Contact between Augen Gneiss and Chlorite Schist Observed Near Suri Dobhan

The Chlorite schists continues from the above said location towards the Andheri Khola up to the Jamune village (Figure 4-10). About 2860 m length of HRT stretch is anticipated to pass through the chlorite schist rock type.

Figure 4-10: Outcrop of Chlorite Schist observed in Surge Tank Site

Further north is seen another band of augen gneiss. This augen gneiss band is termed as the Chagu-Chilangka Augen Gneiss by Schelling (1987). Presence of this rock sequence in the tunnel stretch is anticipated to be approximately 253 m long.

The rock is followed up-section by chlorite schist, meta-sandstone and quartzite. A succession of medium- to thin-banded pale grey to white, very fine Quartzite occurs towards the top of the last sequence. These rocks belong to the Laduk phylllte of Schelling (1987). This rock sequence in the tunnel stretch is anticipated to be approximately for 901 m length.

To the south of the Tatopani, dark grey to black, parallel-laminated meta-sandstones and graphitic schists exist. In the vicinity of the Tatopani, a first meta-carbonate band is seen (Figure 4-11).

Figure 4-11: Outcrop of Meta-Carbonate Rocks at the Tatopani Waterfall

The band consists of thin to medium beds of grey to dark grey dolomite with magnesite and talc (Figure 4-12).

Figure 4-12: Outcrop of Magnesite Observed Near HRT / Shear Zone Intersection

The meta-carbonate band is alternating with black graphitic schist. The graphitic schist with meta-sandstone bands continues further north and further two more Meta-carbonate bands appear. The grade of metamorphism gradually increases upwards, and the rock gradually changes from chlorite schist to biotite schist and then to garnet schist. This succession of graphitic schists, garnet schists, amphibolites, and quartzites was classified under the Khare Phyllites by Schelling (1987). These rock sequences in the tunnel stretch is anticipated to be approximately for 3026 m length.

Finally, at Tallo Jagat village, kyanite schist and gneiss of the Higher Himalayan succession overrides the Lesser Himalayan sequence along the MCT (Figure 4-13 & Figure 4-14). Schelling (1987) classified the Higher Himalayan crystallines of this section under the Alampu schists and Rolwaling migmatites. This rock sequence in the tunnel stretch is anticipated to be approximately 1215 m long.

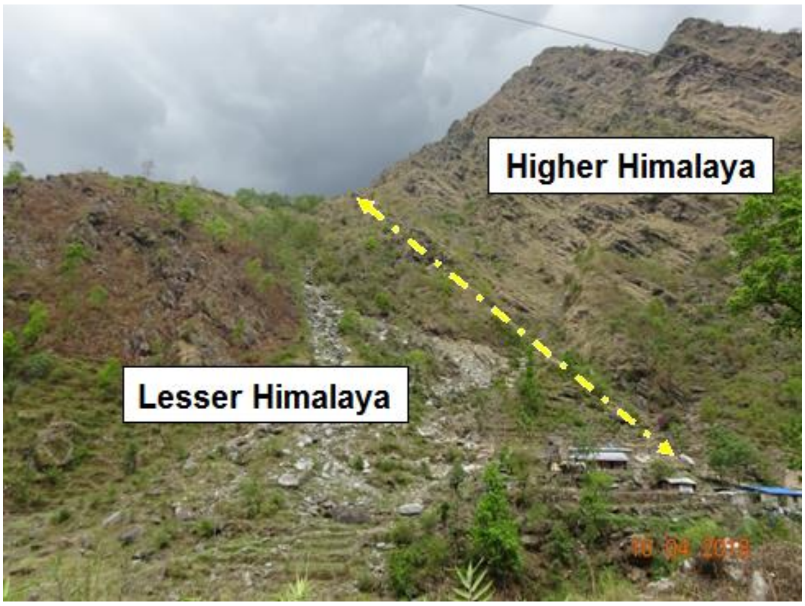

Figure 4-13: Main Central Thrust at Tallo Jagat Delineating Higher & Lesser Himalaya

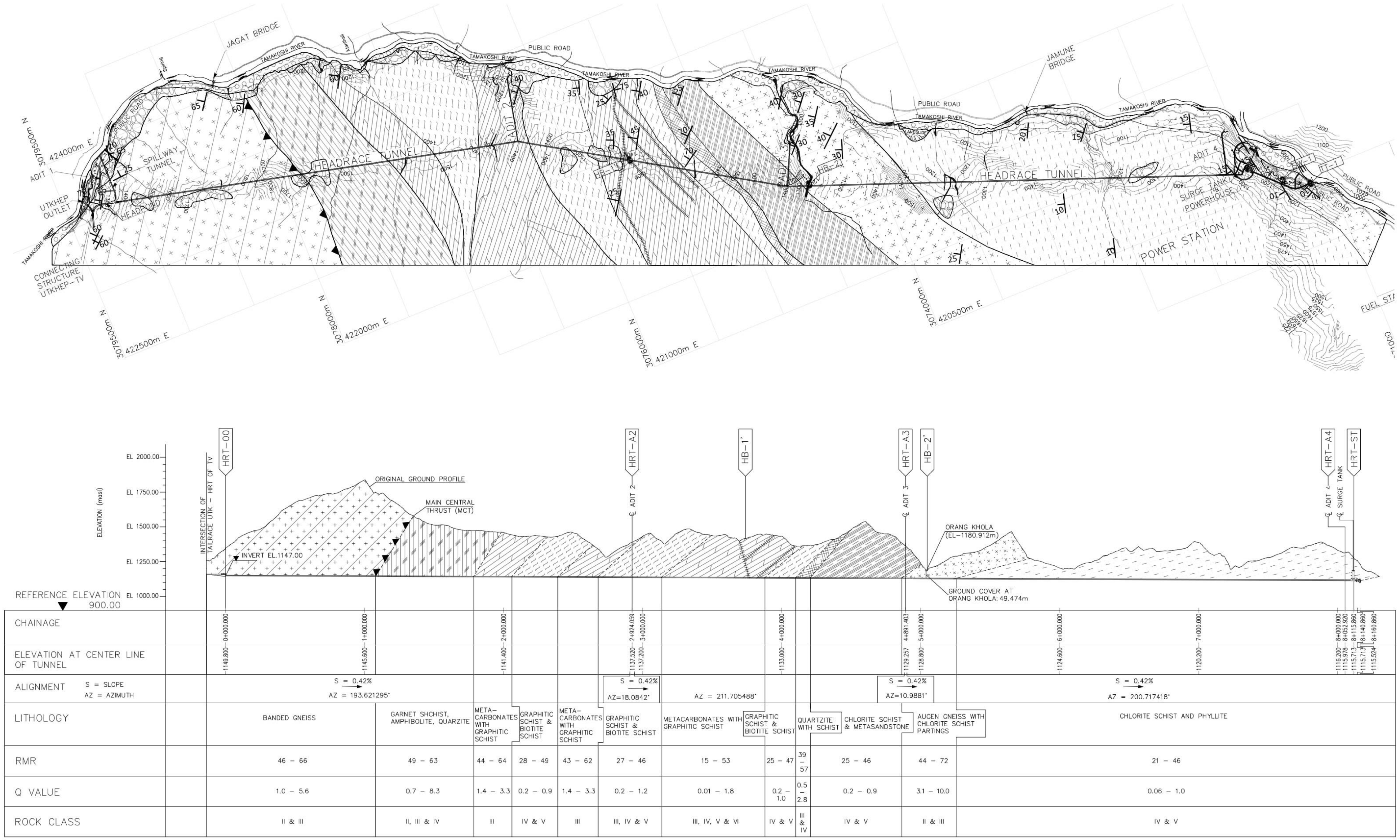

Figure 4-14: Updated Geological Plan and Section along the HRT Alignment of the Tamakoshi V Project

4.3.4 Geological Structures in the Project Area

4.3.4.1 The Main Central Thrust (MCT)

At Tallo Jagat village, kyanite schist and gneiss of the Higher Himalayan succession overrides the Lesser Himalayan sequence along the MCT (Figure 4-15). Schelling (1987) classified the Higher Himalayan crystallines of this section under the Alampu schists and Rolwaling migmatites.

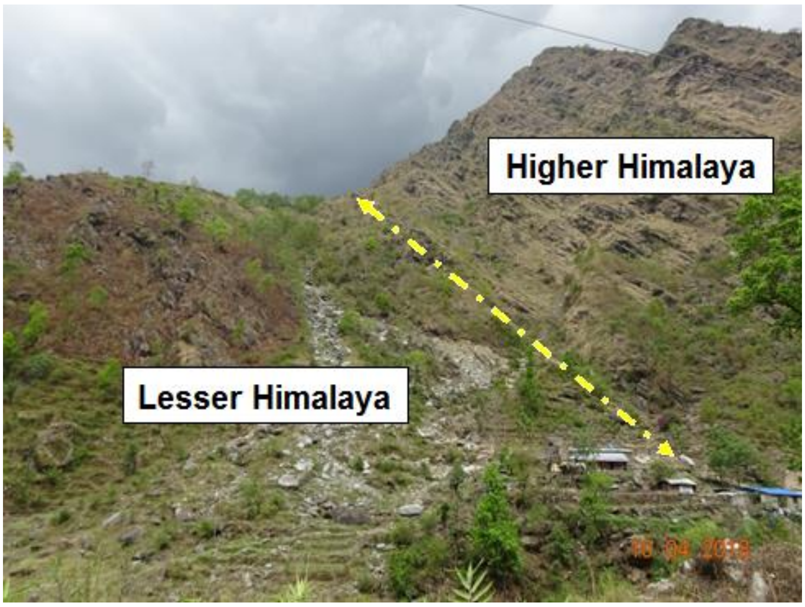

Figure 4-15: MCT Zone Observed at Tallo Jagat Village

Near the MCT zone, the rocks have strike towards North West to South East and have dip angles of 60^0 - 65^0 towards the North.

In the North of MCT zone, topography with high mountain cliffs can be observed. The bedrocks are composed of garnet, amphibolite & kyanite bearing schist and gneiss.

On the other hand, in the south of MCT zone, garnet schist, amphibolite & quartzite are observed near the contact. To the further South, topography of low altitude mountains are observed. These mountains are composed of alternating succession of meta-carbonates and graphitic schist & biotite schist.

Sheared and crushed rocks are not observed on the ground in and around the MCT zone.

4.3.4.2 Shear Zones at Tatopani

The shear zone passes through a deep gorge at Tatopani. The gorge is having width of approximately 60 m and striking east - west with dip angle approximately of 75^0 towards South. The shear joint is one of the main joint sets in this area (Figure 4-16).

Figure 4-16: Shear Zone at Tatopani Striking along the Gorge

Some hot water spots can also be observed near the shear zone (river bed level) at Tatopani village. The shear zone was mentioned as the MCT in the Feasibility report. However, the updated geological study during detailed design has confirmed it as a shear zone.

The shear zone is characterized by sheared and fractured rock mass consisting graphitic schist, biotite schist and dolomite (Figure 4-17). The shear zone also contains some bands rich in magnesite and talc.

Figure 4-17: Outcrop of Angular Breccia Observed on the Way from Kurlan to Tatopani

A small tributary is also observed through the deep gorge. Quantity of water flow during October was approximately 100 liter/ minute. Existence of soft and fractured material in the shear zone anticipate heavy ingress of water in the tunnel during excavation.

4.3.4.3 Landslide at the Jagat Village

A thick landslide is observed at Mathillo Jagat village above the spillway. The landslide scarp is striking from north west to south east with dip angle ranging widely from 25^0-30^0 to 60^0-70^0 towards south.

Material in the sliding zone is fractured and rubblized. The material at places appears like bedrock but orientation of foliations in the detached blocks is dissimilar with the foliation of the bedrock. The blocks are completely disturbed and sometimes appear like minor folds (Figure 4-18).

Figure 4-18: Landslide at Mathillo Jagat Village

4.4 Seismicity in Nepal

4.4.1 Generalf

The Himalayan Range lies at the northern margin of the Indian subcontinent, where the Indian and Eurasian plates (i.e. the Lhasa Block) collided in the Eocene, around 55 million years ago (Argand 1924; Powell and Conaghan 1973; Molnar and Tapponnier 1975). The collision led to extensive crustal shortening and upheaval, resulting into the formation of the quintessential collided orogen – the Himalaya (Figure 4-19). The range is of arcuate shape, extends for over 2400 km in the east– west direction, and is convex towards the south. To date, this youngest and highest mountain range in the world continues to grow.

Himalayan seismicity is attributed to the movement of the Indian plate relative to the Eurasian plate at a rate of about 5 cm per year. When the convergence is locked in some sector, the energy is ultimately released in the form of tremors along the mountain range and in its surroundings (Seeber and Armbruster 1981; Pandey et al. 1995; Bilham et al.1997). Most of the convergence is accommodated within the Himalaya by movement on various thrusts. These active faults are the major source of seismicity in Nepal.

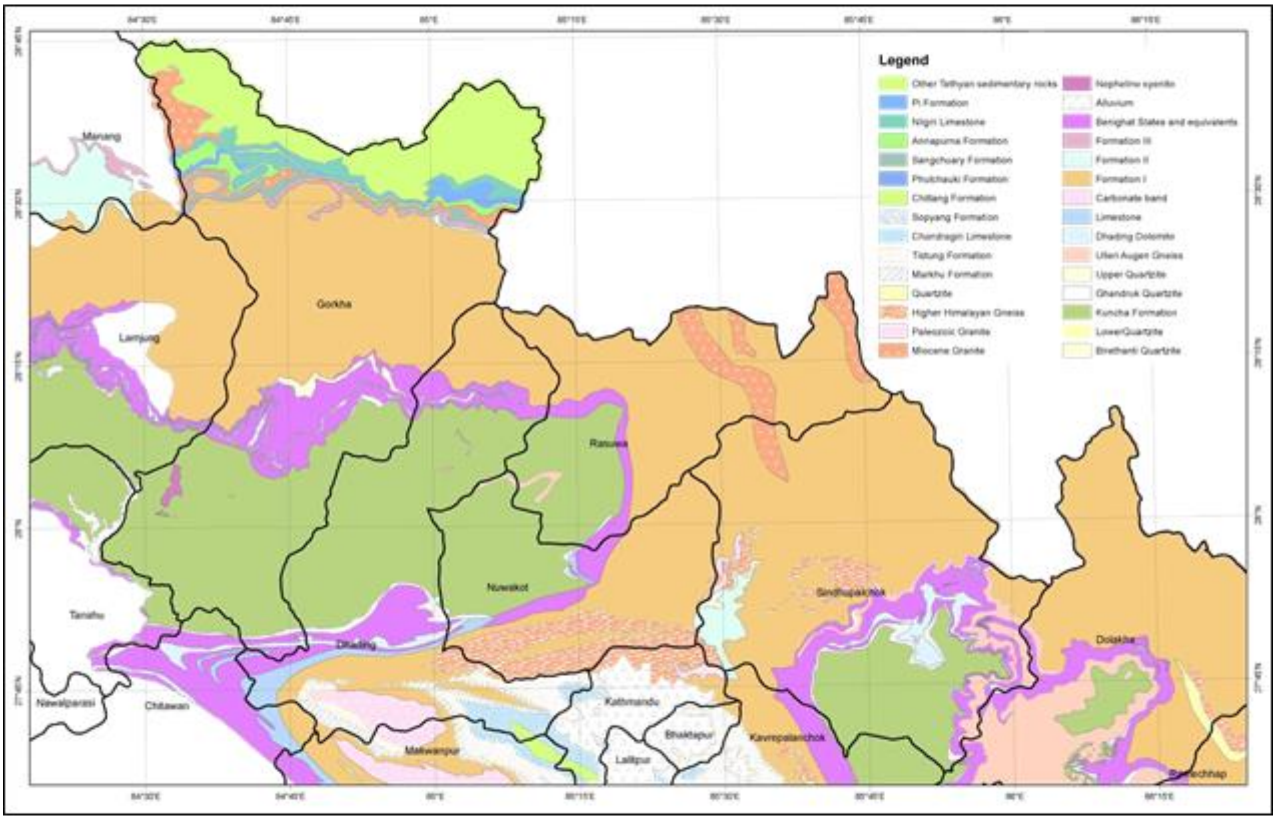

Figure 4-19: Geological Map of the Earthquake-Affected Region of West and Central Nepal
Source: Modified from Dhital (2015)

The investigated area belongs to the Gorkha, Dhading, and Rasuwa districts.

The Himalayan Range and neighboring tract have experienced many and great tremors, namely the Nepal earthquake of 1833 with an estimated magnitude of 7.8 (Bilham 1995), the Shillong earthquake of 12 June 1897 with an estimated magnitude 8.7, the 1905 Kangra earthquake of estimated magnitude 8.0, the 1934 Nepal–Bihar earthquake of estimated magnitude 8.4, and the 1950 Assam earthquake of magnitude 8.5 (Oldham 1899; Middlemiss 1910; Auden 1935; Sharma and Malik 2006). On August 1988 the Udayapur earthquake of magnitude 6.6 struck east Nepal and its focal depth was estimated at 57 km (Dikshit 1991).

Micro seismicity in Nepal is characterized by a narrow belt that follows approximately the front of the Great Himalayan Range. This kind of confined distribution reflects deformation between the upper and lower crusts along the Main Himalayan Thrust under the Lesser and Higher Himalaya (Pandey et al. 1999; Avouac 2003). Though the entire country is seismically active, there is a significant lateral variation. Micro seismic activity is quite intense in east and far-west Nepal, whereas the level of seismic activity is low in west Nepal. The earthquakes are generally shallower than 30 km and they are clustered around a depth of about 20 km. The frequency– magnitude relationship of micro seismicity follows the Gutenberg–Richter law (Pandey et al.

1999), where the b-value varies between 0.75 and 0.95, and does not seem to change significantly over areas.

4.4.2 Earthquake of 25 April 2015

The continuous plate convergence has resulted into a typical seismo-tectonic feature in the Himalaya, which governs the entire seismicity in the region (Pandey et al. 1995). The tectonics of Nepal is controlled mainly by the Main Himalayan Thrust and the other splay thrusts, the Main Central Thrust (which is about 20 Ma old and presently an inactive fault) and other active faults such as the Main Boundary Active Fault System. These thrusts are propagating from north to south throughout the Himalaya and they demarcate the major rock units of the entire Himalaya. According to Pandey et al. (1995), the crustal ramp along the MHT is responsible for the accumulation of the elastic strain, while the southern flat that is locked during the interseismic period acts as a geometrical asperity and accumulates elastic stress. Most of the earthquakes are thus located in this locked part, i.e. along the foothills of the Higher Himalaya. The Mw 7.8 Gorkha earthquake occurred on 25 April 2015 and its epicenter was located at 28°15'07'' N latitude and 84°07' 02'' E longitude, at Barpak, about 80 km NW of Kathmandu (Figure 4-20). Following this main shock, many large aftershocks occurred to the east of the main shock (Figure 4-21 and 4-22). There was also a large aftershocks of Mw 7.3 that occurred in Dolakha on 12 May 2015. The 25 April Gorkha earthquake also occurred along the MHT at the foothills of the Higher Himalaya. This earthquake occurred as a result of faulting near the foothills of the Great Himalayan Range. In this case, the fracture propagated from Barpak to the east, up to Dolakha, and to the south respectively. According to the USGS, the earthquake was caused by a sudden thrust, or release of built-up stress, along the major fault line (the MHT) where the Indian Plate, carrying India, is slowly diving underneath the Eurasian Plate, carrying much of Europe and Asia. The valley of Kathmandu, shifted 3 m to the south in a matter of just 30 seconds.

The Gorkha earthquake of 25 April 2015 and its many strong aftershocks brought about massive destruction, including 8,686 deaths and 16,808 injuries (reported on 25 May 2015 by the Ministry of Home Affairs, the Government of Nepal).

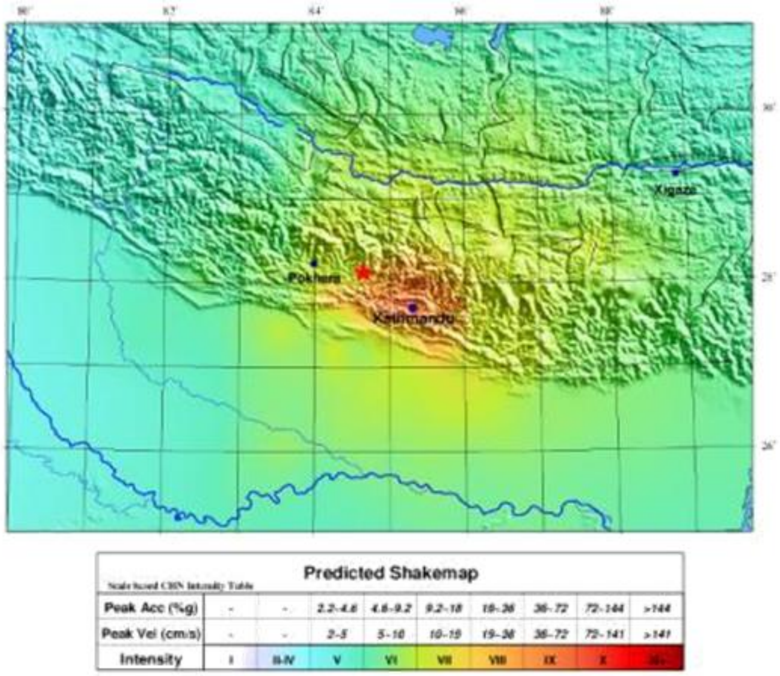

Figure 4-20: Predicted Shake Map of the Gorkha Earthquake (Source: USGS 2015)

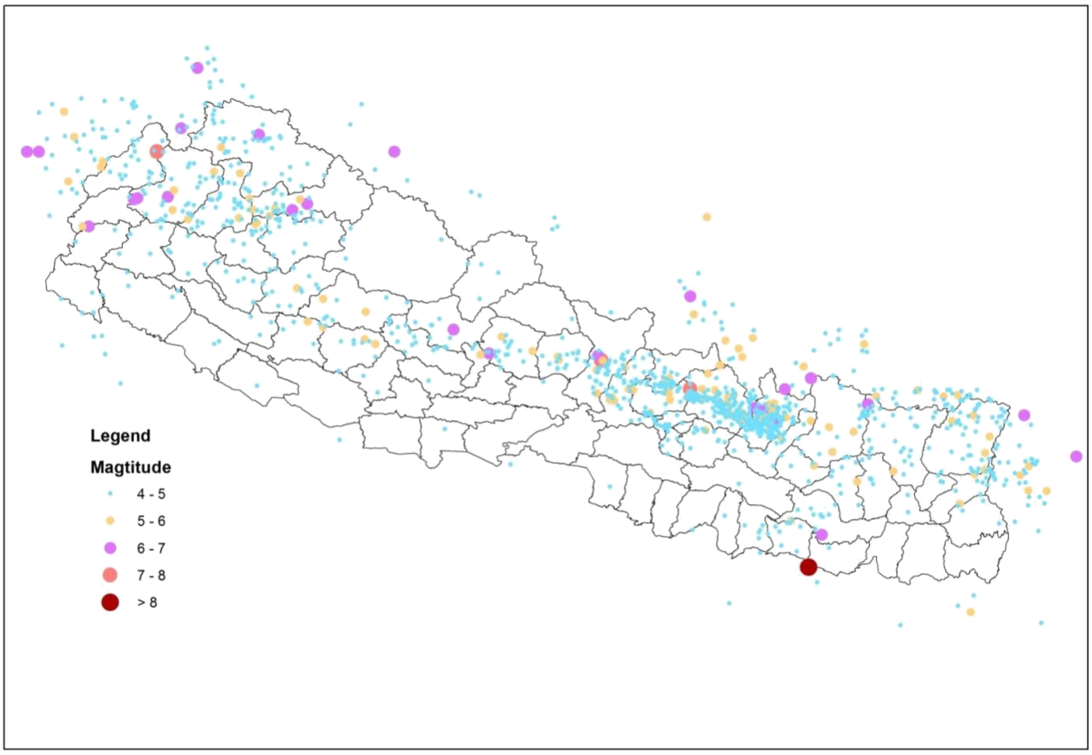

Figure 4-21: Seismicity of Nepal with a Local Magnitude Exceeding 4, Occurred between 1995 and March 2016. *Source: Department of Mines and Geology, Kathmandu*

The 1934 Bihar–Nepal earthquake of Mw 8.4 is also shown.

The 25 April 2015 Gorkha earthquake and its aftershocks in central Nepal ruptured a portion of the Main Himalayan Thrust at 11:56 a.m. NST (06:11:26 UTC) at a depth of approximately 15 km. The low-angle (less than 10 degrees) rupture zone initiated from the northwest and extended due southeast for a length of about 120 km and width of about 60 km. The main shock happened within the Great Midland antifoam, which runs from west to east for more than 800 km throughout the Nepal Himalaya and beyond (Hagen 1969; Dhital 2015). The core of the Great Midland antiform is made up of the Paleoproterozoic Kuncha Formation, consisting of phyllites and meta-sandstones with sporadic amphibolite bands. This monotonous formation is more than 4 km thick and is succeeded by about a few km thick sequences of pink or white quartzites, red-purple slates or schists, and carbonates with a few bands of augen or banded gneiss (equivalent to the Ulleri gneiss in west Nepal). The epicenter of main shock lies within the youngest formation of the area, represented by graphitic schists, equivalent to the Meso- to Neoproterozoic Benighat Slates (Dhital 2015) (see Figure 4-19). These youngest rocks of the Lesser Himalaya occurring in this region are several kilometers thick and constitute the north limb of the Great Midland antiform, which is also known here as the Kunchha–Gorkha Anticlinorium (Pêcher (1977).

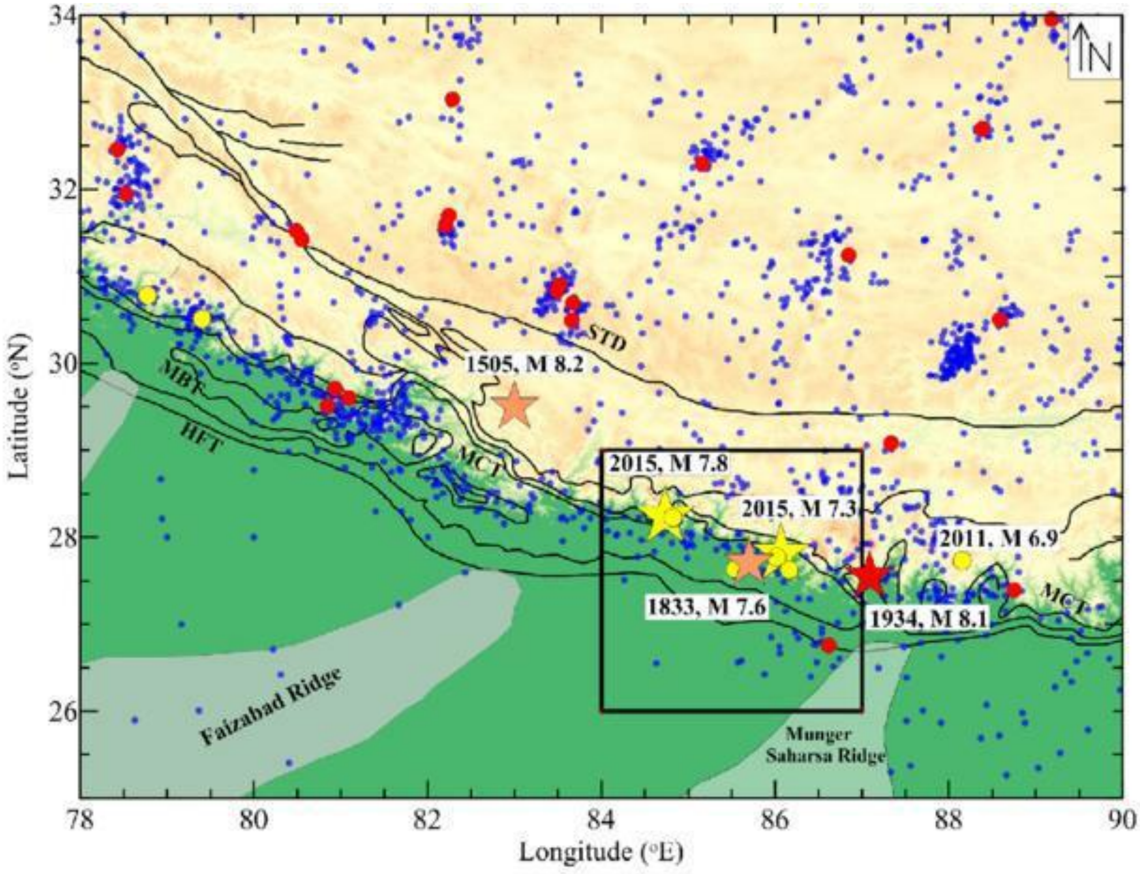

Figure 4-22: Seismo-tectonic Map around the Himalayan Seismic Belt *Source: Arora et al. (2016)*

4.5 Field Investigations

4.5.1 Core Drilling

4.5.1.1 Objective

Main objective of the present study is to confirm characteristic of the sub-surface materials encountered during investigation to the extent of necessary information for designing purpose. Basically the study includes establishment of bedrock, assessment of bedrock and overburden material and also to find out groundwater level. The investigation by core drilling provided most valuable information on the geological sub-surface condition.

4.5.1.2 Scope of Work

The scope of the work for undertaking investigation of geotechnical study of Tamakoshi V Hydroelectric Project can be summarized as follows:

- Diamond core-drilling work within project area

- Proper storing and precise leveling of core samples
- Standard Penetration Test (SPT) / Direct Cone Penetration Test (DCPT) tests in the borehole
- Permeability and Water Pressure (Lugeon) Test in the borehole
- Preparation of core logs and drilling report

4.5.1.3 Executed Boreholes

Five exploratory boreholes have been proposed and completed within the project area during present stage of study. They are TT-1, SW-1, HB-1' & HB-2' and SP1. The drill hole HB-2' is an inclined hole having inclination of 13^0 from vertical. Rests of the drill holes are vertical. See Part F3 of this report for borehole locations.

Total 217 m of drilling has been carried out at present stage of study.

Co-ordinates of boreholes, locations, depth of drilling with orientation are summarized and presented in the table below:

Table 4-3: Details of the Completed Boreholes

Borehole No.	Location	Northing (m)	Easting (m)	Elevation (masl.)	Drilling Depth (m)	Orientation/ Azimuth (^{0})
SW-1	Tailrace Tunnel	420533.811	3071310.384	1043.2	40	Vertical
TT-1	Outlet Structure	420436.546	3071134.996	1012.3	32	Vertical
HB-1'	HRT / Shear zone intersection	422442.949	3075605.578	1412.75	80	Vertical
HB-2'	HRT / Orang Khola intersection	421793.545	3074476.152	1185.217	65	13^0 with vertical
SP-1	Spillway Portal	423888.815	3079126.628	1161.81	60	Horizontal/ N283
Total depth (m)					**277**	

4.5.1.4 Engineering Geological Description of the Drill Cores

Investigation Drilling at Borehole SW – 1 (Total Depth – 40.0m)

Drill hole SW-1 is located at the side of the public road above the proposed tailrace tunnel approximately at the center (Figure 4-23). Drill hole was started on 25th January 2018 and terminated at the depth of 40.0m on 05th February 2018. Purpose of the drilling was to check rock cover above the tunnel and also to know the geological condition in the tunnel.

Figure 4-23: Core Drilling being Operated in Borehole SW – 1

In the drill run, a colluvial deposit was found at the depth from 0 to 32.2m. It is composed of boulders and cobbles of augen gneiss embedded in the matrix of sandy silty soil. Then, bedrock of augen gneiss was found at the depth from 32.2 to 40m. The rock mass is having a gneissic texture with phenocrysts of quartz & feldspar. It is coarse to very coarse grained, having closely to moderately spaced foliation joints, slightly to moderately weathered and medium strong to strong in strength. The rock belongs to high grade metamorphic rock.

On the basis of encountered material, this drilling helped not only to design the tailrace Tunnel but also helped to fix the alignment of the tunnel.

In-situ tests like SPT, DCPT, Lugeon test were carried out in the drill hole SW – 1 sequentially with the drilling. Out of the tests in the entire drill length, SPT test was carried out in 2 sections, DCPT in 8 sections, Permeability test in 10 sections and water pressure test was carried out in 1 section. Water level was recorded at a depth of 25.10 m.

Name of the core drilling rig is Vol – 35 hydraulic rotary drilling machine of capacity 150m depth. In the drill hole, NX – casing was used at the depth from 0 to 25.5m. Drill cores of NX – size were obtained at the depth from 0 to 30m. Likewise, drill cores of NQ – size were obtained at the depth from 30 to 40m.

Investigation Drilling at Borehole TT – 1 (Total Depth – 32.0m)

Drill hole TT-1 is located on side of the public road above proposed outlet structure (Figure 4-24). Drill hole was started on 8th February 2018 and terminated at the depth of 32m on 17th February 2018. Purpose of drilling was to know the geological condition at the foundation level to design the outlet structure.

Figure 4-24: Core Drilling being Operated in borehole TT – 1

In the drill run, fill material was found at the depth from 0 to 2.4m. Then, bedrock of Augen Gneiss was found at the depth from 2.4 to 32m. Rock mass is having a gneissic texture with phenocrysts of quartz & feldspar. It is coarse grained, having closely to moderately spaced foliation joints, slightly to moderately weathered and strong in strength. Rock belongs to high grade metamorphic rock.

Detail geological logging with photographs of the drill cores is incorporated in Part F3 of this Detailed Design Report.

In-situ tests like DCPT, Lugeon test were carried out in the drill hole TT – 1. Out of the tests in the entire drill length, DCPT was carried out in 1 section, permeability test in 1 section and water pressure test was carried out in 5 sections. Water level was recorded at a depth of 29.80 m.

Name of the core drilling rig is Vol – 35 hydraulic rotary drilling machine of capacity 150m depth. In the drill hole, NX – casing was used at the depth from 0 to 3.85m. Drill cores of NX – size were obtained at the depth from 0 to 6m. Likewise, drill cores of NQ – size were obtained at the depth from 6 to 32m.

Investigation Drilling at Borehole HB – 1' (Total Depth – 80.0m)

Drill hole HB-1' is located at HRT / shear zone intersection in a ridge between Adit 2 & Adit 3 (Figure 4-25). Drill hole was started on 07th March 2018. Original borehole got stuck at the depth of 65.3m on 11th April 2018. Another hole was re-drilled 70cm away from original location on 1st July 2018 (Figure 4-26). Although, proposed drilling depth was 170m, hole was discontinued at the depth of 80m on 16th July 2018, as it was deemed not necessary to go into further depth due to existence of similar rock sequence.

Purpose of the drilling was to know the nature and properties of the rock/material in the fractured/brecciated zone and to know groundwater condition at the HRT level.

Figure 4-25: Core Drilling Being Operated in Original Borehole HB – 1'

Figure 4-26: Core Drilling Being Operated in Re-Drilled Borehole HB – 1'

In the drill run, a colluvial deposit is found at the depth from 0 to 2.5m. It is composed of highly weathered boulders and cobbles of micaceous graphitic schist embedded in the matrix of sandy soil. Thereafter, bedrock of meta-carbonates especially siliceous dolomite is found at the depth from 2.5m to 80m. Rock mass is bluish gray, fine grained, slightly to moderately weathered, having closely to moderately spaced foliation joints and medium strong in strength and fractured at places. Groundwater table encountered at a depth of 49.10m.

There was water loss completely in the entire drill run. Core loss has been observed at some sections due to presence of sheared and fragile graphitic schist in between the meta-carbonate rocks.

Detail geological logging with photographs of the drill cores is incorporated in Part F3 of this Detailed Design Report.

Borehole had to be stabilized by cement grouting for many times in sequential way with drilling due to poor geological condition. Therefore, the Lugeon test could not be performed in some sections. However, in the entire drill length, the Lugeon test was performed at 8 sections. Water level was recorded at a depth of 49.1m.

Name of the core drilling rig is Longyear - L & T Mumbai, a hydraulic rotary drilling machine – wireline system, of capacity 300m depth. In the drill hole, NX – casing was used at the depth from 0 to 1.95m. Drill cores of NX – size was used at the depth from 0 to 1.95m. Then, drill cores of NQ – size were used from the depth of 1.95m to 80m.

Investigation Drilling at Borehole HB – 2’ (Total Depth – 65.0m)

Drill hole HB-2’ is located at the HRT / Orang Khola intersection (see following figure). Drill hole was started on 21st May 2018. Hole was completed at the depth of 65m on 6th June 2018. Drill hole is inclined by 13^0 with vertical. Purpose of the drilling was to know the nature and properties of the in-situ rock mass and to check steady state water level (Figure 4-27).

Figure 4-27: Core Drilling being Operated in Drill Hole HB – 2’

Rock mass is having a gneissic texture with phenocrysts of quartz & feldspar. It is coarse to very coarse grained, having closely to thickly spaced foliation joints, un-weathered and strong to very strong in strength. The rock belongs to high grade metamorphic rock. Core recovery in this hole is 100%.

Detail geological logging with photographs of the drill cores is incorporated in Part F3 of this Detailed Design Report.

Lugeon tests are being carried out sequentially with the drilling. In the entire drill length, Lugeon test was performed at 8 sections.

A piezometer pipe at the drill hole has been installed from the depth of 0m to 65m after the drilling completed.

Name of the core drilling rig is Vol – 35 hydraulic rotary drilling machine of capacity 150m depth. The drilling is carried out with NQ – size in the entire run of 65m.

4.5.1.5 Conclusions

1. Project area is located in the rocks of augen gneiss, chlorite schist, graphitic schist, garnet schist, meta-carbonate of the Lesser Himalaya and banded gneiss of the Higher Himalaya.
2. Five exploratory boreholes were completed at site for detailed investigation. They were SW-1, TT-1, HB-1', HB-2' and SP-1. Total length of completed drilling is 277m. The boreholes were executed in the locations of the Spillway Terminal Structure (SP-1), Outlet Structure (TT-1), along the Headrace Tunnel (HB-1' near a fractured zone and HB-2' near Oran Khola) and the Tailrace Tunnel (SW-1).
3. In-situ tests, i.e. Standard Penetration Tests (SPT), Dynamic Cone Penetration Tests (DCPT) and Permeability tests were carried out in the boreholes SW-1 & TT-1 in overburden material of colluvial deposit. In case of SPT and DCPT tests, both were terminated as they could not penetrate to the required depth in 50 blows. Likewise, the coefficient value of the permeability tests represents permeability of low range in both boreholes.
4. Lugeon tests were performed at the boreholes SW-1, TT-1, HB-1' & HB-2' in bedrock. In all cases, water flow was found to be of turbulent type. Likewise, hydraulic conductivity of the rocks represents medium (frequently) to high (occasionally) permeability class.

4.5.2 Electrical Resistivity Tomography (ERT) Tests

4.5.2.1 Introduction

For the geophysical investigation, a fieldwork was carried out during 12th to 18th July 2018. During the fieldwork, measurement of electric current, primary voltage and decay voltage has been carried out to calculate resistivity and chargeability. Chargeability information of sub-surface could be an interpretational aid to 2D-ERT sections in case of ambiguity in the interpretation of electrical resistivity tomography.

4.5.2.2 Physical and Geological Basis of Electrical Resistivity Methods

Different response to the applied voltage between two points in the sub-surface is responded by the flow of electric current in variable quantities. This is the main basis for the response of the applied voltage by different layers and bodies in the subsurface. Result to applied voltage mainly depends on the capacity to conduct electric current by different materials in the subsurface. In 2D-electrical resistivity tomography (2D-ERT) alternating current (AC) of very low frequency is passed through the geological subsurface. The response of current flow on geological formations, such as clay, silt, sand, gravel, boulders, and bedrock are different. By virtue of the different capacity of different materials to conduct electricity it is possible to separate different materials from each other. Electrical resistivity of a material depends both on the matrix (rock and/or granular) and on the salinity of the water and degree of saturation of the pore space. The influence on the current conduction by pore water salinity and its saturation is high in high porosity formations than in low porosity formations.

For metamorphic bedrocks if unaltered have low porosity (usually in the range of 0.1%-3%, rarely reaches to 10%) and very few pores are interconnected. Pores in metamorphic rocks resemble very fine capillary tubes. Below the zone of weathering even if the regional water table is at depth the water will rise in these capillary tubes. In other words, due to fine pore structure these rocks have good moisture holding capacity. Because of low porosity the electrical resistivity of the unaltered metamorphic rocks depends predominantly on the rock matrix and less extend to the mineralization of the water in the capillary. In sedimentary rocks like shale, there are enough interconnected pores in the rock therefore these are very conductive in nature. On the other hand, these rocks have higher moisture holding capacity which increases the conductive nature of the sedimentary rocks. If some kind of shale contains graphite mineral the rock shows high conductive nature.

There is high grade metamorphic rock like gneiss and schist in the investigation site. These rocks comprise several minerals such as muscovite, biotite, sericite, chlorite, and garnet. Similarly, the psammatic gneiss contains abundant minerals of quartz and feldspar. In most places, the bedrocks are fractured and overhanging on the slopes. The Tamakoshi River has made terrace on either bank of river in many places. Bedrocks on the downhill slope are covered with thick colluviums and the river terraces with alluvial deposits. The geophysical investigation areas are on the sloping terrain, where fresh rocks are rarely observed. Instead, wet cultivated land, colluviums, and soil with vegetation are existing along the ERT profiles.

Measurements of decay voltages have been carried out to estimate chargeability of the sub-surface. The measurements have been carried out in nine profiles. Among them, two ERT profiles were taken near to tailrace tunnel; three profiles were taken adjacent to ventilation tunnel. Additionally, four ERT profiles were taken along head race tunnel. The purpose of each profile is different and very crucial in design.

The geographical position of start and end point of each ERT profile line are taken as coordinates in easting and northing format. For this, the national projection coordinate system, Modified Universal Transverse Mercator (MUTM) with central meridian at 84^{o}E.

Table 4-4: Location and Profiles of 2D-ERT Surveys, Tamakoshi V Hydropower Project, Dolakha

Profile No.	Location	Surface length (m)	Starting point		End point	
			Easting	Northing	Easting	Northing
ERT-11	Tailrace Tunnel	174	420498	3071252	420421	3071123
ERT-12	Tailrace Tunnel	144	420420	3071170	420531	3071272
ERT-21	Near to powerhouse	66	420776	3071394	420716	3071398

Profile	Location	Surface	Starting point		End point	
ERT-22	Near to powerhouse	144	420846	3071399	420727	3071407
ERT-23	Near to powerhouse	96	420774	3071348	420778	3071421
ERT-31	Headrace Tunnel	258	422536	3075755	422404	3075542
ERT-32	Headrace Tunnel	258	422441	3075771	422499	3075527
ERT-41	Headrace Tunnel	324	423208	3077685	423278	3077977
ERT-42	Headrace Tunnel	324	423322	3077704	423164	3077959
Total length surveyed = 1,644 m.						

4.5.2.3 Study objectives

Objective of the proposed study of 2D-Electrical Resistivity Tomography (2D-ERT) are:

1. To establish ground profile showing different layers of soil and rock,
2. To find out depth to bedrock
3. To find out jointed, fractured and sheared zone (weakness zone) along the profile
4. To find out possible faulted or thrust zone beneath the surface

4.5.2.4 Conclusions & Recommendations

Detailed interpretation of the results obtained from the 2D-ERT survey has already been discussed. From the discussions and the field observations following conclusions and recommendations are drawn.

Conclusions

1. Geologically, project area lies in the Lesser Himalaya and the Higher Himalaya of the Central Nepal comprising crystalline gneiss, schist, quartzite, and migmatites.
2. In the proposed tailrace tunnel area, there is overburden of dry colluviums, which overlays weak bedrocks. The approximate depth of overburden is about 10 m. The sound bedrock is expected at an elevation of 1012 m., which is exposed on the surface at the end of tailrace tunnel.
3. There is more than 15 m. thick overburden of colluviums and fractured rocks on the sloping terrain in the vicinity of proposed terminal buildings. The sound bedrock could be find beneath the overburden. The alluvial deposits is also thick on the right bank of Tamakoshi River.
4. There are two weak zones along the profile ERT 3-1 and ERT 3-2, which supports the presence of sheared zone or zone of soft materials.
5. There is also weak zone of bedrocks along profiles ERT 4-1 and ERT 4-2. It also indicates the presence of fault gauge or sheared soft rocks.

Recommendations

1. Core drilling is recommended in the identified weak zones along profiles ERT 4-1 and ERT 4-2 to get actual thickness and condition of the rock masses along the tunnel alignment.
2. Core drilling is recommended in the weak rock zone over the tailrace tunnel.

4.6 Test Adit

4.6.1 General Introduction

Test adit portal is located at the Suri Dobhan near right bank of the Tamakoshi River at about 80m downstream from the confluence of the Tamakoshi River & the Khare Khola. Elevation wise, it is located at about 45m below the Charikot – Lamabagar access road. Coordinates of the portal is E = 420817.463 & N = 3071401.003. Invert elevation of the portal is 998.0 masl.

Test Adit is 175.7m long having design shape of inverted ‘D’ and size of 2.5m x 2.5m. Chainage from 125 m to 175.7 m of the adit lies in the proposed powerhouse cavern. Gradient of test adit is dipping inward by 0% to 10%. Excavation of adit commenced on 4th May 2018 and completed on 21st July 2018. See Part F3 of this report for design drawing of the Test Adit.

Portal area is located on the colluvial deposit comprising of boulders, cobbles & gravel of augen gneiss & schist embedded in the matrix of sandy silty soil. Bedrock encountered after about 4m of excavation in the colluvial deposit (Figure 4-28).

Figure 4-28: Test Adit Portal

Test adit is proposed within powerhouse hill at a level of the ventilation tunnel. Its objective is to conduct few in-situ tests to confirm sub-surface geological conditions for design of the powerhouse cavern. Hydrofracture tests, Block Shear tests and Plate Load tests are proposed for In-situ testing.

4.6.2 Construction Methodology

Tunnel excavation was carried out by drilling and blasting method by experienced supervisors and well-trained tunnel crew. Detailed surveying was carried out before beginning of excavation to fix the designated portal location. In the adit the survey was conducted every day as a work cycle to set the adit alignment, to determine adit profile and to ensure the gradient as per design. Proper scaling of the adit profile was done after every round of blasting. The blasted material was stockpiled in the designated area. Drainage system was properly maintained in the adit by constructing side drains and using sump pumps. Geological assessment and logging was carried out frequently. Support system in the adit was followed according to the geological assessment. Support system was executed in a cycle time with excavation.

Following system of support measures were implemented:

1. Scattered rock bolting to secure occasional unstable single blocks
2. Rock bolting in a systematic, pre-determined pattern
3. Rock bolting in a systematic, pre-determined combined with fiber reinforced shotcrete
4. Steel ribs, with or without wire mesh in weakness zones with serious rock stability problems.

4.6.3 Engineering Geological Description

Rock mass is composed of augen gneiss with bands of chlorite schist. Rock contains assemblage of quartz, feldspar, muscovite & biotite. Phenocrysts of quartz & feldspar are frequently observed. Rock mass is coarse grained, moderately strong to strong, has rough & slightly undulated surfaces, having very closely to moderately spaced foliation joints. Joints are having infillings of silt, clay & iron stain of thickness from >1 to 3 mm. Chlorite schist is weak to moderately strong, wavy & undulated, having closely spaced foliation joints (Figure 4-29). See Part F3 of this Detailed Design Report for as built engineering geological drawing of the test adit.

Figure 4-29: Rock Mass observed in the Test Adit

Rock mass is slightly to moderately weathered between chainage 0 m & 90 m. However, fresh to slightly weathered rock mass is observed between chainage 90 m & 175.7 m.

Four well developed joint sets including foliation joint with few random joints are observed. Dip direction / dip angle of the joints are as follows: JS0 (foliation joints): 05^0 – 20^0/320^0 - 360^0, JS1: 65^0 – 85^0/ 030^0 - 060^0, JS2: 60^0 – 80^0/ 150^0 - 180^0 & JS3: 65^0 – 85^0/ 190^0 - 220^0 and random joints: 70^0/ 070^0- 085^0, 70^0 - 80^0 / 120^0 - 145^0 & 75^0/ 255^0 - 265^0.

During excavation, rock surfaces were damp to wet between chainage 0 m & 60 m and chainage 120 m & 175.7 m. Flowing groundwater was encountered between chainage 60 m & 120 m.

The RMR in the adit ranges from 29 to 56. Tunneling Quality Index (Q) ranges from 0.08 to 4.0. Rock mass class is expected to be in a range of III, IV & V.

4.6.4 Support System

Excavation of the test adit was started on 4th May 2018 and completed on 21st July 2018. Total working days in the adit was 78. Per day rate of excavation was 2.25 m.

Ch. 0 to Ch. 125 of adit was supported with 5 cm to 8 cm thick fiber reinforced shotcrete (FRS) and pattern rock bolts. The rock bolts were of length 2.5 m having diameter of 25 mm. 4 to 5 number of rock bolts were installed in a section as pattern bolting (Figure 4-30).

Figure 4-30: Installed Support in Test Adit

Ch. 125 to Ch. 175.7 of the adit was supported with pattern rock bolts.

For control blasting, fore poles were installed at few sections where deemed necessary. In such conditions, installed fore poles were of length 4 m having diameter of 25 mm. 4 to 6 numbers of fore poles were installed at a time.

Steel structures were also installed from Ch. 0 to Ch. 12, Ch. 14 to Ch. 20.7, Ch. 65 to Ch. 74.3, and Ch. 77 to Ch. 83 as an additional support to protect from rock fall from the crown. Geological overbreak occurred due to presence of locally developed sheared and fracture zones from said sections during excavation.

Adit portal was supported with pattern rock bolts and fiber reinforced shotcrete. Length of the rock bolts for the portal was 3m.

4.6.5 Conclusion

i. Rock mass is composed of fresh to moderately weathered augen gneiss with bands of Chlorite Schist.

ii. Four well developed joint sets including foliation joint with few random joints are observed.

iii. Rock surfaces were damp to wet in general but flowing groundwater was encountered between chainage 60 & 120m.

iv. Based on 3D geological logging of test adit, rock mass classes observed as per RMR & Q system and respective rockmass class percentage in test adit are given in Table 4-5, Table 4-6, Table 4-7 and Table 4-8.

Table 4-5: Observed RMR and Rock Mass Class in Test Adit

Sr. No.	Chainage (m)	RMR (Observed)	Rock Mass Class
1	0 – 47.5 120 – 130 143 – 150 150 – 175.7	41 - 56	III
2	47.5 – 60 60 – 120 130 - 143	26 - 40	IV

Table 4-6: Observed Q and Rock Mass Class in Test Adit

Sr. No.	Chainage (m)	Q (Observed)	Rock Mass Class
1	0 – 47.5 120 – 130 143 – 150 150 – 175.7	1 - 4	Poor
2	47.5 – 60 60 – 72.5 85 - 120 130 - 143	0.5 - 1	Very Poor
3	72.5 - 85	0.08 – 0.5	Ext. Poor – Very Poor

Table 4-7: Observed Rock Mass Class Percentage (RMR Basis) in Tailrace Tunnel

Sr. No.	Rock Mass Class	Observed Percentage
1	III	51
2	IV	49

Table 4-8: Observed Rock Mass Class Percentage (Q Basis) in Tailrace Tunnel

Sr. No.	Rock Mass Class	Observed Percentage
1	Poor	51
2	V Poor	42
3	Ext. Poor	7

4.7 Laboratory Testing

4.7.1 Introduction

Rock core samples were selected for laboratory testing from the five bore holes performed at different locations in the project to carry out geotechnical analyses of tunnels, caverns, and other civil components in the Tamakoshi V Hydroelectric Project. The relevant boreholes were SW-1, TT-1, HB-1’ & HB-2’ and SP-1 (see Part F3 of this report for locations of the boreholes). Likewise, some core samples were selected for laboratory tests from the borehole ST-2 carried out earlier during feasibility study by Nepal Electricity Authority (NEA) at the proposed Surge Tank site. Core samples were selected by the Consultant’s geologist and geotechnical experts in presence of NEA’s representative engineer. Few of the lab tests were carried out in Pashupati Drilling and Geotechnical Services, Bhaktapur and few of the samples were taken to AECS Laboratory, Noida, India.

The following laboratory tests were conducted on the rock core samples:

1. Elastic Parameters-Modulus of Elasticity & Poisson’s Ratio
2. Indirect Tensile Strength
3. Triaxial Compression Test
4. Point Load Test
5. Uniaxial compressive strength (UCS)

This report covers the test procedures followed and results of laboratory tests conducted.

4.7.2 Experimental Investigations for Rock

4.7.2.1 Preparation of Test Specimens

Rock core specimen for various laboratory tests were prepared in accordance with relevant provisions of IS: 9179-1979. Rock cores of 47.6 mm dia. were cut to proper length, meeting the requirements of the requisite length/diameter ratio and their ends planed and polished using polishing and lapping machine.

Salient features of the test procedures used for various laboratory tests conducted on the rock core specimens are briefly described below.

Modulus of Elasticity & Poisson’s Ratio

Test was conducted as per IS: 9221-1979 “Method for the Determination of Modulus of Elasticity & Poisson’s Ratio of Rock Materials in Uniaxial Compression”. The test specimens comprised right circular cylinders with length to diameter ratio of 2. Axial and circumferential deformations were determined from data obtained by electrical resistance strain gauges. Tangent Modulus and Poisson’s Ratio were determined at 50% of the ultimate stress. Related test results are presented in Table 4-9.

Tensile Strength (Brazilian Test)

Tests were carried out as per IS: 10082-1981 “Method of Test for Determination of Tensile Strength by Indirect Tests on Rock Specimens”. Diameter of the rock core specimen for the Brazilian Tests was 54/55/58mm & thickness approximately equal to half the diameter. The test specimen was wrapped around its periphery with one layer of adhesive paper tape & mounted squarely in the test apparatus such that the curved platens loaded the specimen diametrically with the axes of rotation for specimen & apparatus being coincident. Loading rate of 200 N/s was applied & the maximum load on the specimen was recorded in Newtons. It was ensured that for accuracy of the test, the mode of failure was vertical splitting starting from the centre towards loading points. Tensile strength of the rock was calculated from the following expression:

$$q_t = \frac{2P}{\pi D t}$$

Where

q_t	=	Tensile strength in MN/m^2;
P	=	Load at failure in Newton;
D	=	Diameter of test specimen in mm; and
t	=	Thickness of test specimen measured at the centre in mm

Then, specimens were tested from each drill hole to arrive at a representative value of tensile strength of a rock sample. The test results are furnished in Table 4-10.

Triaxial Compression Test

The test was carried out as per IS: 13047-1991 “Method for Determination of Strength of Rock Materials in Triaxial Compression”. The test specimens comprised of right circular cylinders with length to diameter ratio of 2. The rock core specimens were tested at different confining pressures. Mohr’s shear strength envelopes were plotted for determining shear strength parameters, cohesion intercept, C and angle of shearing resistance ϕ. The test results are presented in Table 4-11.

Point Load Strength Index of Intact Rock – IS 8764 : 1998

The point load test is an index test for strength classification of rock materials. This test should be used to predict the uniaxial compressive strength of un-weathered rock for the purpose of rock mass classification. This test is also used for the determination of tensile strength of the intact rock.

This test is carried out in the laboratory in two ways:

(A) Diametral Test

Core specimens with length diameters ratio greater than 1.5 are suitable for diametral testing.

$I_{s\,(50)}$ = $\frac{P}{D^{1.5}\sqrt{D_{50}}}$ in MPa

Where,

$I_{s\,(50)}$ = Point load strength index in MPa (for the standard core sizes)

P = Failure load in N

D = Core diameter in mm and,

D_{50} = Standard core diameter

(B) Axial Test

Core specimens with length/diameter ratio of 0.3 to 1.0 are suitable for axial testing.

$I_{L\,(50)}$ = $\frac{P}{(DW)^{0.75}\sqrt{D_{50}}}$ in MPa

Where,

$I_{L\,(50)}$ = Point load lump strength index in MPa

P = Peak load in N at failure

(DW) = The minimum cross-sectional area passing through point loads in mm^2

D_{50} = Standard size of lump (50mm)

D = Distance between point loads in mm, and

W = Average width of minimum cross sectional area in mm

Uniaxial Compressive Strength (UCS)

q_c = 22 $I_{s\,(50)}$ in MPa

Where,

q_c = Uniaxial compressive strength in MPa

$I_{s\,(50)}$ = Corrected point load strength, MPa

Uniaxial Tensile Strength (UTS)

q_t = 1.25 $I_{s\,(50)}$ in MPa

Where,

q_t = Uniaxial tensile strength in MPa

$I_{s\,(50)}$ = Corrected point load strength, MPa

<u>Unconfined Compressive Strength (UCS) : IS 9143 – 1979</u>

Unconfined compressive strength also named as uniaxial compressive strength (UCS) test is primarily an index test for strength classification of rock materials.

Length to diameter ratio of cylindrical specimen shall be 2 to 3. The test is carried out in the stress rate of 0.5 MPa/s to 1.0 MPa/s.

q_c = $\frac{P}{A}$ M in MPa

Where,

q_c = Unconfined compressive strength, MPa

4.7.3 Test Results

Results of all the tests on rock core samples are presented in the Table 4-9, Table 4-10, Table 4-11, Table 4-12, Table 4-13 and Table 4-14.

Table 4-9: Test Results of Uniaxial Compressive Strength with Modulus of Rock Core Samples (Saturated Condition)

S.No	Bore Hole No.	Core No.	Depth (m)	Diameter of Rock Core (mm)	Length/Dia. Ratio	Load (kN)	Modulus of Elasticity, E(GPa)	Poisson's ratio, *v*	Uniaxial Compressive Strength (MPa)	
									Observed Value	Corrected Value
1	BH-1	69	-	47.60	2.00	56.90	20.06	0.14	31.99	31.99
2	BH-2	51	-	47.60	2.00	50.20	32.59	0.14	28.22	28.22
3	-	12	-	47.60	1.95	67.70	48.42	0.07	38.06	37.94
4	-	33	-	47.60	1.91	112.40	32.24	0.15	63.20	62.81

Table 4-10: Test Results of Tensile Strength (Brazilian Test) of Rock Core Samples (Saturated Condition)

S.No.	Bore Hole No.	Core No.	Depth(m)	Diameter of Rock Core (mm)	Tensile Strength (MPa) (Brazilian Test)
1	ST-2	81	-	47.60	1.19
2	BH-1	69	-	47.60	5.27
3	BH-2	51	-	47.60	1.94
4	-	33	-	47.60	5.44

Table 4-11: Results of Triaxial Shear Strength Parameters of Rock Core Samples (Saturated Condition)

S.No.	Bore Hole No.	Core No.	Shear Strength Parameters	
			C (MPa)	ϕ (Degrees)
1	ST -2	82,83,84	2.06	41
2	BH-1	65	4.06	43
3	BH-2	58	2.60	43
4	-	34	7.41	43

Table 4-12: UCS and Point Load Test Results

Pashupati Drilling & Geo- Technical Services Pvt. Ltd.

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UNCONFINED COMPRESSIVE STRENGTH TESTS (IS - 9143)

Project: Geotechnical Investigation Works For Tamakoshi V HEP **Date** : Aug. 2018
Client : Nepal Electricity Authority (NEA), Nepal
Consultant : LAHMEYER International, Germany
Location: Singati, Dolakha District

S. No.	Drill No.	Sample No.	Sample Type	Sample Height, Ht. (mm)	Width, Dia. (mm)	Surface Area, A (mm^2)	Weight, Wt. (gm)	Density, gm/cm^3 (ρ)	Load Failure, P (KN)	UCS (MPa)	Failure Types	Remarks
1	SW - 1	11	Dry	92.00	47.00	1734.07	423.00	2.65	81.00	46.71	M	
2	SW - 1	13	Dry	91.00	47.00	1734.07	429.50	2.72	69.00	39.79	M	
	AVERAGE									**43.25**		
3	HB - 1	66	Dry	90.00	47.00	1734.07	457.50	2.93	75.00	43.25	S/M	
4	HB - 1	67	Dry	103.00	47.00	1734.07	525.50	2.94	50.00	28.83	S/M	
5	HB - 1	68	Dry	131.00	47.00	1734.07	664.00	2.92	61.00	35.18	S/M	
	AVERAGE									**35.75**		
6	HB - 2	52	Dry	132.00	47.00	1734.07	631.00	2.76	16.00	9.23	S/F	
7	HB - 2	55	Dry	112.00	47.00	1734.07	508.00	2.62	29.00	16.72	S/F	
8	HB - 2	56	Dry	122.00	47.00	1734.07	591.50	2.80	34.00	19.61	F	
	AVERAGE									**15.19**		
9	TT - 1	31	Dry	122.00	47.00	1734.07	576.50	2.73	95.00	54.78	F	
10	TT - 1	32	Dry	120.00	47.00	1734.07	573.00	2.75	71.00	40.94	S	
	AVERAGE									**47.86**		

Mode of Failure :

S = Shear Failure
M = Matrix Failure
F = Failure due to fracture/Bedding
V = Void fracture
C = Combination

Table 4-13: Result of Point Load Test (Sheet 1)

Pashupati Drilling & Geo- Technical Services Pvt. Ltd.

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Tel : 00977-1-5182310, 9851026210 E-mail: pashupatidrilling@gmail.com

POINT LOAD TEST RESULTS (IS - 8764)

Project: Geotechnical Investigation Works For Tamakoshi V HEP

Date : Aug. 2018

Client : Nepal Electricity Authority (NEA), Nepal

Test Type : Diametral

Consultant : LAHMEYER International, Germany

Location: Singati, Dolakha District

S. No.	Drill No.	Sample No.	Sample Type	Sample Height, D (mm)	Width, W (mm)	De^2 (mm^2)	Load, P (KN)	Is (MPa)	Is (50) (MPa)	UCS, q_c (MPa)	UTS, q_t (MPa)	Remarks
1	SW - 1	13	Dry	47	68		7.44	3.368	3.265	71.840	4.082	
2	SW - 1	21	Dry	47	77		7.81	3.536	3.428	75.412	4.285	
3	SW - 1	23	Dry	47	60		8.28	3.748	3.634	79.951	4.543	
4	SW - 1	24 A	Dry	47	47		7.48	3.386	3.283	72.226	4.104	
	AVERAGE							**3.51**	**3.40**	**74.86**	**4.25**	
5	HB - 1	70	Dry	47	65		1.82	0.824	0.799	17.574	0.999	
6	HB - 1	71	Dry	47	60		1.98	0.896	0.869	19.119	1.086	
7	HB - 1	72	Dry	47	60		2.49	1.127	1.093	24.043	1.366	
	AVERAGE							**0.95**	**0.92**	**20.25**	**1.15**	
8	HB - 2	52	Dry	47	35		3.91	1.770	1.716	37.754	2.145	
	AVERAGE							**1.77**	**1.72**	**37.75**	**2.15**	
9	TT - 1	31	Dry	47	56		5.13	2.322	2.252	49.535	2.814	
10	TT - 1	41	Dry	47	88		9.47	4.287	4.156	91.441	5.196	
11	TT - 1	43	Dry	47	85		8.45	3.825	3.709	81.592	4.636	
12	TT - 1	45 A	Dry	47	82		6.22	2.816	2.730	60.059	3.412	
	AVERAGE							**3.31**	**3.21**	**70.66**	**4.01**	

Pashupati Drilling and Geo- Technical Services Pvt. Ltd.

Table 4-14: Result of Point Load Test (Sheet 2)

Pashupati Drilling & Geo- Technical Services Pvt. Ltd.

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Tel : 00977-1-5182310, 9851026210 E-mail: pashupatidrilling@gmail.com

POINT LOAD TEST RESULTS (IS - 8764)

Project: Geotechnical Investigation Works For Tamakoshi V HEP — **Date** : Aug. 2018
Client : Nepal Electricity Authority (NEA), Nepal — **Test Type** : Axial
Consultant : LAHMEYER International, Germany
Location: Singati, Dolakha District

S. No.	Drill No.	Sample No.	Sample Type	Sample Height, D (mm)	Width, W (mm)	De² (mm²)	Load, P (KN)	Is (MPa)	Is (50) (MPa)	UCS, q_c (MPa)	UTS, q_t (MPa)	Remarks
1	SW - 1	22	Dry	54	47	3233.12	15.76	4.875	5.617	123.567	7.021	
2	SW - 1	24 B	Dry	48	47	2873.89	7.10	2.471	3.019	66.425	3.774	
	AVERAGE							**3.67**	**4.32**	**95.00**	**5.40**	
3	HB - 1	61	Dry	52	47	3113.38	8.19	2.631	3.089	67.954	3.861	
4	HB - 1	62	Dry	44	47	2634.39	6.47	2.456	3.135	68.970	3.919	
5	HB - 1	63	Dry	38	47	2275.16	10.67	4.690	6.442	141.719	8.052	
6	HB - 1	64	Dry	38	47	2275.16	4.73	2.079	2.856	62.824	3.570	
	AVERAGE							**2.96**	**3.88**	**85.37**	**4.85**	
7	HB - 2	54	Dry	53	47	3173.25	2.76	0.870	1.012	22.255	1.265	
8	HB - 2	55	Dry	46	47	2754.14	7.02	2.549	3.182	70.006	3.978	
9	HB - 2	56	Dry	42	47	2514.65	4.74	1.885	2.463	54.180	3.078	
10	HB - 2	57	Dry	33	47	1975.80	5.41	2.738	4.036	88.790	5.045	
	AVERAGE							**2.01**	**2.67**	**58.81**	**3.34**	
11	TT - 1	38	Dry	38	47	2275.16	4.73	2.079	2.856	62.824	3.570	
12	TT - 1	42	Dry	61	47	3652.23	14.91	4.082	4.426	97.369	5.532	
13	TT - 1	44 A	Dry	56	47	3352.87	10.25	3.057	3.459	76.099	4.324	
14	TT - 1	44 B	Dry	51	47	3053.50	6.80	2.227	2.640	58.089	3.300	
15	TT - 1	45 B	Dry	43	47	2574.52	9.84	3.822	4.935	108.575	6.169	
16	TT - 1	46	Dry	56	47	3352.87	11.24	3.352	3.793	83.449	4.741	
	AVERAGE							**3.10**	**3.68**	**81.07**	**4.61**	

Pashupati Drilling and Geo- Technical Services Pvt. Ltd.

4.8 Conclusions and Recommendations

4.8.1 Conclusions

Based on site approach, seismicity, surface geological mapping and explored sub-surface geological conditions, conclusion and recommendation are as below:

1. Entire project and its most of the structures are accessible through the existing highway.

2. Project lies within the high seismic zone but, as all the structures are underground threat to their damage during any such kind of incidence is minimal. Adequately design criterion are taken as per the Maximum Credible Earthquake (MCE) & Design Basis Earthquake (DBE) values to mitigate the seismic effect on structures.

3. Project structures are adequately explored through sub-surface investigations including in-situ testing's during Feasibility Study (FS) and at present stage. Quantum of investigations is mentioned as below:

   i. Drilling: FS stage for about 270 m depth and during present stage for about 277.6 m depth.

   ii. Electrical Resistivity Tomography: FS stage for about 2185 m and during present stage for about 1788 m.

   iii. Test Adit: During present stage 175.7 m

   iv. Test Pits: During FS stage 13 nos.

4. Basis the design as per RMR and Q-system of rock mass classification, expected rock mass classes across the underground structures of project are mentioned below:

Table 4-15: Expected Rockmass Class Percentage Across the Underground Structure

Sr. No.	Rock Mass Class (Q-System)	Rock Mass Percentage % (Tentative)	Rock Mass Class (RMR-System)
1	Fair	25	II - III
2	Poor	45	III - IV
3	Extremely Poor	30	V

4.8.2 Recommendations

Below stated are some recommendations prepared for the Tamakoshi V Hydroelectric Project considering the geological condition of the site:

1. The drillings carried out from the Test Adit in the location of the powerhouse cavern did not allow to conclude on the in-situ rock conditions, since only highly disturbed drill cores were obtained. The geological conditions in this location should be clarified at highest priority by a suitable additional investigation program, which may include additional drill holes, borehole scanning, and other investigations as deemed appropriate.
2. River water sample from Tamakoshi is recommended for chemical test at three different locations viz. near head pond area, downstream to confluence of Orang Khola and nearby powerhouse location to assess its suitability with cement and aggregates for construction purpose.
3. ERT completed in the tailrace tunnel alignment close to the outlet structure, one of the tomogram profiles revealed weak bedrock in the TRT alignment. Therefore, an investigation drilling is advisable to confirm the bedrock along the TRT alignment at a location of weak rock condition as per the tomogram, during construction stage.

5 PROJECT OPTIMIZATION STUDIES

5.1 Introduction

One of the key activities foreseen for the initial phase of the project studies was the review of the overall project layout, and more specifically the location of the outlet of the power waterways, which is also addressed in the ToR. Placing the outlet in different possible locations showed to have a significant influence on the project's gross head, and is thus of paramount importance for the project's layout.

In addition, the Client informed the consultant in the early days of the services that the license boundary of the project had recently been changed from the location stated in the Contract Agreement to a new location some 900 m further downstream on the Tamakoshi River. This principally allowed to place the outlet even further downstream, and to look into additional layout options for the project.

Due to the basic project configuration linearly arranged in the mountains of the right abutment of Tamakoshi River the layout options are generally limited. Variations are confined to the mentioned shifts of the outlet location, shifts of the powerhouse location and minor adjustments of the alignment of the approx. 8 km long headrace tunnel. These variations were investigated, and the results reported in respective project reports. The reports present the considerations relevant for several project layouts from the perspectives of project setting, construction, energy generation and other performance parameters.

At the date of release of the above reports most of the field investigation works which were foreseen to be carried out under the above-referred contract were still in the start-up phase, and results of the investigations were not available. Only a preliminary water level survey had been carried out for the reach of the Tamakoshi river between the potential power outlet locations and a bailey bridge about 1.5 km further downstream; this water level survey was needed for an approximate assessment of the magnitude of, and change in longitudinal river slope for determining the best location of the outlet structure. Other specific field data, like information on the topography, the geological conditions or particulars of hydrological conditions, were especially for the project area upstream from the powerhouse location still absent.

At this time it was nevertheless necessary that the principal layout of the project was fixed to allow the design detailing. Moreover, the layout information should also serve as important input for the field investigations, as the latter activities had to be carried out with particular focus on those locations which were foreseen as sites of the major project structures.

The report was drawn up with a principal partition into three appraisal aspects, i.e.:

- the assessment of the project alignment along Tamakoshi valley as a whole;
- the assessment of the conceptual layout of the main individual project components as already foreseen in the Feasibility Study [1], and
- an appraisal of such components which are of temporary (construction-time related) nature or of add-on character with their implementation currently not decided.

Following this principal concept, the considerations, options and recommendations pertaining to the layouts are subsequently presented in the following Chapters 2 to 4 of this report.

Owing to the early time of preparation of the source documents, numerous input parameters were maintained without changes for the present document. This approach was adopted as a necessary requisite since the parameters selected as input for the source documents were used as the basis for the decision making in favor or against a certain layout solution. Such parameters include e.g. distances and dimensions, cost and tariff data, economic indicators, and the like, which basically relate to the release date of the Feasibility Study. It is nevertheless noted that most of these data did not change significantly during the subsequent period of project development, and the decisions concerning the selection of the favored layout thus remain valid even if the arrangements of some secondary structures were modified during refining of the project layout. At a later stage of the studies updated inputs became available for some parameters like e.g. costs or tariffs; in every instance parameters were however selected in sets which relate to same reference dates.

For reasons of completeness of the report it is noted that no considerations were included in the layout selection with respect to the aligning of the transmission line, since this had indispensably required the results of field investigations to be at hand.

5.2 Overall Project Layout

Generally, the overall layout of the Tamakoshi V HEP is defined by the following predetermined boundary conditions:

- At the project's upstream end, the power water feed-in point is fixed through the waterway connection tunnel which is as part of Upper Tamakoshi HEP (UTKHEP) presently under construction. The interface between UTKHEP and Tamakoshi V HEP is defined at coordinate TKV-2" for which the tunnel layout parameters and cross section are given (see Figure 5-2 on the following page).
- At the project's downstream end, the location of the power outlet structure is shown in the Feasibility Study to be at the upstream tip of a right bank river terrace just opposite of the confluence of the Suri river, approximately at Longitude 86°11'39.2"East and Latitude 27°45'30.4"North. This location is, however, not definite, and an alternative location further downstream along the Tamakoshi river may be considered to feature an enhanced feasibility.
- Whereas the project's available head will be subject to change depending on the chosen location for the power outlet structure, the power waterway discharge is fixed as the downstream waterway release of UTKHEP. Tamakoshi V HEP does not include any structure suitable for the withdrawal of water from Tamakoshi River.

The project layout recommended as per the Feasibility Study shows a some 8.2 km long headrace tunnel in straight alignment. At its downstream end the tunnel is connected to a vertical pressure (drop) shaft which continues via a short high pressure tunnel and four turbine manifolds to the powerhouse cavern. On the power cavern downstream side four draft tubes are combined in a common tailrace tunnel which conveys the water at free surface flow into Tamakoshi River.

This principal project arrangement is shown in Figure 5-1 on the following page.

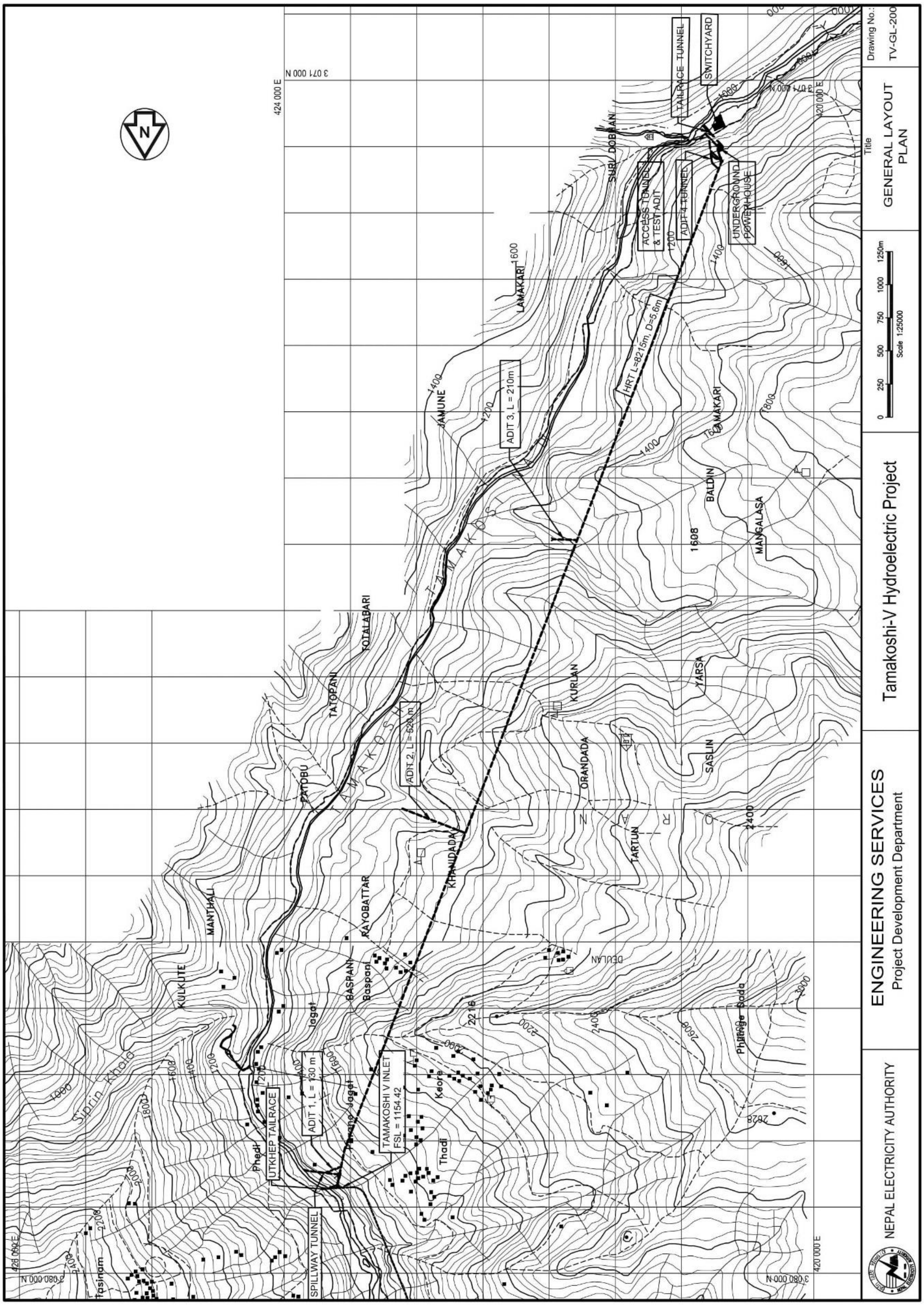

Figure 5-1: Principal arrangement of Structures of Tamakoshi V Hydroelectric Project (taken from [1])

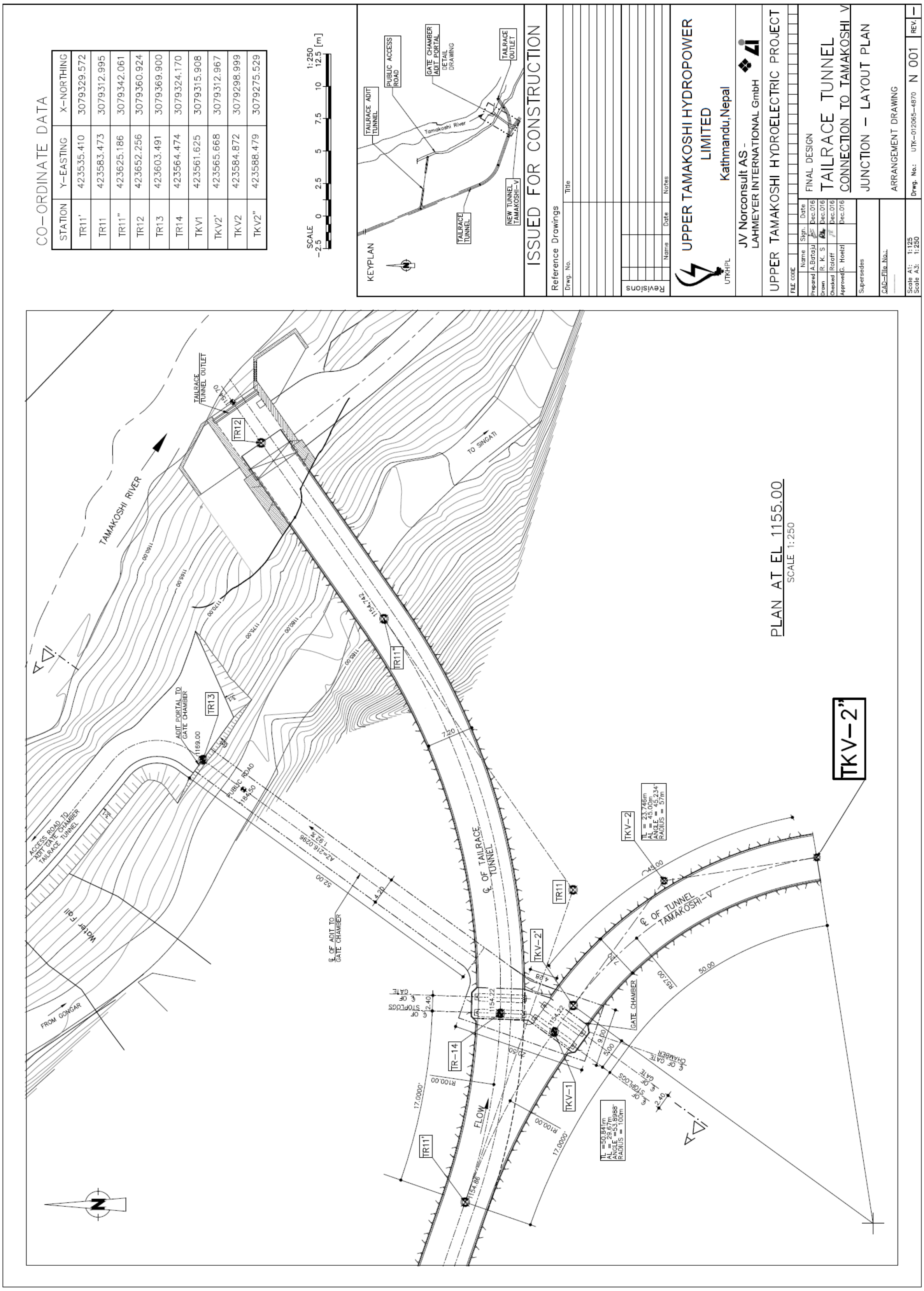

Figure 5-2: Waterway Interconnection between UTKHEP and Tamakoshi V HEP (from [3])

Given the site setting features where the course of Tamakoshi river is generally straight and the headrace tunnel is accommodated in the right bank mountain range it is justified to maintain the principal arrangement for the headrace tunnel with straight alignment in the layout. An alternative alignment following the mountain slope contour more closely was studied in the Feasibility Study; for this alignment it was, however, found that the advantages in terms of smaller rock overburden and shorter adits are more than offset by the greater headrace tunnel length and greater hydraulic losses. Minor adjustments to a straight headrace tunnel alignment are addressed further below in this part of the report; this is done in conjunction with the optimization of the construction planning.

The power station (i.e. the powerhouse cavern, transformer cavern and associated service tunnels) shall preferably be located in the area of the right bank terrace downstream from the confluence of the Suri river and upstream of the village of Bhorle. In this area the valley widens over an approximately 500 m long reach and offers space to arrange the river diversion, accommodate the switchyard and place site installations during the construction period. In addition, the location allows for easy access directly from the main access road to UTKHEP. In the directly following downstream reach the river valley narrows observably, and access to construction sites located here is difficult, costly and associated with environmental disadvantages. Further downstream the river slope decreases, making the extension of the project unfeasible.

5.3 Layout of Individual Project Components

5.3.1 Headrace Tunnel

5.3.1.1 General

As already mentioned above, the basic arrangement of the headrace tunnel as shown in the Feasibility Study for the recommended project layout is maintained as the preferred design option at the outset of the detail design elaboration. The principal layout of the structure was however reviewed under consideration of the following aspects:

- The conditions of the conceptually designed Adits 2 to 4. These adits were reviewed, and their designs fine-tuned with respect to the approaches to the adit portals, the topographic conditions at the adit portals, the adit lengths to the intersections with the headrace tunnel and the adit longitudinal slopes.
- The headrace tunnel principal features. These features include the rock overburden and groundwater table along the tunnel alignment, the sectional lengths determined through the adit intersections and the predicted construction advance rates under consideration of the anticipated geological conditions along the tunnel route.

5.3.1.2 Assumptions and Boundary Conditions

General Alignment of Headrace Tunnel

The overall layout of the Tamakoshi V HRT is defined by the following predetermined boundary conditions:

- At the tunnel's upstream end, the power water feed-in point is fixed through the waterway connection tunnel which is as part of Upper Tamakoshi HEP (UTKHEP) presently under construction. The interface between UTKHEP and Tamakoshi V HEP is defined at coordinate TKV-2" for which the tunnel layout parameters and cross section are given (see Figure 5-2). For the HRT intake (start point of HRT) the coordinates were taken as 3079190.5862 N and 423564.2290 E at preliminary level.

- At the tunnel's downstream end, the location of the surge tank is shown in the Feasibility Study to be placed in the rock slope just above the upstream tip of a right bank river terrace opposite of the confluence of the Khare Khola (Suri River). During the initial site inspection this location was found suitable and thus confirmed, subsequently included in the design of the power station and tailrace tunnel arrangement and finally approved by NEA for further detailing. The respective coordinates of the surge tank (end point of HRT) were 3071609.8811 N and 420697.0757 E at preliminary level.

<u>Permanent Adits & Adits for Construction</u>

The design of the HRT includes four access points to the tunnel alignment which were found during elaboration of the Feasibility Study to be required for the timely construction of the tunnel. Access to these four access points were proposed to be established by respective access tunnels (adits) as follows:

- Access via Adit 1 in the area of the connection tunnel. This adit shall allow construction access to the HRT most upstream part and the headpond (and from there into the connection tunnel). In this area the HRT alignment is generally fixed by the existing connection tunnel, and the HRT/adit intersection may only be moved at little margin along the HRT. Because of construction considerations the intersection point was at preliminary design level arranged 60 m downstream from the connection tunnel intake at the upstream end of the headpond.
- Access via Adit 2 which shall be constructed from a side valley to the Tamakoshi main valley and connect to the HRT some 3 km downstream from the HRT intake.
- Access via Adit 3 which shall be constructed from the side valley of the Oran Khola and connect to the HRT some 5 km downstream from the HRT intake.
- Access via Adit 4 close to the surge tank. The adit shall allow construction access to the HRT most downstream part and may be combined with the construction access to the surge tank and gate chamber of the pressure shaft control gate. Considering the need to arrange several accesses in this area the concept to provide a HRT/adit intersection some 60 m upstream from the surge tank (as shown in [1]) was maintained.

The shorter Adits 1 and 4 may be combined with tunnels providing permanent access to underground hollows. Purposes of permanent use are especially:

- related to Adit 1 the access for maintenance to the headpond, HRT intake and spillway;
- related to Adit 4 the access (as a branch-off from Adit 4) to the gate chamber located directly downstream from the surge tank.

Modifications to the designs of these two adits had to consider the operational requirements during the project's operation phase, and options for modifications of these adits were generally limited. If at all applied, such modifications would not have any impact on the HRT alignment.

Contrary to this, the longer Adits 2 and 3 were foreseen to provide access only during the construction of the project; they shall be plugged at a later stage. In order to allow easy construction of the HRT and avoid the need for continuous maintenance of the adits, they were assumed to be constructed with a 2% slope towards outside so that self dewatering is secured.

5.3.1.3 Comparison of Alignment Options

General

As stated above, the start and end points of the HRT are defined with their respective coordinates and were thus considered invariant for any HRT alignment optimization which targets at a reduction of the overall cost or construction time. Purpose of the analysis reported here below is to determine whether such optimization can be achieved by a shortening of Adit 2 or Adit 3 and moving the HRT out from a straight alignment between start and end point.

Possibility to shorten Adit 3

Adit 3 is foreseen to provide access to the HRT in the area of the Oran Khola side valley. In order to check whether a modification that could improve project construction can be applied to the HRT/adit design in this area, the main design parameters of the HRT were determined for the design option that the waterway will be constructed in straight alignment. The check revealed that at the Oran Khola crossing the internal water head in the tunnel during maximum upsurge in the Surge Tank is only slightly below natural ground level, based on topographic information taken from [1].

The results of the check indicated a margin of some 13 m between local topography and highest internal water pressure at the Oran Khola crossing. A shift of the HRT axis towards the Tamakoshi valley was therefore disregarded from further design considerations. The location of the valley crossing and the intersection point between HRT and Adit 3 axis directly neighboring the valley crossing were adopted as invariant. Preliminary adit design parameters were subsequently calculated based on this assumption.

•	HRT/Adit 3 intersection	coordinates	3074626	N
			421838	E
		invert elevation	1126.17	m asl
•	Adit 3 portal	invert elevation	1118.20	m asl
•	total length of Adit 3		398	m

Possibility to shorten Adit 2

The procedure to check the possibility for a shortening of Adit 2 principally corresponds to the one applied for Adit 3 . For this adit the HRT/adit intersection is however located in a HRT reach of significantly higher rock cover, and the adit itself is designed significantly longer with about 950 m. For the analysis a preliminary adit alignment was designed assuming a straight HRT alignment and considering the predicted local geology.

From the above data it is obvious that the adit will require a respectively long construction time. In view of construction and dewatering considerations a feasible shortening of the adit can however only be achieved by deflecting the HRT towards the Tamakoshi valley. Such deflection can be implemented by moving the HRT/adit intersection point along the adit axis towards the adit portal, for which the adit length decrease is used as indicator. For small deflections, the additional HRT length resulting from this design modification will be marginal, and even at noteworthy deflections the HRT length increase can be maintained in acceptable limits.

The structures which are affected by such design modification are:

- the construction adit which is shortened by a selected part of its length;
- the HRT for which the length is increased along the section from Adit 2 to Adit 1 (fixed intersection) and from Adit 2 to Adit 3 (fixed intersection); and
- the surge tank which increases in size due to the longer HRT.

Furthermore, the design modification influences the hydraulic losses incurred along the HRT due to its greater length. In this respect, the analysis is limited to the consideration of friction losses; as the computed results show these are of limited significance. Other additional hydraulic losses occur as bend losses, which are however of a magnitude which does not influence the results and were therefore neglected.

The above effects occur either as construction cost during the early construction phase of the project or as costs caused by hydraulic losses continuously over the project's operation period. The cost analysis thus was carried out on the basis of discounted cost. Construction cost were expressed as overall cost associated with the variation of a tunnel length by 1 m as follows:

- headrace tunnel, complete, 1 m length 5,890 USD
- surge tank, additional cost per 1 m of headrace tunnel 298 USD
- construction adit, complete, 1 m length 2,365 USD

Three variants (apart from the "base case" of no shortening) for the shortening of the adit were included in the cost analysis; they were selected as 200 m, 400 m and 600 m to cover the range of possible shortenings likely of interest. The cost analysis basically referred to cost variations which are computed in relation to the base case alignment. Construction cost variations associated with the shortening of the Adit 2 are introduced as negative cost, i.e. cost savings.

Friction losses in the HRT were determined by evaluating the Darcy-Weisbach formula for friction losses. Considering a medium wall roughness of the HRT concrete lining of k = 1.0 mm the associated head loss (loss in available net head at the turbine) was calculated as 0.88 mm when referring to a tunnel diameter of 5.6 m and a rated discharge of 66 m^3/s. When relating this to the data for rated condition as given in [1] the respective loss in annual energy generation was calculated as 2.63 MWh, and respective annual revenue losses could be quantifyied.

The total discounted cost associated with the shortening of Adit 2 were subsequently computed as the sum of discounted construction cost and discounted operation cost. The results of the optimization are presented in the graph of Figure 5-3.

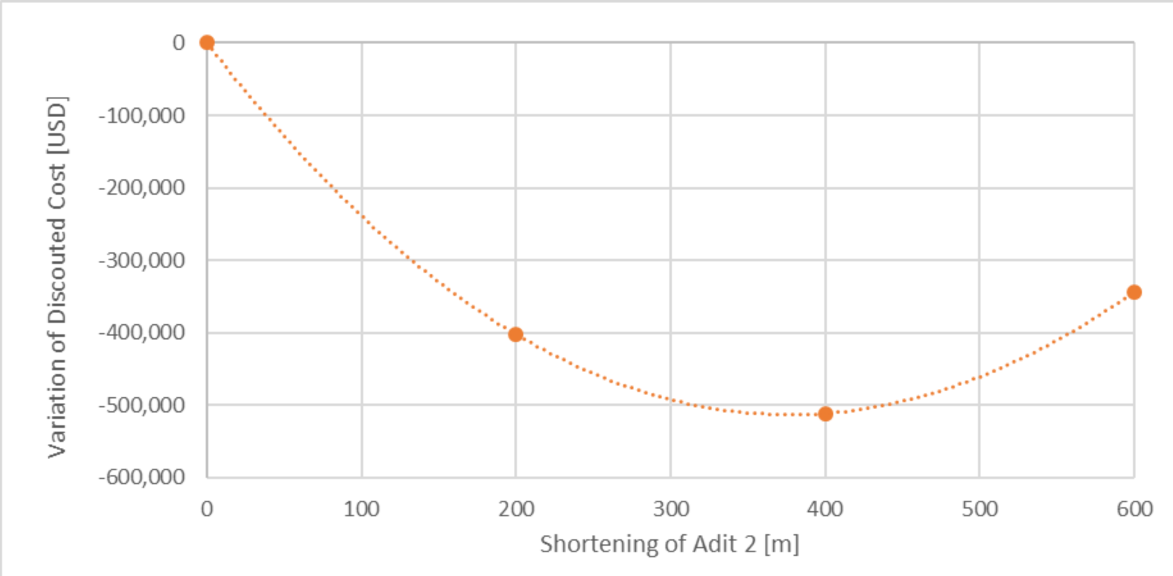

Figure 5-3: Variation of Total Discounted Cost vs. Shortening of Adit 2

In the above graph, the minimum of the total discounted cost is found for a shortening of Adit 2 of approximately 375 m; for this point the cost savings were determined as approx. 512,900 USD.

The variant marked by a shortening of Adit 2 of 375 m and a respectively deflected HRT alignment was checked for any criteria counting against such alignment solution (insufficient rock cover, adverse geological conditions, etc.); such features could not be determined. The HRT alignment deflected by 375 m at the intersection with Adit 2 was therefore confirmed as technically and economically feasible and adopted for the subsequent review related to the construction scheduling.

Further details about the alignment optimization are included in Part A1 of the Detailed Design Report.

5.3.1.4 Construction Planning Considerations

Basic Data used for the Planning

The checks whether the HRT designed with an alignment as determined in the preceding chapter can be constructed within acceptable time have to be based on a set of construction progress rates which is commonly built up from rates which by experience could be achieved on projects of similar characteristics. For the Tamakoshi V project essential key parameters were taken into account with reference to the geological drawings of [1] where sections with the rock classes II to V are used to classify rock of different quality. For the above HRT key parameters the construction rates were selected as by experience achievable for the construction planning analysis:

• Tunnel excavation	ascending	1.0 to 5.0 m/d
•	decending	-10% of above
• HRT concrete lining		10.0 m/d

Excavation of each HRT section was assumed to be executed from up- and downstream faces simultaneously as soon as access via the respective adits is available.

The excavation progress for the adits was adopted at the same rate as stated above for the HRT itself, since the cross sections do not deviate significantly from that of the HRT. Due to the longitudinal slope assumed for the construction adits, it was further assumed that only Adit 1 would be excavated descending over a short section. All other parts of the adits would be ascending excavation.

The planning was carried out presupposing that excavation of each tunnel section should be completed in the shortest time possible, and concrete lining of the section would start only after the tunnel had been fully excavated. In a second step the construction procedure for that section which required the longest time for construction (excavation and concreting) was modified as follows:

- It was assumed that excavation at one face would stop at an earlier point in time in order to allow an earlier start of the concrete lining installation. Since concreting is for dewatering purposes usually preferred to be installed in descending direction, the excavation in ascending direction was determined as the excavation operation stopped at earlier point in time.
- Excavation in descending direction would continue until break through. Installation of the concrete lining would continue from the high end of the tunnel section after having been completed in the lowermost part.

The HRT section between the adit intersections of Adit 3 and Adit 4 is the tunnel section, which was adjusted in line with the step 2 approach described above.

Influence of HRT Excavation on Project Implementation Time

The excavation period derived from the excavation planning for the HRT were subsequently incorporated in a construction planning analysis which considered all project structures which were identified as possibly having an impact on the project's overall construction period. For this part of the analysis the following structures were ruled out to be of influence:

- the connection tunnel and headpond (these structures can separately be accessed once the Adit 1 is available, and can be constructed during excavation/concreting of the HRT section between Adit 1 and Adit 2);
- the surge tank and gate chamber (these structures can separately be accessed once the Adit 4 is available, and can be constructed during excavation/concreting of the HRT section between Adit 3 and Adit 4);
- the pressure shaft and upstream manifold (these structures can separately be accessed after Adit 4 and the access via powerhouse cavern to the u/s manifold is available, and can be constructed during excavation/concreting of the powerhouse cavern);
- the downstream manifold, tailrace and outlet structure (these structures can separately be accessed from downstream via the outlet structure).

For the above structures it is reasonably assumed that their construction requires shorter time than, and that their construction activities do not significantly interfere with the construction of the HRT and the powerhouse cavern.

For the construction of the HRT and the powerhouse cavern with associated access tunnels a construction planning was prepared using an abbreviated (Level 1) construction time schedule (CTS); this schedule is shown in Part A1 of the Detailed Design Report. The construction periods determined for the adits and headrace tunnel sections were integrated in the planning at appropriate places. The powerhouse cavern was assumed to be constructed with 3 generating units; the small hydro unit is not expected to influence the construction time critically. Milestones attached to the unit blocks indicate the earliest date when the powerhouse main crane can approach the individual unit bays (required for installation of the respective spiral case). A mobilization period and the construction of the access tunnels to the main structures (including allowances for the construction of adit portals) was considered separately.

Given the aforementioned parameters for construction progress and sequencing, the planning shows that the project's overall construction time is governed by the construction of the powerhouse cavern. The analysis shows the completion date for the HRT as 35.7 months after Start of Construction (SoC), whereas water at unit 1 for wet testing and commissioning is required only 38.3 months after SoC. The project's overall construction period is obtained as 3 years and 10 months.

The analysis hence allows for a period of about 2.7 months for pressure testing of the HRT. However, if the construction time in the HRT section with rock class V increases by one month, the time available for pressure testing will reach a minimum requirement of 2 months. If the construction progress in the same section drops from 1 m per day to about 0.8 m per day, the HRT construction will become time-critical for the project, even when neglecting any time requirement for HRT pressure testing. The reserve for a rescheduling of excavation and concreting, as this has been applied for the HRT section between Adits 3 and 4, is for this tunnel only marginal.

5.3.1.5 Results and Conclusions related to the Headrace Tunnel Layout

The preceding sections present detailed descriptions of the analyses carried out with the aim (i) to confirm or optimize the alignment of the headrace tunnel (HRT) under technical and economical aspects and (ii) to determine whether the HRT in the recommended alignment is time-critical for the project implementation as a whole. The results of the analyses are summarized as follows:

(i) HRT Alignment Analysis

The analysis for this part of the investigations was carried out under the assumption that the start and end points of the HRT are invariable locations. For the intermediate Adits 2 and 3 a shortening of the adit may principally be considered, aiming at a shortening of the HRT construction time.

- The analysis performed shows that a relocation of the HRT alignment in the area of Adit 3 is not found feasible since in this area the rock cover above the tunnel is rather small.
- For the area of Adit 2 a shortening of the adit and an associated deflection of the HRT alignment is however considered feasible; it is technically possible, and best economic parameters are determined for a shortening of the adit by 375 m.

(ii) Construction Time Analysis

This part of the analysis was carried out for the HRT alignment assumed to be modified according to the findings of the HRT alignment analysis. The essential results of the analysis are:

- The HRT construction is for the adopted HRT alignment not time-critical with respect to the implementation of the overall project. According to the associated construction time schedule a period in access of 2 months (considered as minimum requirement) is available for HRT pressure testing.
- If the construction time required for the HRT section in rock class V increases by more than 1 month, the HRT becomes time-critical since the minimum time for HRT pressure testing of 2 months is no

longer available. If the excavation progress in the same HRT section drops from 1 m per day to below 0.8 m per day, no time will be available for HRT pressure testing.

5.3.2 Power Station and Tailwater Way

5.3.2.1 Layout Concepts

In line with the overall project layout described above four alternative layouts for the power station and tailrace tunnel/channel were principally taken into consideration. They make use of the existence of the terrace on the right river bank just downstream of the confluence of the Suri River and the downstream reach of Tamakoshi Rver down to Bhorle bridge.

The basic arrangement features of the alternatives are generally captured as follows:

- The alternative denominated as Alternative 0 corresponds to the design recommended in the Feasibility Study. In this layout the power station is located in the mountain ridge adjacent to the upstream extent of the terrace. The tailwater way is partly designed as free flow tunnel and over its final part as channel where it cuts through the tip of the terrace. At the channel exit section the river water level is taken from [1] as 993.1 m a.s.l.
- In the Alternative I layout the power station is arranged in the same location as in Alternative 0. The tailwater way leads however to a junction with the Tamakoshi River which is located at the most downstream tip of the river terrace; it is conceptually designed as free flow tunnel. Following the preliminary water level survey the water level here is about 13 m lower than at the channel exit of Alternative 0 (destruction of some singular boulders in the river bed may be required to secure this head drop). This level difference can in Alternative I be utilized as additional head for power generation. The turbine setting needs to be adjusted to the lower tailwater level, resulting in a respectively lower powerhouse cavern elevation.
- The layout of Alternative Ia generally corresponds to that of Alternative I. The Outlet Structure is however placed some 940 m further downstream just upstream from Bhorle bridge, thereby making an additional net head of about 9 m available for energy generation. The powerhouse cavern has to be set respectively deeper; the pressure shaft becomes respectively longer.
- The Alternative II layout features the same junction of the tailwater way with the Tamakoshi River as the Alternative I layout. The power station is however moved by some 350 m in downstream direction; this results in a rather short tailwater way conceptually designed again as free flow tunnel. As the upstream surge tank and the drop shaft are moved together with the power station, the length of the headwater way is increased respectively. The short tailwater way is associated with rather small hydraulic losses; the powerhouse cavern will therefore attain the lowest setting height in this alternative.

The four alternatives for the power station and tailwater way arrangements are shown in the drawings attached in Annex A to this part of the report. The orientation of the powerhouse cavern is still tentative; this parameter will be fixed after having explored the direct location with the help of the planned test adit. The designs of the adjoining tunnels will then be updated to suit the eventual powerhouse cavern orientation.

5.3.2.2 Selection of preferred Alternative

All Alternatives are, in comparison to the project layout of the Feasibility Study, principally marked by power waterways of extended lengths and greater gross head. The longer power waterways are associated with an increase in the construction cost, whereas the greater head will allow to generate energy with equipment of greater rated power. This section provides a comparison between the layout alternatives with the aim to identify that alternative which is associated with the overall best economical performance.

The following additional construction requirements with significant impact were determined for the Alternatives:

- For Alternative I a tailrace tunnel of 487 m will be required, whereas the same tunnel as per [1] has a length of 142 m. This results in an additional length of 345 m. Furthermore, the length of the pressure shaft will increase by 13 m due to the required lower setting height of the turbines. The omission of the tailrace channel can be counted as cost reducing construction requirement.
- In Alternative Ia the length of the tailrace tunnel is increased by another 940 m compared to Alternative I, or by 1,082 m compared to the Feasibility Study. The length of the Pressure Shaft will increase by 24 m compared to [1] because of the lower turbine setting height. The Main Access Tunnel will be 120 m longer than for Alternative I. The tailrace channel is also for this Alternative omitted.
- For Alternative II the main cost increasing construction requirement is the added section of the headrace tunnel; the additional section is estimated to be 285 m long. Due to the longer headrace tunnel also the size of the surge tank will slightly increase. And also for this alternative the length of the pressure shaft will increase by 13 m due to the required lower setting height of the turbines. On the tailwater side, the length of the tailrace tunnel will increase by estimated 55 m, whereas the tailrace channel is again obsolete.

Construction cost were expressed as overall cost associated with the variation of a tunnel length by 1 m as follows, as already done for the HRT alignment variation:

- headrace tunnel, complete, 1 m length 5,890 USD
- surge tank, additional cost per 1 m of headrace tunnel 298 USD
- pressure shaft, complete, 1 m length 14,000 USD
- tailrace tunnel, complete, 1 m length 4,960 USD
- tailrace channel, complete, 1 no. of 55 m length 504,660 USD

With the above cost data and the quantities determined for the alternatives, the incremental discounted costs for the alternatives in comparison to the design as per Feasibility Study were computed. For the resulting costs discounted by a factor 1.33 for early construction the following figures were obtained:

- Alternative I 1,848,200 USD
- Alternative Ia 8,887,600 USD
- Alternative II 2,281,000 USD

The valuation of additional generation benefits was done with reference to changed gross heads, additional head losses and respectively scaled-up rated capacities and annual energies. Presuming that the rated power of 87 MW will be available at the rated net head of 149.6 m and rated plant discharge of 66 m³/s, and that the annual generated energy will meet the stated value of 446 GWh, the economic indicators presented below were derived from the analysis (where the rated efficiency was recalculated from other preset parameters):

- Alternative I Rated Capacity = 94.24 MW
  Annual Energy = 483.12 GWh

•	Alternative Ia	Rated Capacity =	99.43 MW	
		Annual Energy =	509.71 GWh	
•	Alternative II	Rated Capacity =	94.24 MW	(as Alternative I)
		Annual Energy =	483.12 GWh	(as Alternative I)

The revenues associated with the additional annual energy amounts were evaluated against the power station & waterway layout of the Feasibility Study by adopting the same basic economic parameters, since these parameters relate to the same reference date as the cost data used for estimation of the differential construction costs. The adopted parameters include:

- energy benefit, NEA scenario, year 2011 — 3.19 NRs/kWh
- energy benefit, NEA scenario, escalated to year 2020, in US ct. — 5.55 US ct.
- project's life time (civil works) — 50 years
- discount rate p.a. — 10 %

The analysis was carried out on the basis of discounted cash flows. For the annual surplus energy amounts generated by the alternatives the evaluation resulted in the following additional revenues computed as discounted values over the project's life time:

- Alternative I — 20,412,175 USD
- Alternative Ia — 35,036,893 USD
- Alternative II — 20,412,175 USD

5.3.2.3 Results and Conclusions related to the Power Station and Tailwater Way Layout

In the following table the most important results and indicators obtained from the analysis of the alternatives are compiled on a comparative basis.

Table 5-1: Key Results & Indicators for Power Station & Tailrace Tunnel Alternatives

Key Results & Indicators	Units	Feasibility NEA	Alternative I vs. FS	Alternative II vs. FS	Alternative Ia vs. FS
Rated Net Head	m	149.6	**162.05**	162.05	170.97
Rated Power Output	MW	87	**94.24**	94.24	99.43
Total Annual Energy	GWh	446.00	**483.12**	483.12	509.71
Incremented/Discounted Construction Costs	USD	0,00	**1,848,200.00**	2,281,000.00	8,887,600.00
Incremented/Dicounted Generation Benefit	USD	0,00	**20,412,174.69**	20,412,174.69	35,036,892.52
Net Benefit (benefit minus cost)	USD	n.a.	**18,563,974.69**	18,131,174.69	26,149,292.52
Benefit/ Cost Ratio			**11.04**	8.95	3.94
Specific Aspects pertaining to Alternative					
- Geology					*)
- Environment					*)

*) Geological maps provided up to the licensed border as indicated in the Contract

From the above table it is obvious that the Alternative I concept, when compared to the FS concept as the reference case, exhibits a very attractive benefit/cost ratio. Alternative II falls slightly short of the result for Alternative I; since both alternatives are expected to yield the same benefits, Alternative II is also marked by a smaller net benefit. For the Alternative Ia the net benefit is determined slightly higher than for Alternative I, however, the benefit/cost ratio is significantly smaller. The latter is due to the significantly greater discounted costs. For Alternative Ia it has to be noted in addition that

- the available geological maps do not cover most of the tailrace tunnel reach, and the geological risk associated with this alternative is hence judged higher than for Alternative I, and
- the alternative is expected to be associated with a significantly greater environmental impact, since the Tamakoshi River will over most of the year carry only small water flows past Bhorle village and up to the Bhorle bridge.

In summary, it was therefore concluded that the Alternative I concept provides the most feasible combination of technical and economical advantages and shall be adopted as the recommended concept for the final project layout.

5.3.2.4 Transformer Cavern

At the early stage of design development it was not decided whether a switchyard entirely above ground (as proposed in [1]) or an underground transformer cavern will be incorporated in the project design. The distance between the end wall of the powerhouse cavern and the access tunnel portal is only some 130 m, and at this short distance the omission of a transformer cavern in favor of an outdoor switchyard may well be feasible.

The decision on the finally preferred concept was made in the course of the detail design elaboration, when sufficiently precise input information for this design decision was available.

The transformer cavern will be arranged in parallel to the powerhouse cavern and across the turbine draft tubes This arrangement safeguards the presence of a sufficient rock cover between the cavern and the waterways, and it locates the cavern in a superior way in relation to the power lines leading to the surface.

5.3.2.5 Turbine Setting Height and Test Adit

In the drawings of the Feasibility Study it is indicated that the test adit will be aligned in the same route as the main access tunnel. With this concept it will pass through the cavern along its upstream wall at maintenance floor level. It is understood that this concept was chosen with the aim to reduce the overall required excavation volume in the power station area.

Under consideration on commonly applied design approaches the shown arrangement features, however, a number of disadvantages, like e.g.:

- The service tunnel system concept requires that in any case two separate escape routes towards outside have to be provided. This can be achieved through provision of two service tunnels, or one tunnel featuring several separated compartments (and a respectively large cross section). The latter option has mostly proven unsuitable because of construction and operational aspects.
- With the test adit located at machine hall floor level, it will be difficult to reach the cavern roof when the full scale excavation of the cavern is carried out. A relatively long ascending extension of the adit has to be excavated before the main excavation can start. Ventilation of, and mucking through this adit are difficult.

For these reasons, a test adit running close to the cavern roof centrally through the cavern is preferred and suggested for the Tamakoshi V powerhouse cavern. Such alignment requires, however, that the appropriate elevation of the adit within the cavern reach is checked. These checks were carried out for three scenarios based on the following assumptions:

- The cavern arrangement corresponds to Alternative I
- The tailwater level in the river was conservatively considered 13.5 m below the water level according to [1], i.e. at 979.6 m a.s.l.
- The hydraulic losses in the tailwater way were estimated as 0.9 m, thus resulting in a tailwater level of 980.5 m a.s.l. at the turbine.

Given these assumptions the scenarios summarized in Table 5-2 below were checked with the intention to determine an elevation for the test adit that will be suitable for the likely equipment configurations.

Table 5-2: Estimated Test Adit Invert Elevation for different Generating Equipment

Generating Equipment Data	4 Units 21.75 MW 428.57 rpm	4 Units 21.75 MW 428.57 rpm	3 Units 29.00 MW 375.00 rpm
Source of Data	Feasibility Study Tamakoshi V HEP	Lahmeyer Int'l. Predimensioning	Lahmeyer Int'l. Predimensioning
Runner	979.62	978.77	978.63
Cavern Roof El.	1004.82	1005.57	1007.19
Adit Invert El.	1000.32	1001.07	1002.69

From the above table the test adit invert elevation is determined lowest for the data set extracted from [1]. Consequently, this elevation will have to be considered for the excavation of the test adit within the cavern reach. Given a test adit length of about 127 m, the adit can be excavated with an ascending slope of 1.82 % if the adit portal is constructed with an invert elevation of 998.00 m a.s.l.

5.3.2.6 Service Tunnels for Power Station

The Feasibility Study falls principally short of depicting a service tunnel system which is suitable to accommodate all functional requirements associated with an underground power station. The basically required functions are briefly summarized here below:

- access for heavy load transports;
- fresh air supply to the cavern (aeration);
- power evacuation from the cavern (e.g. XLPE cable connections);
- power cavern dewatering (if not via the power waterways);
- smoke extraction towards outside (in case of fire);
- second separate escape route (in case of emergencies).

From the above list it is evident that, just to satisfy the last functional requirement of the tunnel system, usually at least two tunnels towards outside are needed.

One of these tunnels can be provided by upgrading the test adit which will be driven into the cavern location at the early stage of the geological survey. Provided that the test adit enters the cavern at a suitable point, it can later be used to incorporate the functions of fresh air supply (normal operation), smoke exhaust (in case of fire), power evacuation and escape route (through a separate compartment in the tunnel). The main access tunnel would function as traffic access, cavern dewatering route, air exhaust (normal operation) and fresh air supply (in case of fire), and it would serve as separate escape route.

Following common experience related to the design of cavern power stations it is a preferred arrangement if the main access tunnel enters the powerhouse cavern through the side wall. In the layout, a short tractor niche can be provided at the opposite cavern side to allow long trailers to enter the cavern. With an arrangement as shown in [1] for the maintenance floor the transport, once arrived, will occupy most of the cavern’s laydown area, and his advance into the cavern will be limited by the unit next to the tunnel entrance.

For the power station arranged as per Alternative I the main access tunnel will likely be aligned in a wide, about 200 m long curve. From its portal located in the back of the outdoor switchyard area it will descend in a long right turn to the machine hall floor of the powerhouse cavern, where it will enter the cavern through the side wall into the maintenance bay. The difference in elevation between the tunnel terminal points is estimated to be about 15 m; this will be confirmed following the availability of the results from the topographic survey.

5.3.2.7 Tailwater Way

Basic information for the river reach under consideration was obtained from a preliminary water level survey. This survey was carried out during the first days of May 2017 and focused on information about the river slope. The results of the preliminary survey are compiled in Table 5-3 below.

Table 5-3: Coordinates & Elevations obtained from Preliminary Water Level Survey

Project: Tamakoshi V Hydroelectric Project
Task: Preliminary Survey of River Bank Water Levels
Location: Bhorle, Singati, Dolakha Zone 45 R
Date: 03.05.2017 to 05.05.2017

No.	Easting (m E)	Northing (m N)	Rel. Elevation Z (m)	Remarks
Right Bank Check Points				
11	420597.1900	3070705.4300	0.00	FS Tailrace Channel Exit (approx.)
12	420577.0343	3070694.6130	-1.10	
13	420548.6034	3070672.2000	-3.15	
14	420468.6624	3070564.6000	-4.94	
15	420431.3769	3070544.5720	-9.00	
16	420357.9290	3070540.5120	-10.89	
17	420282.6998	3070535.3570	-11.57	
18	420237.6760	3070522.5800	-12.15	approx. 70 m u/s Outlet Options I & II
Left Bank Check Points				
21	420478.1820	3070543.4890	-4.95	
22	420395.7687	3070494.8430	-11.72	
23	420317.9967	3070492.6950	-12.27	
24	420266.3722	3070480.2940	-12.86	
25	420229.9198	3070452.1740	-13.92	Tailrace Outlet Options I & II (approx.)
26	420193.0497	3070420.5110	-14.63	
27	420142.7966	3070347.0610	-15.50	
28	420051.4658	3070291.6070	-17.79	
29	419977.9165	3070257.5280	-19.10	
30	419891.9184	3070159.1680	-18.36	
31	419821.4248	3070110.4460	-18.86	
32	419756.3093	3070085.1760	-20.62	
33	419717.3379	3070049.0970	-21.87	
34	419663.1342	3070004.8110	-22.28	
35	419608.1026	3069969.5030	-23.30	
36	419558.2043	3069935.4920	-24.77	
37	419484.9316	3069884.2430	-24.84	approx. 70 m u/s of Bailey Bridge

More information about the site conditions prevailing along the investigated river reach is also presented in Part B “Initial Field Reconnaissance”, Subchapter 4.5, of the Inception Report [2].

5.4 Verification of Diameters of the Power Waterways

5.4.1 General

The general project configuration of the Tamakoshi V HEP was already updated resp. confirmed in the very early phase of the project definition. As stated in Chapter 1, this was a fundamental requirement as it defines, among others, the gross head available to the project and in consequence (at given plant discharge) also the reasonable installation of generating capacity. The parameters determined during that early design phase comprised the alignment of the Headrace Tunnel, the location of the Outlet Structure and the arrangement of the power waterways in the vicinity of the Power Station.

The analysis with respect to the adequate sizing of the power waterways was postponed to a later phase of the project design, when more details were known about the tariffs that will be applicable for the hydropower project and the energy amounts that can be generated and sold. This information, especially the potential for energy generation, became at some parts available only during the advanced phase of the project design, since it required extensive clarifications related to the availability of water at the Tamakoshi V Headpond.

The latter proved to be of particular importance with respect to small flows which will be discharged from UTK HEP as spinning reserve releases and be available for energy generation by a small hydro unit installed at Tamakoshi V. Whereas the overall amount of energy generated by this unit is comparably small, it yet contributes significantly to the total energy generated during the dry season of the year. In the context with the recent clarification that the Tamakoshi V HEP will be classified under the NEA tariff regulations [4] as a ROR project, the total amount of generated dry season energy becomes a key indicator of paramount importance for the viability of the project.

It is nevertheless noted that the tariff regulations finally applied were not entirely known when preparing this report. Respective reference on this is provided further below. Also, uncertainties exist with respect to the unit rates of major construction items which are relevant for the construction of the power waterways. It appeared therefore advisable to apply a reasonable degree of conservatism to the analyses prepared in relation to the sizing of the power waterways.

5.4.2 Assumed Tariffs & Sale of Energy

Several financial and economical input parameters have to be determined at the outset of the assessment of the waterway sizing in order to allow the detailed analyses. Among these are the parameters describing the amounts of energy which will be made available at the point of delivery, and specific conditions applying to the delivery. For Tamakoshi V, it was clarified that the project will be classified as a ROR project (4&8 months) under the NEA tariff regulations [4]. As per these regulations, the applicable tariffs are specified as follows:

Table 5-4: Tariff Specification for ROR Project 4 & 8 Months as per [4]

Option	Season	Rate Rs/KWh (upto 100 MW project)	Min. Dry Season Energy required
2 (Dry and wet season months 4 and 8 respectively)	Wet (Baisakh - Mangsir) *)	4.8	15%
	Dry (Poush - Chaitra) *)	8.4	

*) Baisakh - Mangsir corresponds to mid April - mid December

The above specification thus requires to distinguish into dry and wet season energy amounts. For Tamakoshi V these figures are available from the energy generation simulations presented in Part F2 of Detailed Design Report. For the analyses presented herein the scenario without consideration of Rolwaling RDS being implemented and without outages was selected as the relevant scenario, since (i) the implementation of the Rolwaling River Diversion Scheme was not decided at the time of elaborating the analyses, and (ii) the modeling of outages was so far prepared as a best assumptions approach without having the outage scheduling confirmed by NEA. For this scenario, the energy amounts delivered at Khimti incoming terminals are summarized below:

Table 5-5: Energy Amounts delivered at Khimti SS incoming Terminals (all data in GWh/a)

	Peak Energy	Non-Peak Energy	Total
Wet Season	326.58	72.06	**398.64**
Dry Season	46.76	4.61	**51.37**
Total	**373.34**	**76.67**	**450.01**

For the same scenario, the respective energy data valid for the Tamakoshi V outgoing terminals (i.e. not considering transmission line losses) are:

Table 5-6: Energy Amounts available at Tamakoshi V outgoing Terminals (all data in GWh/a)

	Peak Energy	Non-Peak Energy	Total
Wet Season	338.12	73.61	**411.73**
Dry Season	48.43	4.61	**53.04**
Total	**386.55**	**78.22**	**464.77**

Given the NEA tariff regulation for ROR (4&8 months) the total dry season energy delivered at Khimti SS shall correspond to a required minimum of 15% of the total sellable energy. This leads to the amount of sellable wet season energy which is determined as 291.10 GWh/a. Further, the total sellable energy is derived as 342.47 GWh/a. For the remaining part of generated energy of 107.54 GWh/a it was not clarified whether this can be sold, or will be delivered at no cost. It was hence assumed that this excess energy will not generate revenues. This approach reflects a comfortable degree of conservatism, and moreover implies that a noteworthy margin for a reduction of wet season energy is available which, if materializing, will not directly affect the level of revenues.

The tariff related to the above delivery of energy was determined as a mixed tariff based on the data provided in Table 5-4 above. Assuming that 51.37 GWh/a will be sold at a tariff of 8.4 NPR and 291.10 GWh/a at a tariff of 4.8 NPR, the mixed tariff is determined as 5.34 NPR. For the use in the analyses this tariff was converted into USD applying a conversion rate of 1 NPR = 0.9 USct. (conversion rate reported for 30th June 2018); the mixed tariff is therefore obtained as 4.806 USct.

A back calculation performed on the basis of the energy data provided in the above two tables allows to conclude that 96.6% of the peak energy available at the Tamakoshi V outgoing terminals is delivered at the Khimti SS incoming terminals. This ratio is in excellent match with the line losses of 3.24 MW determined for Tamakoshi V operating at rated condition of 94.8 MW. It allows also to conclude that the entire energy amount of 342.47 GWh/a to be delivered at Khimti SS can be covered by peak energy available at the Tamakoshi V outgoing terminals. The actual amount of non-peak energy to be delivered during the dry season is determined as about 1.35% of the total and is thus negligible for the analyses related to the sizing of the power waterways.

For the simplification of the modeling is was therefore assumed that the entire annual energy to be delivered will be generated at Tamakoshi V HEP at rated conditions, i.e. at a plant discharge of 66 m³/s and a power level of 91.56 MW at the Khimti terminals. At this condition, the energy can be delivered to Khimti over a minimum required time of 3,740.39 hrs.

5.4.3 Verification of Waterway Diameters

5.4.3.1 General Assumptions and Methodology

The verification of the waterway diameters was carried out along the same principal approach as this is also presented further above for the refinement of the Headrace Tunnel alignment. However, in the present case the waterway diameter is the modified parameter, and the calculated variations in total cost are the resulting parameter. The total cost are computed as the total of missed revenues due to increased hydraulic losses (here always determined as additional cost compared to the most favorable option) and the expected construction cost.

The cost incurred due to missed revenues were quantified by using the above data derived for the mixed energy tariff, the operation data of Tamakoshi V for rated conditions, and other plant specific data. The construction cost associated with the individual options had to be computed on the basis of up-to-date civil construction unit rates, which were taken from the Consultant’s data base providing recent data from comparable projects for similar works. The unit rates used in the present analyses are compiled here below:

Construction Data	Unit Rate U/G Excavation	USD/m³	51.50
	Unit Rate U/G Shotcrete	USD/m³	298.00
	Unit Rate Rock Bolt U/G	USD/m	20.60
	Unit Rate Concrete U/G	USD/m³	160.00
	Unit Rate St. Reinf. U/G	USD/t	1580.00
	St. Reinforcement Content	kg/m³	55.00

It is however noted that the above unit rates were selected without reference to the unit rate analyses carried out for the project cost estimate. The cost figures obtained from the analyses hence do not relate to the costs presented in Part D of the Detailed Design Report.

The cost information for lining steel to be considered for the verification of the pressure shaft diameter includes a wider margin of uncertainty. This is due to fluctuating world market prices for steel, the varying “appetite” of steel manufacturers to supply lining steel, and other factors. The figure below presents the fluctuation of prices for hot rolled steel (for demonstration purposes).

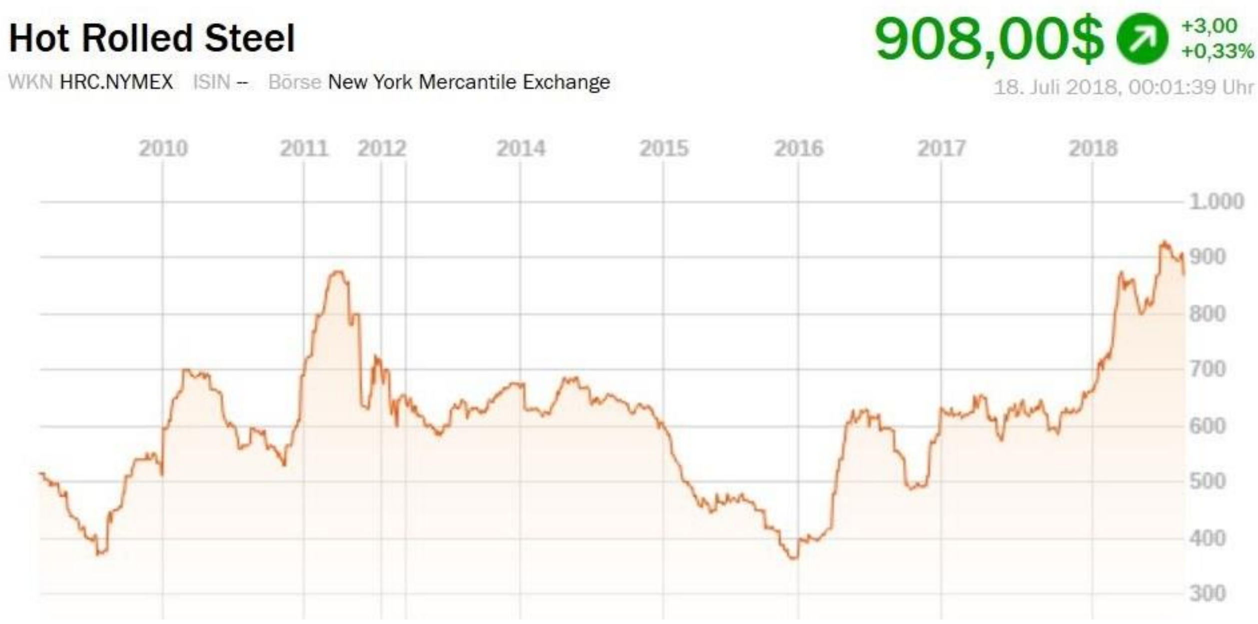

Figure 5-4: Variation of Prices for Hot Rolled Steel (for demonstration purposes)

From recent projects various price information was available, among others from UTK HEP and Tanahu HEP. The Feasibility Study [1] for Tamakoshi V used prices of 2,500 USD/t and 2,800 USD/t. When referring to all available information, prices may be expected in the range of slightly below 2,000 USD/t and more than 3,000 USD/t, which is in reasonably good match with the prices assumed in [1]. For the analyses presented here below prices were assumed between 2,000 USD/t and 5,000 USD/t to cover a sufficiently wide range of possible prices, although the upper margin is somewhat higher than the actually sourced data.

All costs were determined as discounted costs. For the costs associated with missed revenues a project life time of 50 years was adopted, as this had also been done for other comparable analyses. For the construction related costs, the times when these costs are incurred were assumed in line with most recent available information about the construction scheduling. The discount rates were varied in the range from 8% to 12% to allow for the judgement of the influence of a discount rate variation.

5.4.3.2 Verification of Headrace Tunnel Diameter

The verification of the Headrace Tunnel diameter was carried out to cover also the reach of the Tailrace Tunnel. This latter tunnel is sized with the same diameter as the Headrace Tunnel; since no circumstances were identified which would have required a deviating approach (e.g. significantly different geological conditions), this sizing is reasonable. The main parameters used in the analyses were therefore selected as:

- the gross head H 173.45 m
- the tunnel length (HRT + TRT) 8,560 m
- the wall roughness 1.0 mm

- the concrete lining thickness 0.40 m
- the average shotcrete thickness 0.10 m

For tunnel variants included in the study (only shotcrete lining or no lining) the wall roughness was set at 100 mm and 300 mm, respectively. For these variants only an invert concrete was assumed to provide a driveway inside the tunnel and facilitate its dewatering. Friction losses were computed using the Darcy-Weissbach formula.

With respect to the construction periods the start and end dates were selected as listed below. These dates do not match the exactly planned start and end dates in each instance, since tunnel construction will be carried out in a staggered sequencing for the individual tunnel sections; however, the selected dates are deemed to reflect the general construction scheduling in good match:

- start of tunnel excavation 29 months before COD
- end of tunnel excavation 21 months before COD
- start of tunnel lining / finishing works 24 months before COD
- end of tunnel lining / finishing works 10 months before COD

Note: COD = Commercial Operations Date

For the above referred options and variants the analyses were carried out with the aim to obtain the least cost (optimum) tunnel diameters and the related construction costs. The results are compiled in the table below.

Table 5-7: Results for Verification of Headrace Tunnel Sizing - Optimum Diameters

Discount Rate > Tunnel Finishing	8%	10%	12%
Tunnel with Concrete Lining	5.597 (45.43)	5.413 (45.16)	5.253 (45.16)
Tunnel with Shotcrete Lining	6.706 (48.07)	6.483 (48.15)	6.294 (48.49)
Tunnel unlined (support only by bolts)	7.333 (39.16)	7.103 (38.97)	6.915 (38.70)

Notes: Upper figure = tunnel diameter in m, lower figure (in brackets) = estimated total cost in mio.USD.

From the above results it is concluded that the tunnel diameter of 5.6 m is determined under the assumption of 8% discount rate, which is below the discount rate used elsewhere in the study. The additional cost of this option are, however, derived negligible at +0.6%. The design tunnel diameter is thus considered to be selected very close the optimum diameter and is confirmed.

Tunnel variants with shotcrete lining were analyzed to be generally associated with higher total costs; they are thus ruled out for any further consideration. Unlined tunnel variants were, to the contrary, determined to feature lower costs in the range of about 14%. These variants are therefore principally worth to be considered; however, the geological conditions predicted for the tunnel alignment did at the present level of design detailing not support an unlined tunnel, and these variants were consequently ruled out for geotechnical reasons. From the geological/geotechnical survey results available at present, an unlined tunnel might only become technically feasible in the most upstream section of the Headrace Tunnel; the respective decision should be made once the tunnel reach has been excavated and the geotechnical conditions can be assessed by direct inspection and tests.

5.4.3.3 Verification of Pressure Shaft Diameter

The verification of the Pressure Shaft diameter was carried out in likewise manner as presented above for the Headrace Tunnel. The differences pertaining to these analyses include mainly:

- that the backfill concrete behind the steel liner is unreinforced
- that the plate thickness of the steel liner had to be determined for several sections depending on the internal pressure and liner diameter (the simplified approach made use of the vessel formula)
- the analyses focused on the shaft of constant diameter, whereas the bifurcators were treated as being in their designs dependent on the shaft.

The geometric parameters of the shaft and bifurcation section were computed in a separate calculation table based on the set shaft diameter. The pressure shaft itself was considered with a length of 188 m, whereas the total pipe system along the longest pipe had a length of 259 m. Plate thicknesses along the main shaft were varied in four steps, with the minimum plate thickness set as 12 mm. The dynamic pressure rise was adopted from the hydraulics report for the transient conditions in the power waterways as +15.7%. The admissible tensile strength of steel was set at 140 N/mm². A corrosion allowance of 1 mm was considered.

For the construction of the Pressure Shaft it was assumed that the shaft excavation will be done by raise boring with subsequent widening. For the installation of the steel liner it was assumed that this activity will start at the lower bend and the shaft will then be constructed in upwards direction at a speed of 6 m per week while at the same time the steel liner parts downstream from the lower bend will be installed. With a length of the upper part of 175 m the construction of this part was determined to require 6.7 months. The start and end dates were thus selected as listed below:

- start of shaft excavation 30.5 months before COD
- end of shaft excavation 26.5 months before COD
- start of shaft lining / finishing works 23.2 months before COD
- end of shaft lining / finishing works 16.5 months before COD

Note: COD = Commercial Operations Date

For the above referred assumption the analyses were carried out with the aim to obtain the least cost (optimum) shaft diameters depending on varying costs for the lining steel. The table below summarizes the results obtained from the analyses.

Table 5-8: Results for Verification of Pressure Shaft Sizing - Optimum Diameters

Steel Liner Cost / Discount Rate >	8%	10%	12%
2,000	4.273	4.145	4.042
3,500	4.077	3.937	3.844
5,000	3.928	3.81	3.698

Notes: Steel liner cost are in USD/t, results are liner inner diameters in m.

The results obtained for the Pressure Shaft sizing show that the selected shaft diameter of 4.2 m matches for the parameter combination of discount rate / steel liner cost either selected at 9% / 2,000 USD/t or selected as 8% / 2,500 USD/t in an ideal way. The assessment based on elsewhere used discount rates and lately recorded prices would correspond to a combination of approximately 10% / 2,500 USD/t which would support a diameter of about 4.1 m. As noted for the Headrace Tunnel, the diameter of the Pressure Shaft is hence selected slightly greater than the optimum diameter, but very close to the optimum. The diameter is therefore generally confirmed by the analyses.

5.5 Further Verifications of Project Components

5.5.1 Upstream Surge Tank

The basic arrangement of the upstream surge tank as shown in the Feasibility Study is at the present design stage principally confirmed. Especially the design concept of arranging the structure entirely underground is judged to be an adequate design solution, considering that the rock in this area appears to be well competent to allow the accommodation of a structure of that size and that the structure will be situated in a place properly protected from potential rock fall from higher rock slopes.

The precise location of the surge tank may still be fine-tuned in the course of the detail design development. This will be done taking the following aspects into account:

- general underground rock conditions, as they will have been surveyed through a core drilling which is foreseen to be carried out in the surge tank location;
- hydraulic performance of the structure;
- modification of the structure for purposes of enhanced construction and/or cost;
- access to the base and top of the structure.

As stated for the location of the surge tank, also the type of the surge tank will be reviewed during the detail design elaboration. This check will focus on the option whether a surge tank with upper and lower surge chambers or with throttle in the connecting shaft may prove feasible with respect to performance, construction and/or cost. This review will, however, not influence the principal arrangement of the structure.

5.5.2 Pressure Shaft and u/s Valve Chamber

The layout arrangement of the pressure shaft and the upstream valve chamber are at this stage of the design development generally confirmed.

It is nevertheless noted that the design is judged to be relatively compact. The valve chamber is arranged rather close to the surge tank base which may cause a relatively great seepage inflow into the chamber. Also, the vertical bends of the pressure shaft are designed with a bend radius of only about R = 1.0 D (where D is the waterway diameter), whereas standard designs are for reasons of acceptable hydraulic performance usually prepared with a radius of about R = 3.0 D. Such radius would also enhance the construction of the bend steel liner which is commonly executed as a multi-sectional miter bend.

The straight high pressure waterway section just upstream of the u/s manifolds provides a sufficient spatial reserve to accommodate vertical bends of greater radii without necessitating the surge tank to be shifted in upstream direction. The pressure shaft may, however, have to be shifted slightly in downstream direction.

5.5.3 Upstream Manifolds

The principal layout arrangement of the upstream manifolds is confirmed at this stage of the design elaboration.

It is, however, recorded that the design presented in [1] is yet of quite rudimentary nature, and the bifurcations will require slightly more space towards upstream than what is shown in the drawings. This additionally required space can be made available through the change of the turbines to clockwise rotating equipment, so that a shifting of the powerhouse cavern due to this design detailing is not expected necessary.

5.6 Components without Impact on the Overall Project Layout

5.6.1 River Diversion Works

The Feasibility Study does not present any conceptual design drawings for the river diversion works which will be required at the construction site for the power outlet structure.

From e.g. UTKHEP it is known that the river diversion works which are constructed around the outlet structure of that project are of rather limited spatial extent. A widely similar situation is also expected for the Tamakoshi V project. In the location foreseen for the construction of the Tamakoshi V outlet structure the river occupies only a part of the valley which features here a widened profile. The opposite (left) bank consists of a terrace of very low elevation above river water which is apparently flooded during every year’s high water season. This terrace can be used to divert the river temporarily to the left so that a cofferdam can be constructed around the construction site of the power outlet structure at the right bank.

A principal arrangement of the above river diversion concept is shown in the drawings of the Detailed design Report. The further elaboration of this concept will be a task of the EPC contractor for the Civil Works.

5.6.2 Temporary Site Roads

Temporary site roads may be required e.g. to reach the locations of the construction adit portals, contractors’ yards and places of equipment pre-assembly, areas for transmission tower erection, quarries and disposal areas, specially assigned places like e.g. for explosives storage, and the like. These roads are principally foreseen to be used during the project construction period, but the areas shall be reinstated to pre-construction status upon completion of the construction activities.

The alignments of these roads will be fixed on the basis of the results from the topographic survey campaign, and with reference to design parameters that are usually applied to construction roads not intended for unrestricted public use. To the extent that such roads lie within construction areas handed over to individual contractors for construction purposes, such contractors may construct supplementary temporary roads for their exclusive use.

If at the end of construction any such roads shall be transferred to local communities for further public use, this shall ideally be coordinated already upfront of road construction with the authorities in charge.

Temporary roads are not expected to have any impact on the layout of permanent structures of the Tamakoshi V Hydroelectric Project.

5.6.3 Quarries & Disposal Areas

Quarries and disposal areas are by nature areas of temporary use. These areas, including their approach roads, shall generally be managed in the same way as temporary site roads, i.e. they may be used during the project construction period, but the areas shall be reinstated to pre-construction status upon completion of the construction activities. Any deviation from this principle shall be coordinated upfront of the utilization of such areas with the authorities in charge.

Quarries and disposal areas are not foreseen to have any impact on the layout of permanent structures of the Tamakoshi V Hydroelectric Project.

5.6.4 River Catchments

The catchment of rivers was as per the design presented in [1] not foreseen for the Tamakoshi V Hydroelectric Project.

Such installations, if decided to be added to the project in future, are not foreseen to have any impact on the layout of permanent structures of the Tamakoshi V Hydroelectric Project.

5.7 Summary and Recommendations

Within the investigations described in this report the layout of the Tamakoshi V HEP was scrutinized for numerous design aspects. The layout shown in the Feasibility Study [1] was for this purpose used as the starting point. The overall project arrangement design as well as the designs shown for individual structures were evaluated and compared to possible design alternatives.

The results of the evaluations can be summarized as follows:

- The overall project arrangement design is generally confirmed. Given the site setting features where the course of Tamakoshi river is generally straight and the headrace tunnel is accommodated in the right bank mountain range it is justified to maintain this arrangement with basically straight project alignment in the layout.
- For the headrace tunnel in particular the analyses showed that a minor deflection of the alignment combined with a shortening of the construction adit no. 2 (Adit 2) will provide advantages with respect to the construction time and the overall economic performance parameters. A shortening of the

adit by some 375 m is thus recommended together with a deflection of the headrace tunnel between Adits 1 and 3. For the remaining headrace tunnel sections the layout shall remain unchanged.

- The layout of the power station (powerhouse cavern as governing structure) was investigated in conjunction with the alignment of the tailwater way and the location of the outlet. The outlet was varied with option between the location as per [1] and a location close to a bailey bridge some 1,500 m further downstream. Among the investigated alternatives the layout featuring the power station in the location as per [1] and the outlet in a location almost 500 m further downstream was found to be most favorable. The tailwater way will for that layout be entirely a tunnel, and a channel will become obsolete.
- The diameters of the power waterways could be verified by respective economic analyses. The sizes of the head- and tailrace tunnels of 5.6 m diameter and of the pressure shaft with a diameter of 4.2 m are selected slightly at the larger end, however close to the optimum diameters. Especially with respect to the pressure shaft it is nevertheless worth noting that the world market price for steel is a highly volatile parameter. The results obtained from the present study shall therefore be understood as subject for review when the project is tendered, and the EPC bids are received.
- With respect to the remaining project structures (u/s surge tank, pressure shaft, u/s & d/s manifolds, service tunnels) layout corrections will mainly result from observed site conditions and the locations and orientations of the main structures. These layouts are not expected to be of decisive influence on the overall feasibility of the project.

For the layout design which should be further developed in the detailed design it was therefore recommended to modify the layout presented in the Feasibility Study in such way that (i) the headrace tunnel is shifted slightly towards the Tamaokoshi valley at the junction with Adit 2, thereby allowing a shortening of the adit in the recommended magnitude. The outlet of the power waterways was recommended to be placed at the downstream extension of the river terrace at Suritar, some 500 m downstream from the powerhouse cavern. For this alternative, the tailwater way was recommended to be designed entirely as pressurized tunnel.

Following the approval of the recommendations by the Client the layout design described above was used as the basis for the detailing of the project design.

5.8 References

[1] Feasibility Study of Tamakoshi-V Hydroelectric Project,

Project Development Department, Engineering Services, Nepal Electricity Authority, Kathmandu, August 2011, specifically

Volume I - Main Report

Volume III – Annex

Final Drawings

[2] Tamakoshi V Hydroelectric Project, Inception Report

Lahmeyer International GmbH, Bad Vilbel, Germany, May 2017

[3] Upper Tamakoshi Hydroelectric Project, Construction Design

JV Norconsult AS - Lahmeyer International GmbH, Norway & Germany, December 2016

[4] NEA BOARD DECISIONS ON THE POWER PURCHASE RATES AND ASSOCIATED RULES FOR PPA OF ROR/PROR/STORAGE PROJECTS EFFECTIVE FROM 2074/01/14 (April 27, 2017)

Annex A Power Station and Tailwater Way, Layouts, Alternatives

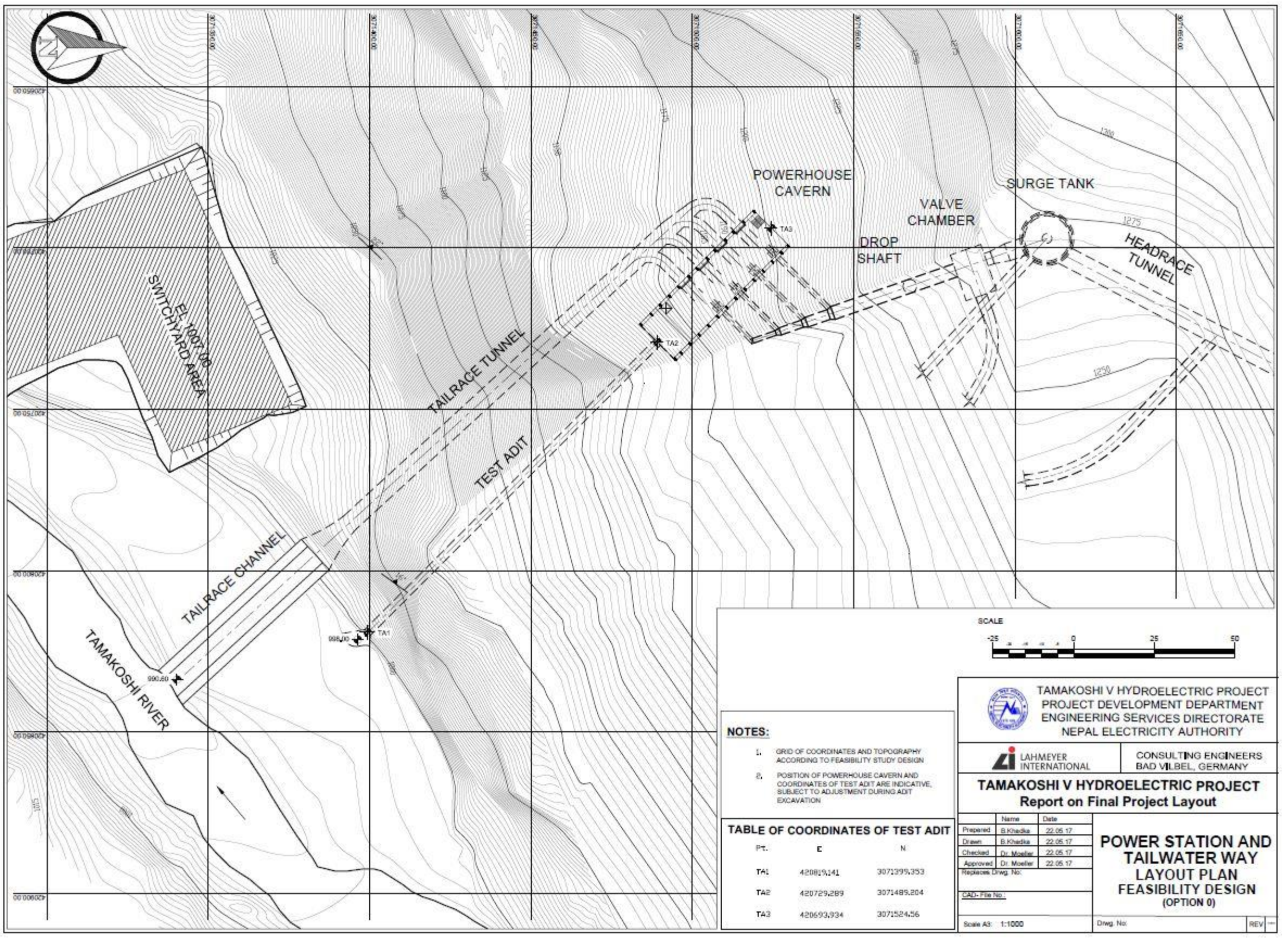

Figure 5-5: Layout Alternative 0 (as in Feasibility Study)

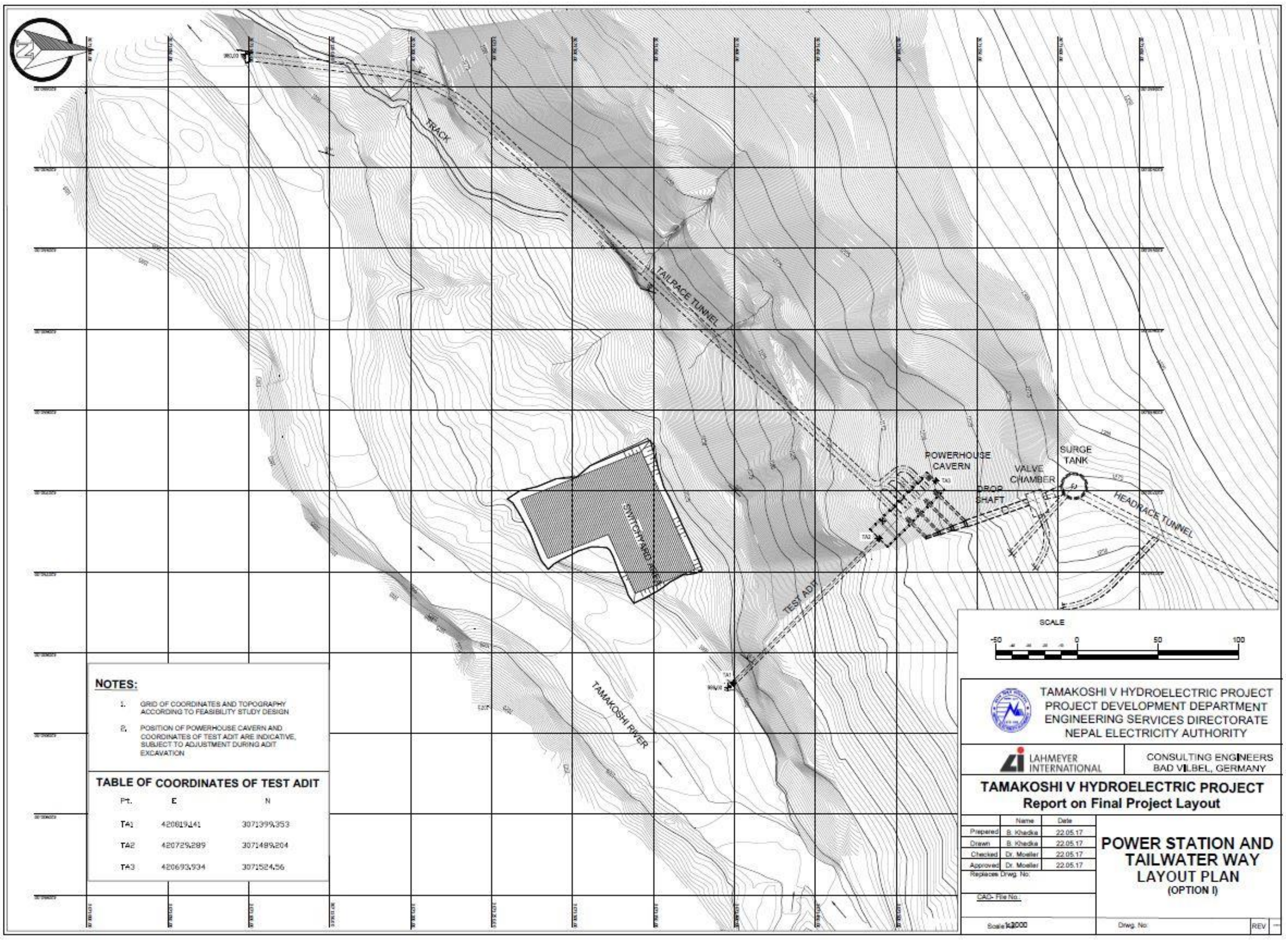

Figure 5-6: Layout Alternative 1

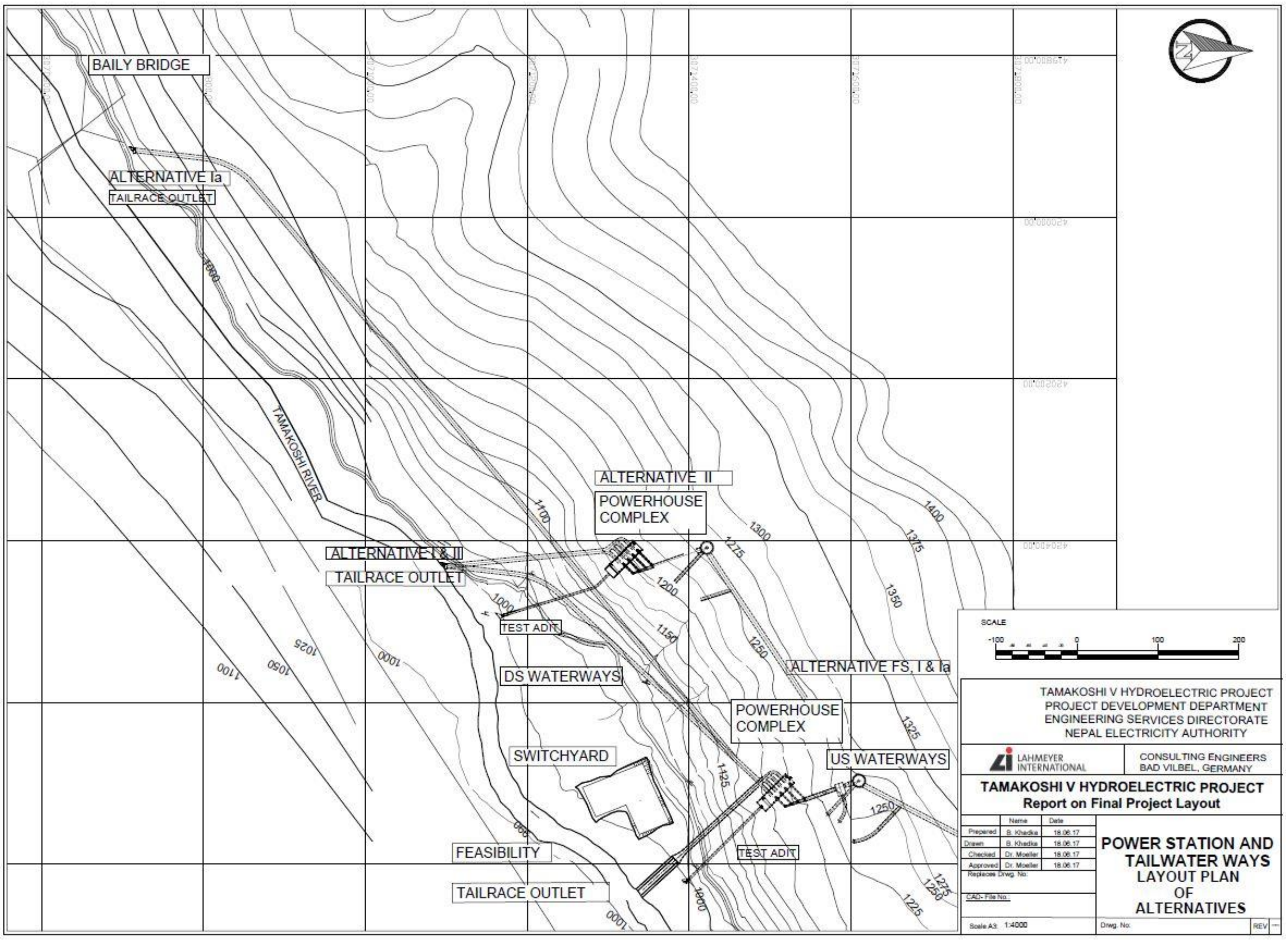

Figure 5-7: Layout Alternative 1a

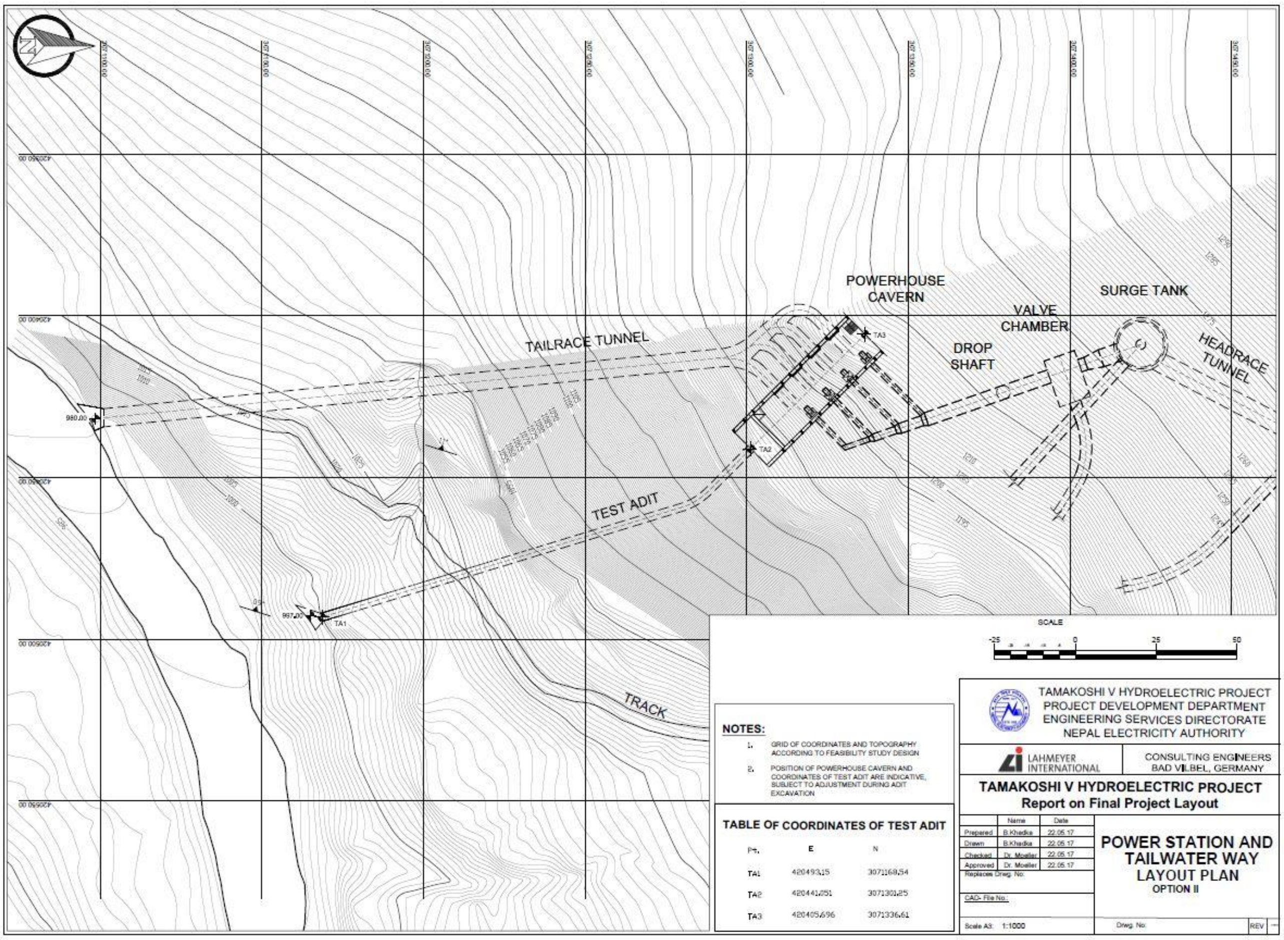

Figure 5-8: Layout Alternative 2

6 HYDRAULIC DESIGN

6.1 Headworks Area

6.1.1 Introduction

The Headpond is one of the salient features of the Tamakoshi V HEP. The Headpond was planned to receive the outflow from the Tailrace Tunnel of the Upper Tamakoshi hydroelectric project (UTK HEP) which was, at the outset, designed to have an outfall at the Tamakoshi River. During planning of the Tamakoshi V HEP it was decided that the outflow from the Tailrace Tunnel should, alternatively, be diverted towards the Headpond through a the Connecting Tunnel. The flow in the Connecting Tunnel is a free surface channel flow, as it is in the UTK HEP tailrace tunnel, it discharges into the Headpond to create a head for the Tamakoshi V HEP. Therefore, the inflow into the Headpond depends entirely upon the operational release from UTK HEP. The sketch showing the plan of the Headpond and Connecting Tunnels is displayed below:

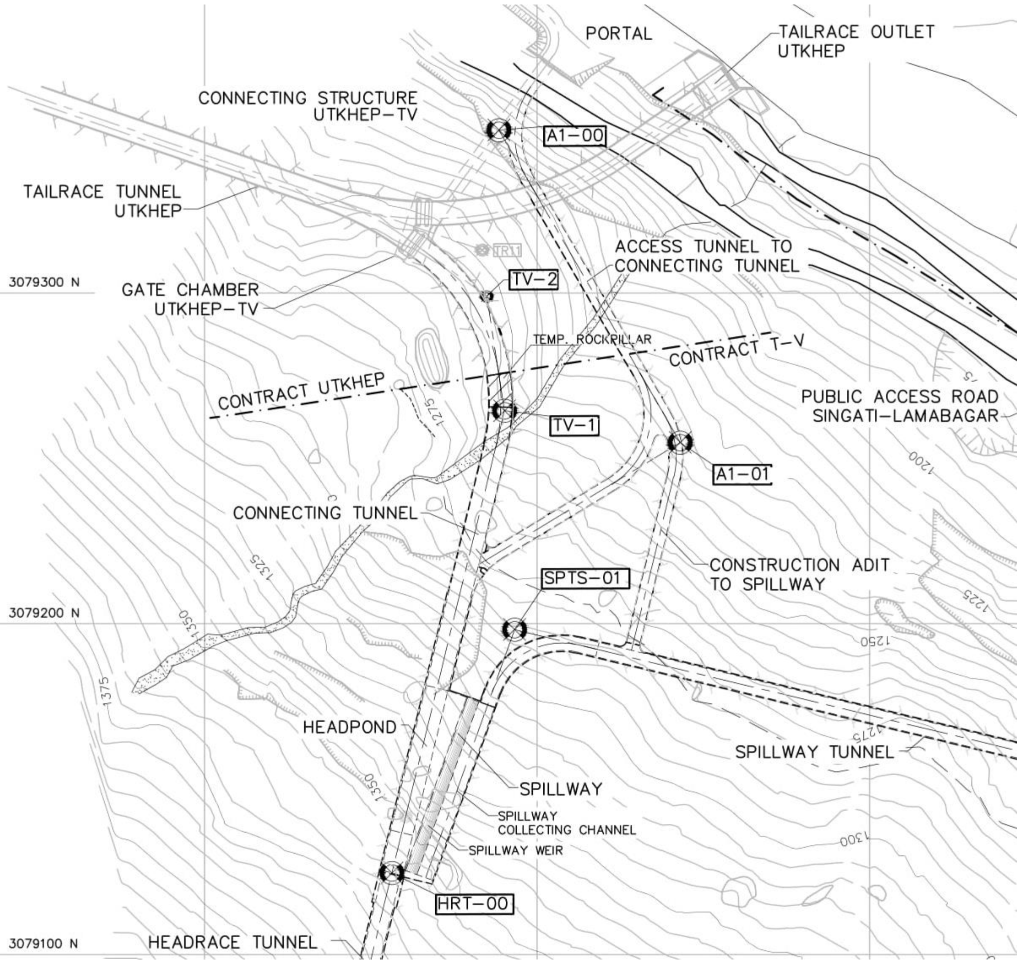

Figure 6-1: Sketch of the plan of Headpond and related structures.

The Headpond was proposed with a lateral spillway weir foreseeing the following scenarios:

- The Tamakoshi V water conductor system does not have any outlet provisions between the Headpond and the turbines. Therefore, during a plant shutdown condition of the Tamakoshi V and the UTK HEP is still discharging water through the Connecting Tunnel to the Headpond it is essential to release the water from the system; and

- during the transient behavior of flow in the water conductor system of Tamakoshi V, when UTK HEP is still discharging through the Connecting Tunnel to the Headpond, it it is essential to release the water from the system in order to avoid abrupt back propagation of flow into the Connecting Tunnel, which might cause a malfunction of the UTK HEP.

The Headpond was optimally designed so that during any operational condition, when the system encounters the aforementioned scenarios, it will function adequately. Hence, an outflow system in the form of a lateral spillway at the Headpond was proposed.

6.1.2 Headpond: Hydraulic Design Criteria

The Headpond size was tentatively adjusted to meet the submergence criteria. Other design criteria were set and examined to optimize the size of the Headpond. The design criteria are described in the subsequent sections.

6.1.2.1 Area-Capacity Curve

The area-capacity curves have been calculated from the geometry of the Headpond. The results of the same are displayed below:

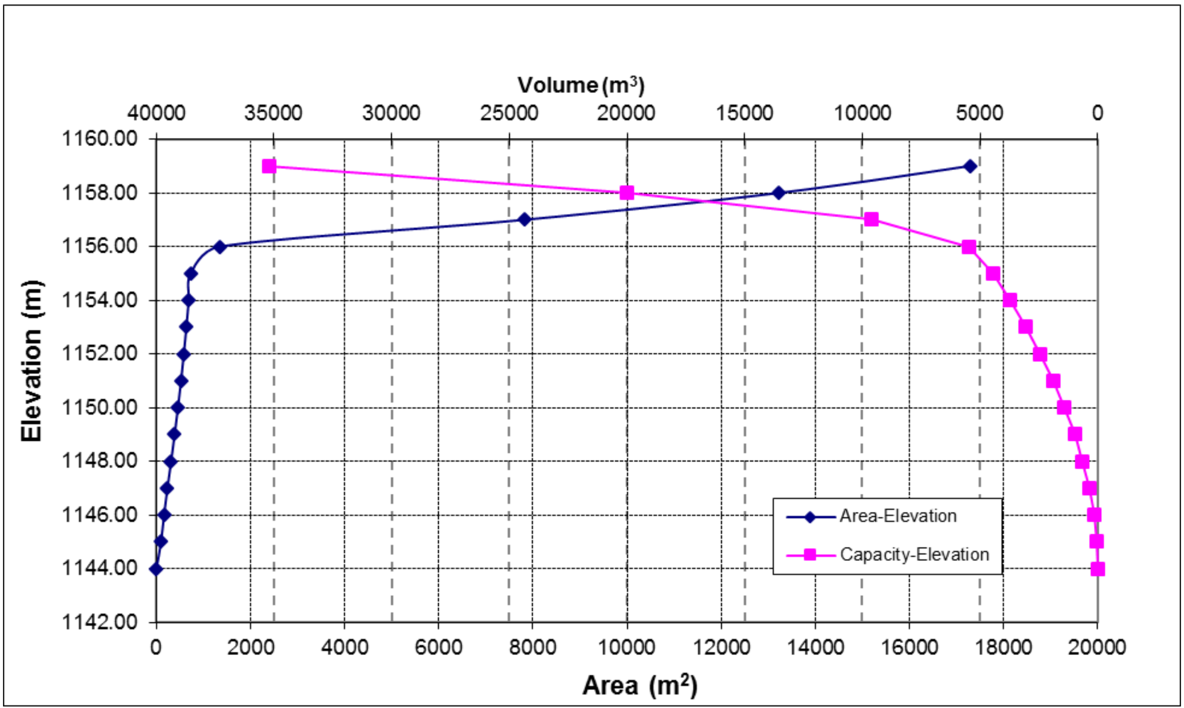

Figure 6-2: Area-Capacity Curves of the Headpond

The above figure is indicative of the capacity of the Headpond available at a particular elevation.

6.1.2.2 Submergence for Intake

Since the intake will receive controlled flow from UTK HEP, it is necessary to check the normal depth of flow at the intake when it receives the maximum release of Q = 68 m³/s from UTK HEP. The same has been simulated using the steady state model HEC-RAS by setting up the model with given geometry and Manning's roughness (n=0.014). This concurs with the previous study [1]. It was observed from the results that, at Q = 68 m³/s the water surface elevation at the Headpond reaches EL 1155.08m.

The model, described in the subsequent subchapter, was utilized for simulating the transient flow behavior in the Headpond. The resulting rating curve for the aforesaid model is displayed in the figure below.

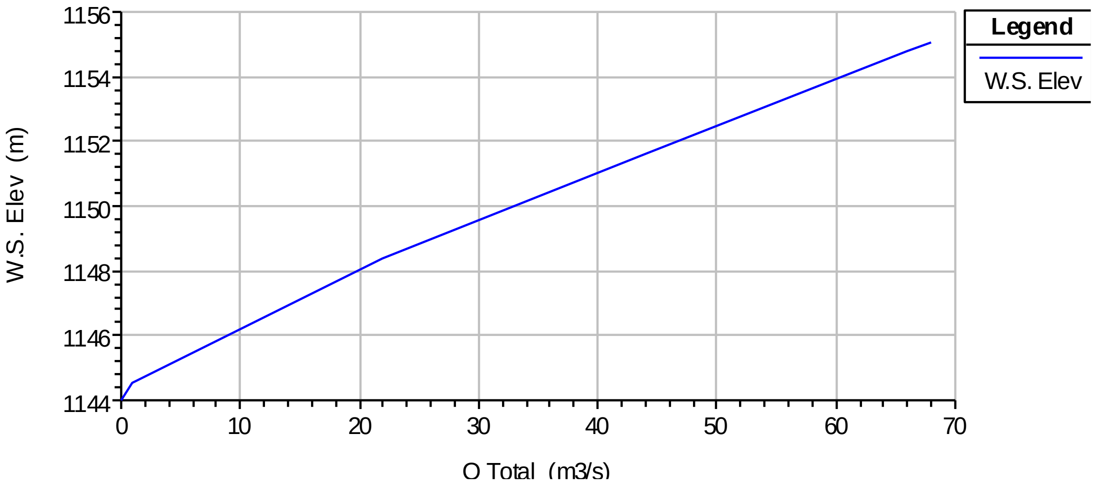

Figure 6-3: Area-Capacity Curve of the Headpond

6.1.2.3 Spillway Weir and Tunnel

General

The purpose of the Spillway Weir and Tunnel is to evacuate water surges, which can result from a sudden shut-down of Tamakoshi V. Details regarding the operation and the determination of the underlying maximum flow rate during a surge can be found in the transient study under Chapter 2.3. AMain result of this transient study yielded a maximum flow rate of 102.9 m³/s, which will serve as the design discharge for the surge evacuation system, which comprises (1) the lateral Spillway Weir and (2) the Collecting Channel followed by the Spillway Tunnel.

The rationale behind the hydraulic design of the tunnel is to avoid supercritical flow within the collecting channel and the subsequent horizontal bend of the alignment. Supercritical flow in bends or curves creates cross waves (aka shockwaves) which are unfavourable in tunnels as they reduce the freeboard and create wavy, dynamic water surfaces. Subcritical flow is maintained by limiting the longitudinal channel slope in the collecting channel and horizontal bend to 0.5%. Slightly downstream from the horizontal bend, as illustrated in drawing Q 1231, the longitudinal bed slope increases to 1%, which then induces supercritical flow.

Dimensioning of the lateral Spillway Weir

At the outset of the hydraulic design of the surge evacuation system, the required minimum crest length of the Spillway Weir was calculated. The crest elevation of the weir was chosen to be 1,158.2 m asl in order to leave 0.2 metres freeboard on top of the rated water level in the Connecting Tunnel / Headpond. This free-board is ample to avoid water losses due to a wavy surface in the Connecting Tunnel / Headpond, which is unavoidable during regular operation. Moreover, the overflow head over the weir was designed to be 0.9 metres, which is one of the outcomes of the above mentioned transient analysis. For the type of weir, A WES-profile weir type was selected. This type is well-proven and known for its good performance while keeping the pressures along the ogee in a non-critical negative range.

With these constraints, the required weir length was calculated. The discharge Q follows the POLENI equation for unsubmerged flow across weirs (unsubmerged conditions are confirmed in the following Subsection 1.2.3.3):

$$Q = \frac{2}{3} \cdot \mu \cdot L \cdot H_e^{\frac{3}{2}} \cdot \sqrt{2g} \qquad \text{Eqn. (1)}$$

where:	Q	discharge capacity	[m³/s],
	H_e	overflow head	[m],
	μ	weir coefficient (0.74 for WES-type)	[-],
	L	effective length of the crest	[m].

The effective length L of the crest was determined by the following equation[2]. This equation takes the effects of piers and abutments into account, which reduce the discharge capacity due to two-dimensional effects on the approaching flow pattern:

$$L = L' - 2(PK_p + K_a) \cdot H_e \qquad \text{Eqn. (2)}$$

where:	L’	net length of the crest	[m],
	P	number of piers	[-],
	K_p	pier contraction coefficient	[-],
	K_a	abutment contraction effect	[-].

Since there are no piers on the weir crest, the equation can be reduced to:

[2] USBR, 1987: Design of Small Dams

$$L = L' - 2 \cdot K_a \cdot H_e \qquad \text{Eqn. (3)}$$

With an abutment contraction coefficient of K_a = 0.2, an effective length of 55.5 metres results in a discharge of 102.9 m³/s at a surcharge of 0.9 m. For a length of 55.5 metres and the entire range of overflow head between 1,158.2 and 1,159.1 m asl, the capacity curve as shown in the figure below was calculated applying equation 1.

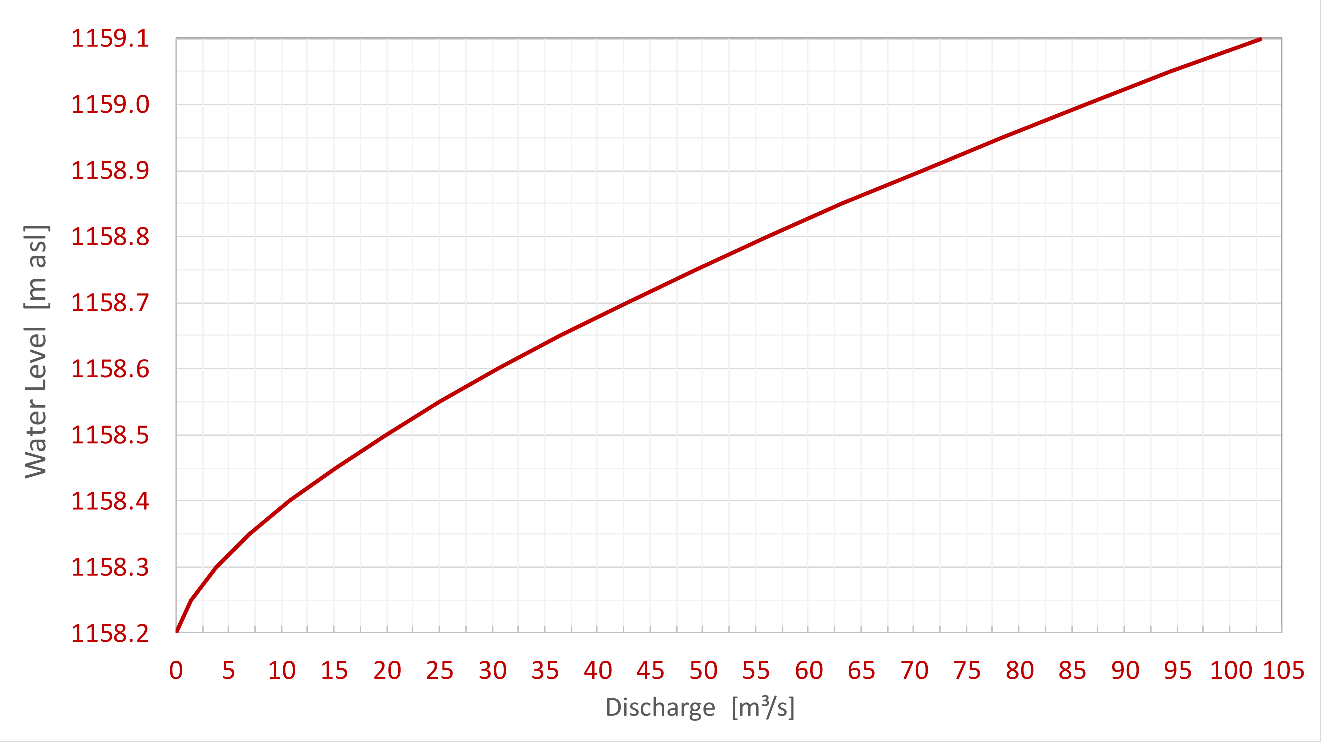

Figure 6-4: Capacity-elevation curve for the lateral Spillway Weir

Design of the Collecting Channel and the Spillway Tunnel

To determine the dimensions of the Collecting Channel and the Spillway Tunnel, the well-known one-dimensional numerical model HEC-RAS was employed. The primary focus of this HEC-RAS model was on the Spillway Tunnel, where the flow conditions are clearly one-dimensional and can be investigated very well with a one-dimensional model, whereas, the water levels in the collecting channel are highly three-dimensional because of the lateral inflow. Therefore, the water levels calculated for the collecting channel shall be considered as an estimation and subject to further investigation by the EPC contractor in a physical or numerical 3D model.

The longitudinal profile through the Spillway Tunnel is shown in Figure 1-25. The plot shows that the water level at the U/S end of the collecting channel (1,157.9 m asl) is well below the weir crest level of 1,158.2 m asl. This water level ensures that the flow over the ogee is unsubmerged. Moreover, the plot shows the acceleration of the flow from the collecting channel to the exit of the tunnel. Based on the present computations, the dimensions of the collecting channel (bed width 5 metres) and the tunnel (4.4 metres) are confirmed.

With a maximum of 7.57 m/s, the flow velocities in the Spillway Tunnel are well below the critical velocity (of about 25 – 30 m/s) beyond which aeration of the chute is needed to prevent cavitation.

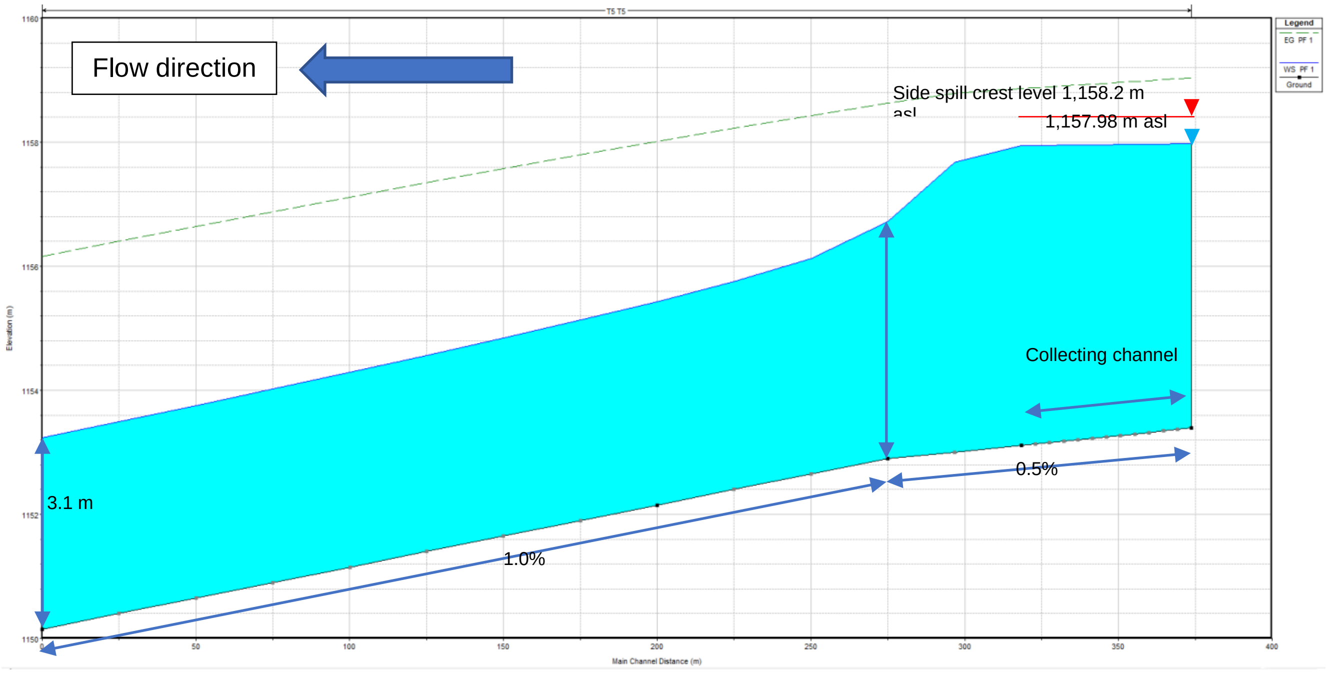

Figure 6-5: Longitudinal section through the Spillway Tunnel and the water surface

6.1.2.4 River Rating Curve at Outlet Portal of the Spillway

The river reach at the Outlet Portal, based on the survey data, was modelled using HEC-RAS. The steady state model was developed for the reach. The geometry file of the river reach, cross section station and geometries, as well as the downstream reach lengths of the channel and overbanks for each cross section were utilized. The topography of the area was extracted from the survey drawings. A section of Tamakoshi River was defined as the study area. The river geometry was defined by the alignment and cross sections of the river, created with civil 3-D and exported as GIS data as *.geo file. The cross sections were extended to ensure that all water from the flood was confined in the cross sectional area.

Overbank geometry - topographic data

For the overbank geometry, the topographic maps provided by surveyors were used.

Final HEC-RAS cross sections

Final HEC-RAS cross sections were extracted using the field survey data and the topographic data. The schematic alignment of the river and the cross-section are displayed in the figure below.

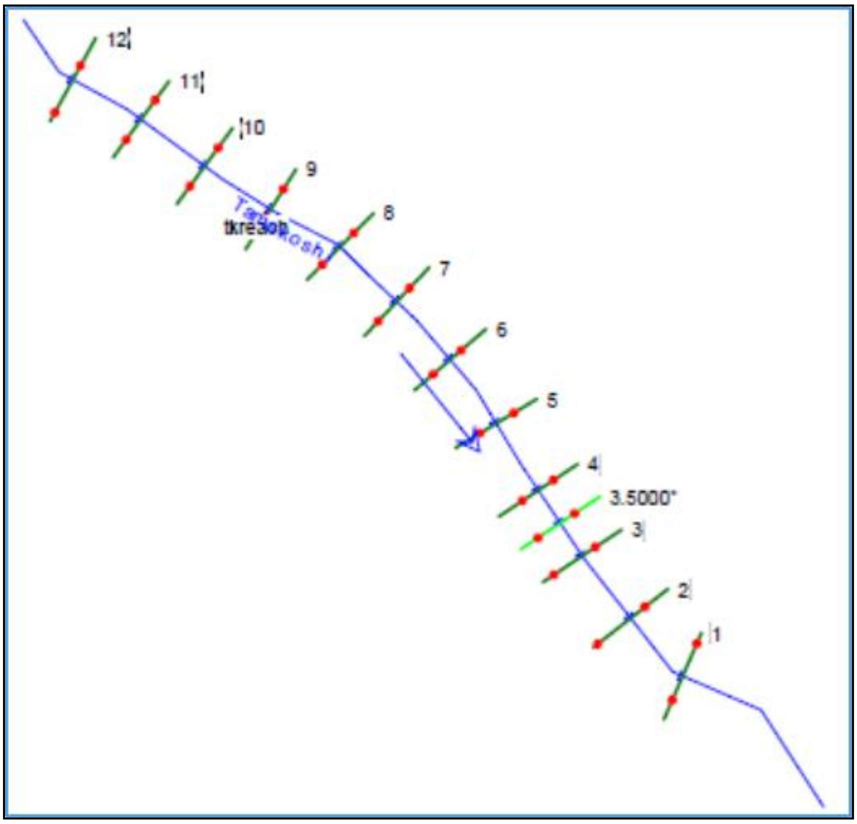

Figure 6-6: Spillway Outlet Reach (XS-3.5*)

In the above figure, the cross-section number 3.5* represents the section closest to the spillway outlet.

Flow Resistance or Roughness

The Manning's n-value was used to help calculate the energy losses between cross sections due to friction. The Manning's n-value depends on a number of factors which include: surface roughness; vegetation; channel irregularities; degree of meander; obstructions; size and shape of the channel. For the present study, the reach was assigned different Manning's n values for the channel and overbank flow areas.

Manning's n values for the HEC-RAS model were determined with reference to the HEC-RAS User's Manual (USACE, 2008). The 'n' values of 0.045 for the overbanks and 0.06 for the channel as recommended by the User's Manual is consistent with the 'n' values adopted in previous studies [1].

Boundary conditions

The HEC-RAS models were run under the assumption of subcritical flow. Normal flow depth was chosen as the downstream boundary condition. Normal flow depth computations were based on energy slope which was approximated by the channel slope of the downstream reach as determined from topographic data.

The river slope near the project area was determined as follows.

Table 6-1: River slope near the Headpond and Spillway Terminal Structure of TK-V HEP

Location	Distance between upstream and downstream contours	u/s contour (m)	d/s contour (m)	Slope	Slope (%)
Headpond	1574	1175	1125	0.03176	3.17%

Steady Flow Data

Steady flow analysis was used to route five flows of 66, 300, 1,000, 1,700, and 2,500 m³/s through the river. This range was chosen because it represents; design flow, flow when Upper Tamakoshi HEP is shut down, and other flows close to Q_2 yr, Q_{25} yr and Q_{1000} yr floods.

Results

The rating curve of cross-section number 3.5* representing the section near the Spillway Outlet is displayed in the figure below. The resulting equation to determine the water level is $H = 0.1338\ Q^{0.5456} + 1146.29$.

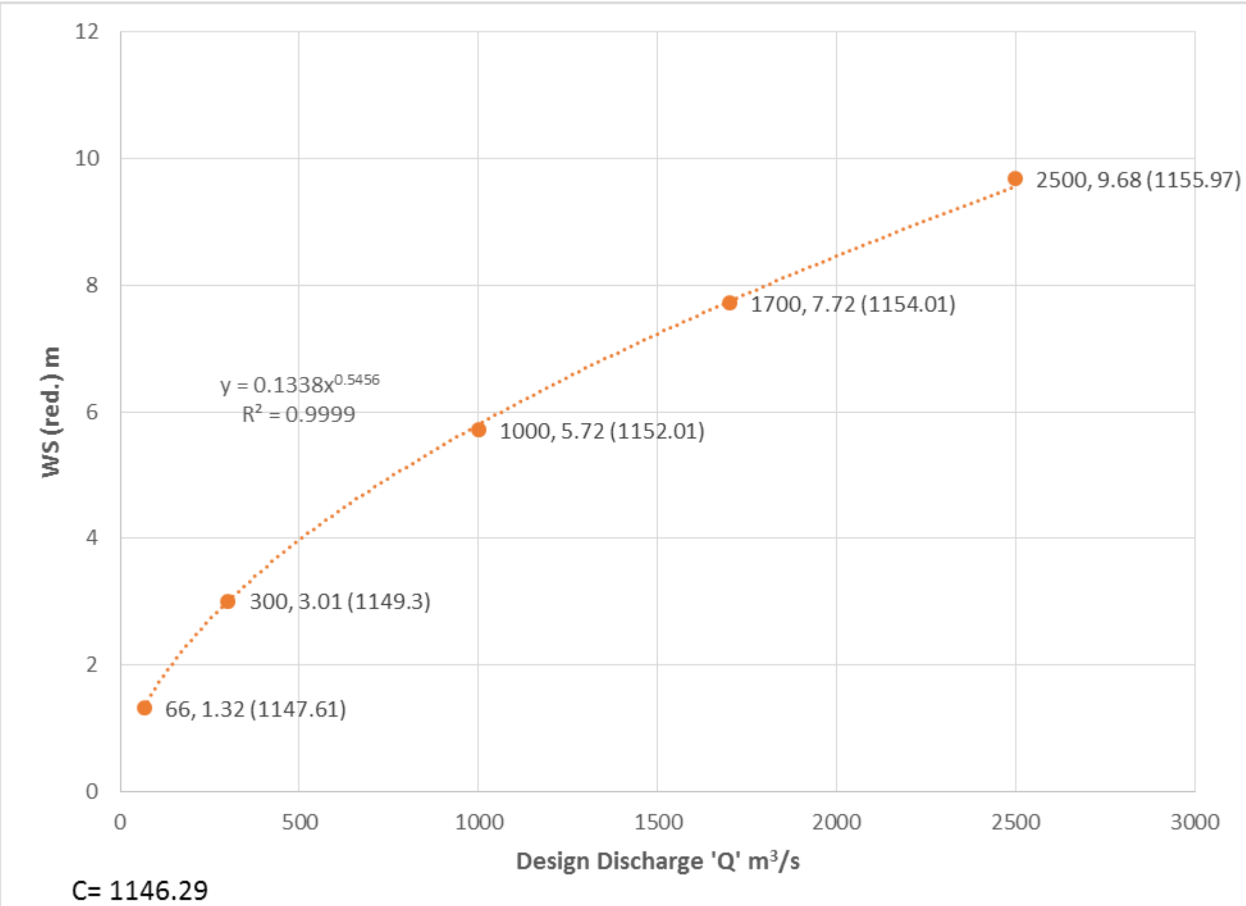

Figure 6-7: Rating Curve valid for Spillway Terminal Structure

6.1.2.5 Backwater Effect

The backwater development at the Headpond due to the sudden shutdown of Tamakoshi V HEP and the wave propagation towards the Connecting Tunnel were examined by developing a hydraulic model of the system from the outlet point of the UTK HEP to the end of the Headpond. Several numerical models using HEC-RAS were developed to check whether the hydraulic system is free from any abrupt change in flow depth in the Connecting Tunnel with optimizations of the size of the Headpond, the weir crest length, and the weir crest level. The details of the model study are described in the subsequent subchapter. It is noted that only the accepted case of the model study, which resulted in optimum sizing of the Headpond is addressed in the description.

6.1.3 Hydraulic Modelling: Connecting Tunnel and Headpond

6.1.3.1 Model Setup

General

The following drawings were used as the basis for setting up the model:

- UTK HEP: Drawing No. UTK-012065-4810-S-002 Final Design Tailrace Tunnel (Longitudinal Section, Cross-Section and Detail; dated-11.11.2011; and
- Tamakoshi V HEP Detailed Engineering Design: Drawings Nos. 31-00053-DD-4410-Q 1201; - Q-1210, -Q-1215; -Q1216; -Q-1220 and -Q1221.Detail, dated-31.07.2017.

Applied Convention

The following conventions were applied:

- with reference to drawing, Drawing No. UTK-012065-4810-S-002, the starting elevation of the Connecting Tunnel is shown in Section-B-B, and the end elevation of the Headpond was obtained from Drawing No. 31-00053-DD-4410-Q 1220
- depending upon the sectional change in the Connecting Tunnel at varying intervals, the tunnel was divided into 8 cross-sections for model study, which were further divided/interpolated for model requirements;
- as the Connecting Tunnel and the Headpond are open channels, the one-dimensional numerical model HEC-RAS by USACE was used to model the system; and
- the critical condition in the Headpond will be the scenario when the Tamakoshi V plant shuts down suddenly while the UTK HEP is still discharging the maximum flow of 68m^3/s.
- the simulation date and time 05 September 0000 hours to 05 September 0025 hours are merely indicative of start time 0 sec to end time 1,500 sec.

Brief Description of HEC-RAS Software

HEC-RAS is designed to perform one-dimensional hydraulic calculations for a full network of natural and constructed channels. The HEC-RAS model comprises two major components:

- steady flow component
- unsteady flow component

Steady flow component

The steady flow component of the modeling system is intended for calculating water surface profiles for steady gradually varied flow. The system can handle a single river reach, a dendritic system, or a full network of channels. The steady flow component is capable of modeling subcritical, supercritical, and mixed flow regime water surface profiles.

The basic computational procedure is based on the solution of the one-dimensional energy equation. Energy losses are evaluated by friction (Manning's equation) and contraction/expansion (coefficient multiplied by the change in velocity head). The momentum equation is utilized in situations where the water surface profile is rapidly varied. These situations include mixed flow regime calculations (i.e., hydraulic jumps), hydraulics of bridges, and evaluating profiles at river confluences (stream junctions). The effects of various obstructions such as bridges, culverts, weirs, spillways and other structures in the flood plain may be considered in the computations. The steady flow system is designed for application in flood plain management and flood insurance studies to evaluate floodway encroachments. Also, capabilities are available for assessing the change in water surface profiles due to channel improvements, and levees.

Unsteady flow component

The unsteady flow component of the HEC-RAS modeling system is capable of simulating one-dimensional unsteady flow through a full network of open channels. The unsteady flow equation solver was adapted from Dr. Robert L. Barkau's UNET model (Barkau, 1992 and HEC, 1997). This unsteady flow component was developed primarily for subcritical flow regime calculations. The hydraulic calculations for cross-sections, bridges, culverts, and other hydraulic structures that were developed for the steady flow component were incorporated into the unsteady flow module. Additionally, the unsteady flow component has the ability to model storage areas and hydraulic connections between storage areas, as well as between stream reaches.

The equations derived by Barkau are the basis for the unsteady flow solution within the HEC-RAS software. The implicit finite difference equations encrypted in HEC-RAS representing the conservation of mass and momentum are as follows:

- the conservation of mass (continuity) equation,

$$\Delta Q + \frac{\Delta A_c}{\Delta t}\Delta x_c + \frac{\Delta A_f}{\Delta t}\Delta x_f + \frac{\Delta S}{\Delta t}\Delta x_f - \overline{Q}_1 = 0 \qquad \text{Eqn (4)}$$

- the conservation of momentum equation,

$$\frac{\Delta\left(Q_c \Delta x_c + Q_f \Delta x_f\right)}{\Delta t \Delta x_e} + \frac{\Delta(\beta V Q)}{\Delta x_e} + g\overline{A}\left(\frac{\Delta z}{\Delta x_e} + \overline{S_f} + \overline{S_h}\right) = 0 \qquad \text{Eqn (5)}$$

where;

Q	=	discharge
$\overline{Q}_1$	=	average lateral inflow
A_c	=	channel flow area
A_f	=	floodplain flow area
A	=	average flow area
S	=	storage from non-conveying portions of cross section
x_c	=	distance along the channel
x_f	=	distance along the floodplain
x_e	=	equivalent flow path
S_f	=	average friction slope for the entire cross section
$\overline{S_h}$	=	average slope over the interval Δx_e
t	=	time
V	=	velocity
g	=	acceleration due to gravity
z	=	water surface elevation

The implicit finite difference equations above are transformed into their linear form and then Priessmann Box scheme encrypted in HEC-RAS simulates the model for output. The scheme is also known as four-point implicit scheme.

Cross-sections

The input data that was used in building the river model are listed with their assigned name as tabulated below:

Table 6-2: Cross-sections of Interconnecting Tunnel and the Headpond

Cross-sections (XS)	Distance from the immediate downstream XS	Remarks	Cross-sections (XS)	Distance from the immediate downstream XS	Remarks
XS 8	10	Connecting Tunnel Reach	XS 4	44.66	Headpond Reach
XS 7	7.8	Connecting Tunnel Reach	XS 3	17.00	Headpond Reach
XS 6	2853.71	Connecting Tunnel Reach	XS 2	32.54	Headpond Reach
XS 5	22.06	Headpond Reach	XS 1	0	Headpond Reach

For stability reasons, as the time scale of boundary condition is in seconds (discussed below in subsection 1.3.1.5), the cross-sections in the model were interpolated in an interval of 5m. The cross-sections used in HEC-RAS are presented in the Annex to Part A3 Chapter 1 of this report.

Roughness

The Manning's roughness n = 0.014 was adopted throughout the tunnel reach including the head pond. This selection of Manning's n concurs with the previous study of UTK HEP [2].

Unsteady Initial and Boundary Conditions

Initial condition

The initial flow in the model was 68 m³/s.

Upstream boundary condition

The upstream boundary condition is the inflow hydrograph with constant flow of 68 m³/s for the duration of time equal to 1,500 sec at an interval of 1 sec.

Downstream boundary condition

The transient flow time series (duration 1,500 sec) at the Headpond due to the load rejection scenario for the minimum roughness case, which is explained in detail in the subsequent chapter, was the downstream boundary condition for the present study.

The upstream and the downstream boundary conditions are displayed in the figure below:

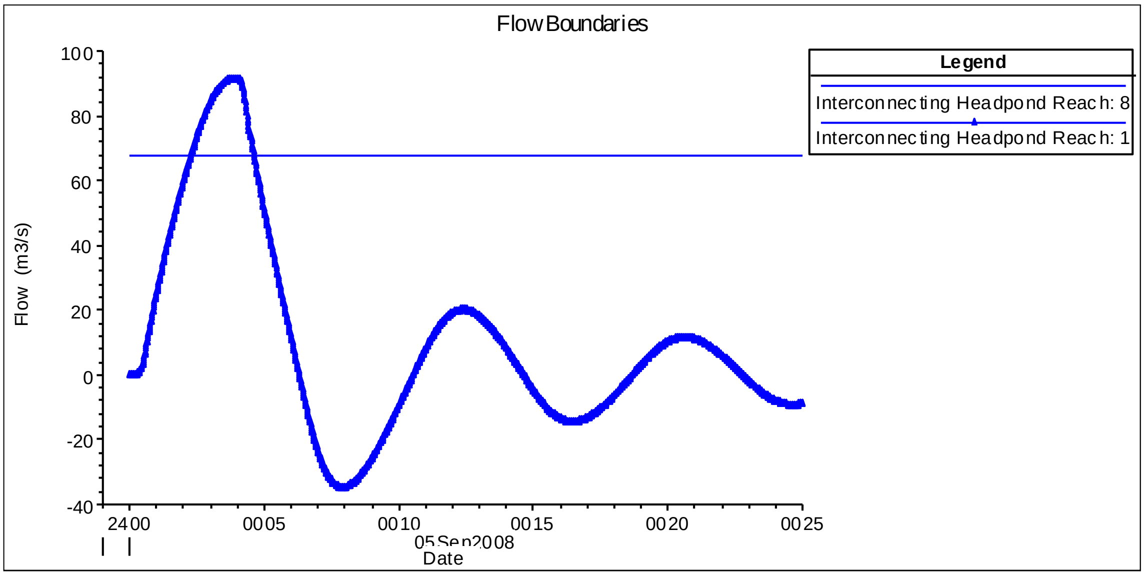

Figure 6-8: Upstream and downstream boundary conditions

With the aforementioned conditions the model was simulated. The results are discussed in the subsequent subchapter.

Lateral Spillway Weir at the Headpond

After several iterations, with changed dimensions of the Headpond and taking into account the side spillway, the crest level of the lateral weir was finally fixed at EL 1158.1 with the required length of the crest of 50 m. The schematic plan view and longitudinal section of the weir in the HEC-RAS model are displayed in the figures below.

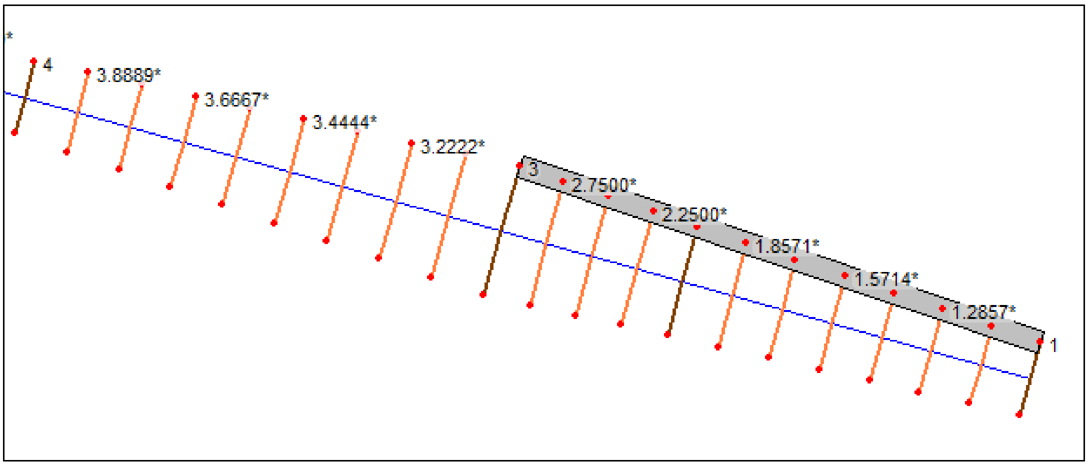

Figure 6-9: Schematic Plan View of the Side Spillway in HEC-RAS Model

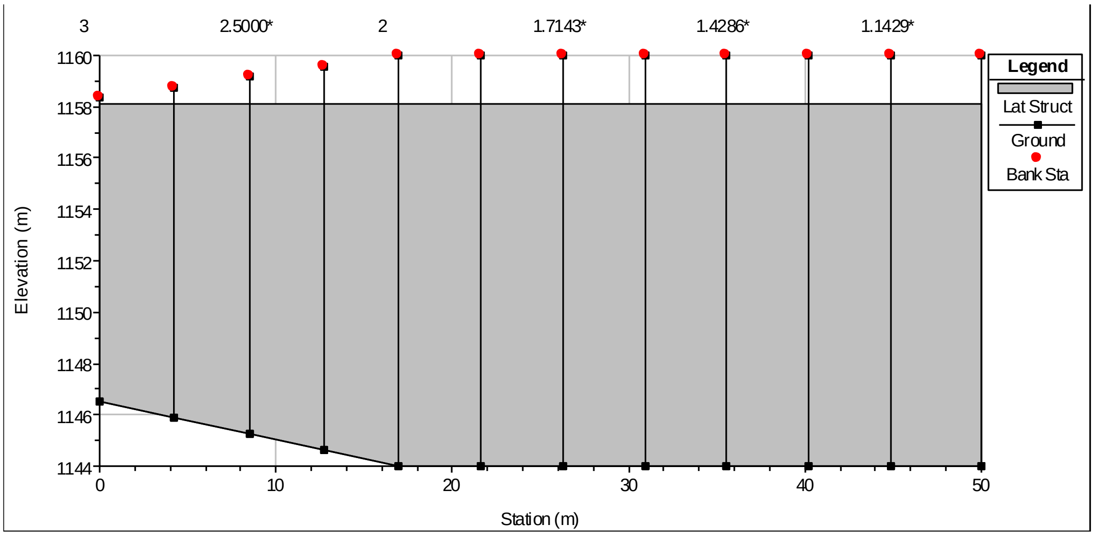

Figure 6-10: Longitudinal Section of Headpond featuring the Side Spillway in HEC-RAS Model

6.1.3.2 Results and Discussions

The unsteady model, discussed above, was simulated for a simulation period of 1,500 sec. The longitudinal profile with maximum water surface elevations along the UTK HEP tailrace tunnel and the Headpond is displayed in the figure below.

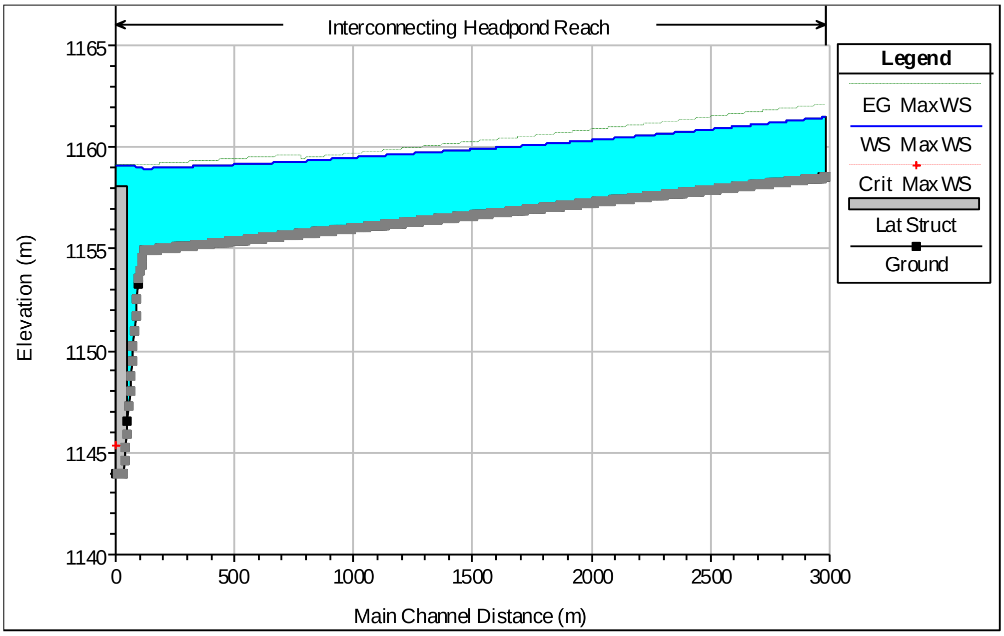

Figure 6-11: Maximum Water Surface Profile in the Reach from UTK HEP tailrace tunnel to Headpond

The above water surface profile is further elaborated in the figure below.

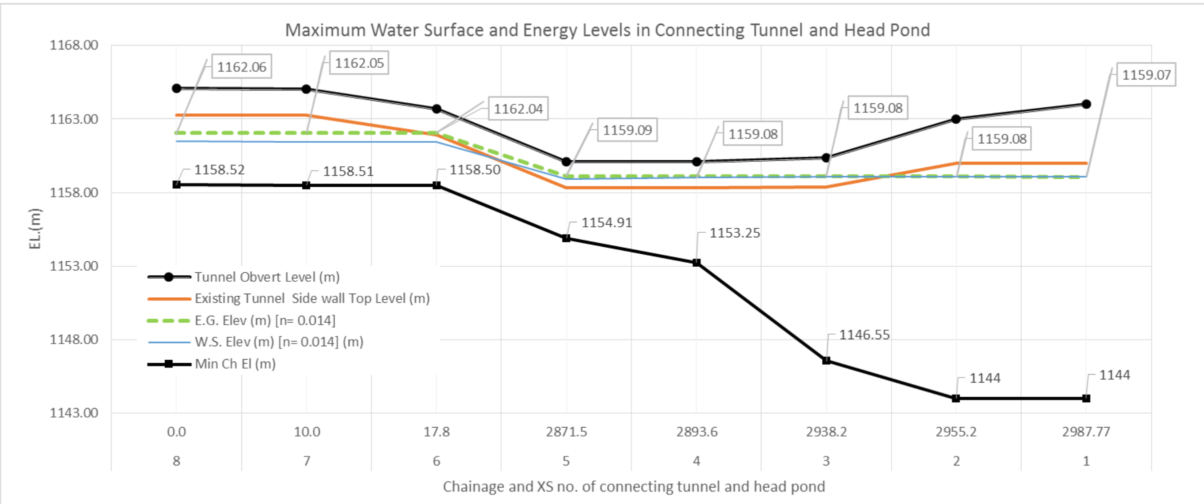

Figure 6-12: Maximum Water Surface Profile with Details of the Connecting Tunnel and Headpond

The above figure displays the invert and obvert levels of the Connecting Tunnel and the Headpond. It is noted that the minimum cover, from the top of the side walls to the obvert levels of the tunnel, is 1.8 m. Since the model was developed considering the scenario of the transient condition ,which is critical for the Headpond, the resulting maximum water surface profile does not exceed the maximum tailwater level EL

1162.35m of UTK HEP. The available freeboard in the tunnel, with reference to the maximum water surface levels, ranges from 1.1m to 4.89m, which is acceptable and the modelled size of the Headpond meets the critical condition scenario.

The results of the simulated time series of flow and stage at XS-8 and XS-1 are displayed in the figures below.

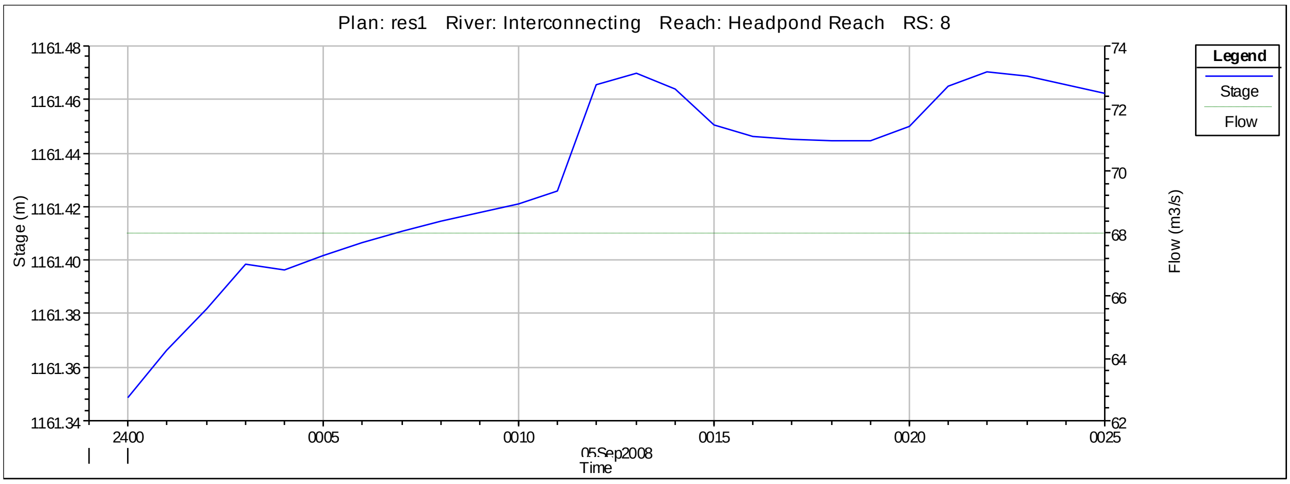

Figure 6-13: Results of Simulated Stage-Flow Time Series (time shown in minutes) at XS-8 (the start of the Connecting Tunnel)

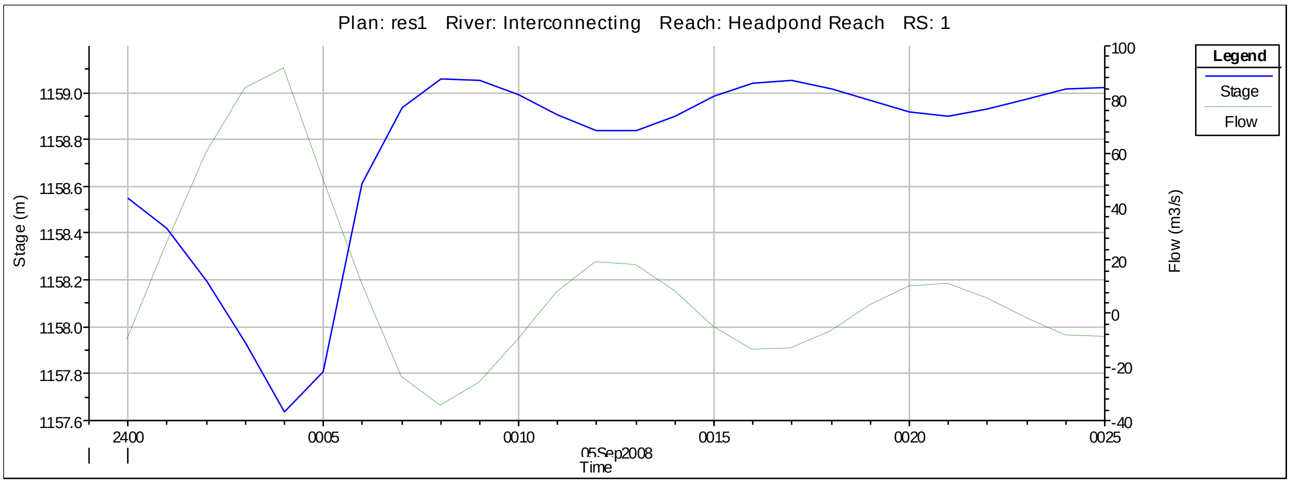

Figure 6-14: Results of Simulated Stage-Flow Time Series (time shown in minutes) at XS-1 (the end of the Headpond)

The maximum stage resulted with a Headpond elevation of 1159.1m. For the purpose of design of Spillway weir and the appertunances, the maximum inflow (Q = 68m^3/s) from the Tailrace outlet of the UTK HEP and the maximum transient flow (Q =34.9 m^3/s) towards the Headpond from Tamakoshi V Headrace Tunnel were summed together for a total discharge of, Qd = 102.9 m^3/s. It is advisable to use this discharge for the design work..

6.1.4 Conclusion

The following have been concluded:

- the length of the Headpond shall be 96.07 m;
- the invert level in the Headpond near the intake shall be EL. 1144.00 m;
- for Tamakoshi V operations, the operating water level of EL.1158.00 m is confirmed;
- the Spillway Weir crest level shall be 1158.20 m;
- the crest length of the weir shall be 55.5 m; and
- the design discharge through the Spillway Weir shall be 102.9 m^3/s.

6.2 Water Conveying Tunnels and Surge Tank

6.2.1 General Description

The Tamakoshi V Hydroelectric Project envisages utilization of the discharge released through the Upper Tamakoshi HEP tailrace tunnel (open channel), which joins a Connecting Tunnel to the Headpond of Tamakoshi V. The highest regulated water level (HRWL) of EL 1158.00 m was proposed.

The main components of the water conveying systems in the Tamakoshi V project are:

- Headrace Tunnel (HRT);
- Surge Tank
- U/S Valve Chamber
- Pressure Shaft
- High Pressure Tunnel & Upstream Manifolds
- Downstream Manifolds and
- Tailrace Tunnel (TRT)

The total length of the concrete lined Headrace Tunnel (HRT) is 8,116 m. The diameter of the HRT is 5.60 m. The concrete lined portion of the tunnel terminates just D/S of the Surge Tank and from there becomes steel lined. From the Surge Tank, the discharge continues through to the Valve Chamber and then descends nearly 140 m before continuing horizontally to the first u/s bifurcator. The length of this steel lined section is 200 m.

The Pressure Shaft divides into 4 branches (three branches are 2.425 m diameter and one branch is 0.95 m diameter). The discharge through the turbines is led through the downstream manifold and the Tailrace Tunnel of 5.60 m diameter to the Outlet Structure. The length of the Tailrace Tunnel to the beginning of the Outlet Structure is 404 m. From here the water passes through the Outlet Structure where the turbine water is returned into the Tamakoshi River.

A schematic view of the water conveying tunnels is displayed below:

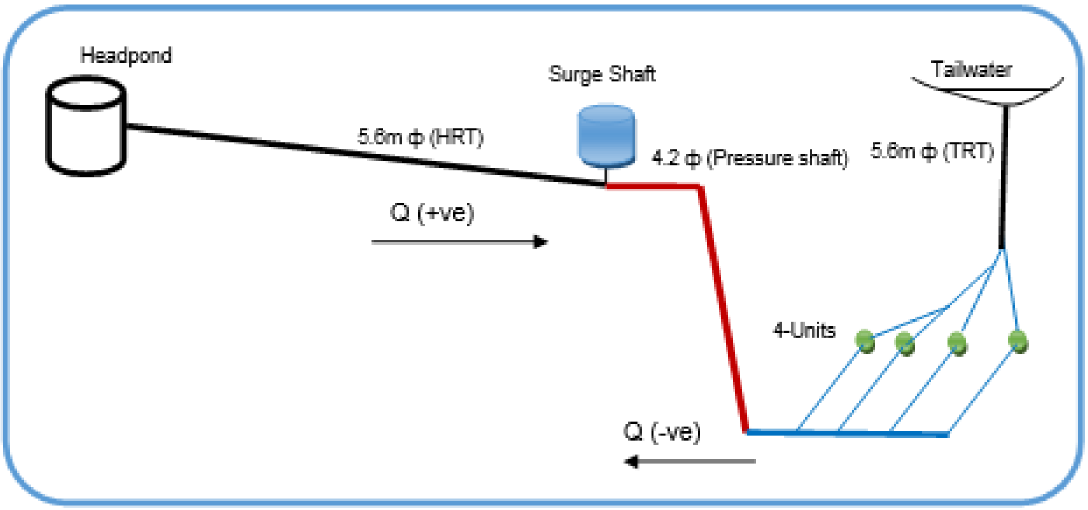

Figure 6-15: Schematic view of the water conveying system

6.2.2 Head Losses

The head loss for the project was calculated by summing the losses from the Intake at the Headpond to the Outlet Structure, which includes major components, such as Headrace Tunnel, Pressure Shaft, and Tailrace Tunnel. The major part of the head losses was due to friction in the conduits, whereas, the minor head losses was due to entry, bends, valves, contractions and expansions of the water conveying system.

6.2.2.1 Input Parameters

The input parameters utilized for head loss computations are listed in the table below:

Table 6-3: Input parameters

Parameter	Value	Unit
Highest regulated water level (HRWL)	1158.00	m
Normal tailwater level	984.00	m
Gross head	174.00	m
Design discharge	66.00	m^3/s
Diameter of headrace tunnel	5.60	m
Diameter of pressure shaft	4.20	m
Diameter of tailrace tunnel	5.60	m

6.2.2.2 Roughness

The Manning's roughness parameters for the types of water conveying conduits are indicated in the table below.

Table 6-4: Roughness parameters

Parameter	Value
Manning's 'n' for concrete lined tunnel [5]	0.014
Manning's 'n' for steel lined pressure shafts [5]	0.011

6.2.2.3 Loss Coefficients

The head loss coefficients due to various types of losses are listed in the table below.

Table 6-5: Coefficients of head loss

Parameter	Co-efficient, K
Gate grove slot loss [5]	0.1
Transition loss [5]	Varies with change in size of the conduit
Bend [5]	Varies with change in bend angles and radius of bend conduit
Valve [5]	0.1

6.2.2.4 Head Loss Computations

Friction Losses

The head losses due to friction have been worked out by applying the following equation,

$$h_f = n^2v^2L/R^{4/3} \qquad \text{...Eq. (1)}$$

where,

h_f = head loss due to friction (m);

n = Manning's roughness coefficient in the water conveying tunnel/conduit;

v = approach velocity of flow in the tunnel/conduit (m/s);

L = length of the tunnel/conduit (m); and

R = hydraulic radius of the conduit (m).

Minor Losses

The minor head losses due to entry, bend, valve, manifold, and gate groove are functions of velocity head ($v^2/2g$) , where v is the approach velocity [5]. The general form of the equation that is applicable for minor losses is given by,

$$h_{minor} = K_x\, v^2/2g \qquad \text{...Eq. (2)}$$

where,

h_{minor} = minor head losses (m);

K_x = Coefficients of head loss, where “x” represents entry, bend, valve, and fate groove;

v = approach velocity of flow in the tunnel/conduit (m/s); and

g = acceleration due to gravity (m/s^2).

Transition Losses

The transition losses are due to contraction and expansion of the conduit sections in the direction of flow.

Contraction Loss

The head loss due to contraction of conduit is given by,

$h_c = Kc\ (v_2^2 - v_1^2)/2g$ …Eq. (3)

where,

h_c = head loss due to contraction (m);

Kc = coefficients of contraction loss [5];

v_1 = approach velocity of flow at the start of the contraction (m/s);

v_2 = approach velocity of flow at the end of the contraction (m/s); and

g = acceleration due to gravity (m/s^2).

Expansion Loss

The head loss due to contraction of conduit is given by,

$h_{ex} = Kex\ (v_1 - v_2)^2/2g$ …Eq. (4)

where,

h_{ex} = head loss due to expansion (m);

K_{ex} = coefficients of expansion loss [5];

v_1 = approach velocity of flow at the start of the expansion (m/s);

v_2 = approach velocity of flow at the end of the expansion (m/s); and

g = acceleration due to gravity (m/s^2).

Exit Loss

The exit loss, in the present study, was the case of expansion loss as sectional length of exit system expands, thereby, inducing recovery of loss.

6.2.2.5 Results of Head Loss Computations

The head loss up to the turbine axis was calculated as 10.3 m, whereas, the head loss from the Tailrace Tunnel to the Outlet Structure was calculated as 0.8 m. The total head loss for Tamakoshi V HEP was calculated as 11.1 m.

The summary of the calculated head losses is listed in the table below.

Table 6-6: Summary of head loss computations

S.N.	Components of Head Loss	Head Loss	Unit
A.	**Head Loss from Intake to Head Race Tunnel**		
	Head Loss in Intake		
	Entry Losses	0.014	m
	Transition (Expansion) Loss (From Rectangular to Circular Section)	0.00016	m
	Total Losses in Intake	0.014	m
	Head Loss in Headrace Tunnel		
	Frictional Loss in Headrace Tunnel	7.298	m
	Head Loss in HRT Bends		
	Bend-1	0.030	m
	Bend-2	0.030	m
	Total Losses in HRT Bends	0.060	m
	Head Loss in HRT	**7.372**	**m**
	Contraction Loss (from HRT to Pressure Shaft Circular Shape)	**0.1740**	**m**
	Total Losses from Intake to the End of HRT , H1	**7.55**	**m**
B.	**Head loss in Pressure Shaft (Steel Lined)**		
	Frictional Losses up to Valve House	0.064	m

S.N.	Components of Head Loss	Head Loss	Unit
	Losses in Valve Passage	0.12	m
	Frictional Losses up to Manifold	0.42	m
	Bend Loss 1	0.139	m
	Bend Loss 2	0.139	m
	Head loss in Upstream Manifold		
	Loss in Branch 1	0.116	m
	Friction Loss after bifurcation (Segment 1)	0.022	m
	Loss in Branch 2	0.104	m
	Friction Loss after bifurcation (Segment 2)	0.038	m
	Loss in Branch 3	0.116	m
	Friction Loss after bifurcation (Segment 3)	0.0970	m
	Contraction Loss in Reducer	1.429	m
	Total Losses in Pressure Shafts, H2 =	**2.79**	**m**
C.	**Head Loss in Draft Tube and Downstream Mani-fold**		
	Frictional Loss in Drop Section	0.23	m
	Bend-1	0.254	m
	Frictional Loss in Elbow	0.028	m
	Transition Loss in Elbow	0.0207	m
	Frictional Loss in Rectangular Duct	0.01	m
	Transition (Expansion) Loss from duct to circular	0.002	m
	Frictional Loss in Manifold Branch 1	0.019	m
	Frictional Loss in Manifold Branch 2	0.007	m
	Total Losses in Draft Tube and Manifold, H3 =	**0.31**	m
D.	**Head Loss in Tailrace Tunnel System**		

S.N.	Components of Head Loss	Head Loss	Unit
	Frictional Loss in TRT	0.40	m
	Head Loss in TRT Bends		
	Bend-1	0.027	m
	Total Losses in TRT Bends	0.027	m
	Head Loss in TRT	0.43	m
	Head Loss in Outlet Structure Arrangement		
	Head Losses in Transitions (from Circular to Rectangular)	0.0000	m
	Frictional Loss in Rectangular Conduit	0.003	m
	Gate Groove Losses	0.021	m
	Exit Losses	0.0014	m
	Head Loss in Transitions and Rectangular Conduit including the Exit Loss	0.026	m
	Total Losses in TRT System, H4 =	**0.45**	m
	Head Losses in the Upstream Water Conveying System (H1+H2) = HLus =	**10.3**	**m**
	Head Losses in the Downstream Water Conveying System (H3+H4) = HLds =	**0.8**	**m**
	TOTAL HEAD LOSSES, HL_{TOTAL} =	**11.1**	**m**

The details of the head loss computations are displayed in Annex A to Part A3 Chapter 2 of this report.

6.2.3 Transient Analysis: Water Conveying System

6.2.3.1 Objectives

The objectives of the present study are:

- to examine the requirement of surge shaft or other transient system in the project specified location;
- to determine the size of surge shaft, taking into consideration the safety of water conveying system and to derive the maximum upsurge and minimum down surge in the surge shaft;
- to find the maximum pressure in the penstock near turbine resulting from sudden shutdown;
- to check the water conveyance system for negative pressures; and
- to judge proper operating time of turbines at load rejection and load acceptance cases.

6.2.3.2 Applied Conventions

The following conventions were applied:

- The water conveying tunnels were named as upstream water conveying system from the start of the HRT to the turbine, whereas, the system from the turbine axis to outlet structure via tailrace tunnel was named as the downstream water conveying system;
- For simplicity, the word "unit" in the report refers to turbine, wherever they are contextually mentioned;
- For simplicity, the upstream water conveying system was named Model-1, whereas, the downstream water conveying system was named Model-2;
- The transient models were set up by including all the units, 3 main units +1 additional small unit, in operation.
- Since the maximum release from UTKHEP project is 68 m^3/s, the transient models have been set up for various cases of load rejection and load acceptance with a given input of 68 m^3/s; and
- Since the rated discharge of the additional small unit is 3.35 m^3/s, two of the main units shall run with a rated discharge of 22 m^3/s, whereas, the third main unit adjacent to the additional small unit shall run with a discharge less than the rated discharge, which is 20.65 m^3/s so that the total discharge through all four units is 68 m^3/s.

6.2.3.3 Surge Tank in the Upstream Water Conveying System

Criteria for Surge Tank Requirement

The design of pressure shafts must consider the related issues of water hammer and speed control. In general, the pipeline / turbine systems meeting the following conditions do not require additional protective devices:

$\Sigma Li.Vi/Hn > 3$ to 5 (S.I.Unit) [4] …Eq. (5)

where,

$\Sigma Li.Vi$ = product of individual length of HRT and pressure shaft with their respective velocities; and

Hn = minimum net head;

Hn = 162.87 m

$\Sigma Li.Vi/Hn = (8120.86 \times 2.68 + 199.29 \times 4.8)/(162.87)$

= 132.49 > 3 to 5

Therefore, the preliminary investigation indicates that a surge shaft is primarily required as a surge control component in the project system. As the transient propagates throughout the entire pressurized system, a surge tank prevents the transient from propagating into the low pressure tunnels.

Hydraulic Design Criteria of Surge Tank

The hydraulic design criteria for the Surge Tanks are:

- the type of surge tank for the project shall be restricted orifice Surge Tank.

- as per the Thoma criterion for the present case, Thoma area of the Surge Tank is given by,

$$A_{th} = \frac{A_t L}{2g\beta H_o}$$ …Eq. (6)

where,

Ath = Thoma area of surge tank (m^2);

At = equivalent area of head race tunnel (m^2);

L = length of head race tunnel (m);

ß = coefficient of hydraulic loss for Head Race Tunnel up to Surge Shaft,

H_0 = net head on turbine (m), and

g = acceleration due to gravity (m/s^2).

Hydraulic Design of Surge Tank

Design area of Surge Tank

The minimum cross sectional area required for the Surge Shaft, from the hydraulic stability point of view (Ath), was estimated by applying Eq.(1) and was calculated as 77.5 m^2. After applying a factor of safety equal to 2.0 [5], the required area was 155.0 m^2. The area gives a diameter of 14.05 m. The adopted diameter was 15 m. The surges were computed based on design area of 176.71 m^2 (ref. Annex B to Part A3 Chapter 2 of this report).

Orifice dimensions and gate openings

The criteria for adopting the orifice diameter has been set as per IS: 7396 Part (1). An initial estimate of surge height corresponding to change in discharge was computed by the applying the following equation:

$$Z_* = V_1 \sqrt{\frac{L\, A_t}{g A_s}}$$ …Eq. (7)

where,

V_1 = velocity in Head Race Tunnel upstream of surge tank

L = length of head race tunnel upstream of Surge Tank

A_t = cross-sectional area of head race tunnel

A_s = cross-sectional area of Surge Shaft

The area of orifice was designed based on the orifice losses limiting condition by Calame and Gaden [5] as given below:

$$\frac{Z_*}{\sqrt{2}} + 0.25\mathrm{h_f} \leq \mathrm{h_{or}} \leq \frac{Z_*}{\sqrt{2}} + 0.75\ \mathrm{h_f}$$ …Eq. (8)

The minimum loss was adopted for the designing of the orifice area of the surge shaft and the corresponding diameter was calculated as 2.5 m and this was adopted for the design. At this stage, the size of gate slots has not been finalized.

Maximum upsurge

For a given tank size, the optimum size of the orifice is based on the balanced head design so that the maximum tunnel pressure below the surge tank equals the maximum upsurge level. For maximum upsurge, a full load rejection was assumed. The parameters considered for maximum upsurge were the maximum normal headwater, minimum head losses between reservoir and Surge Tank, and discharge at all units running.

As per IS: 7396 (Part1) -1985, Section-5.5.3.3, the maximum upsurge above the reservoir level was derived from the equation below:

$$\frac{L}{2g\phi(\beta+\eta)^2V_0^2} - \frac{Z_m}{(\beta+\eta)V_0^2} + \left[1 - \frac{\beta}{(\beta+\eta)} - \frac{L}{2g\phi(\beta+\eta)^2V_0^2}\right] x \left[e - \frac{2g\phi}{L}(\beta+\eta)Z_m + \beta V_0^2\right] = 0$$ …Eq. (9)

where,

$V_0 = V_1$ = velocity in the Head Race Tunnel upstream of the Surge Tank (m/s);

L = length of the Head Race Tunnel upstream of the Surge Tank (m);

β = coefficient of hydraulic losses;

$\beta V_0^2 = h_f$ = head loss in the tunnel up to the Surge Tank (m);

η = coefficient of resistance for orifice such that hor = η (V1-V2)2;

Φ = ratio of area of the Surge Tank to that of conduit As/At; and

Z_m = maximum surge level above maximum reservoir level.

Maximum down surge

For a maximum down surge estimation, the minimum normal headwater, maximum head loss, and load acceptance from 50 percent to 100 percent in the shortest reasonable time was take as the criteria. In the present study, the balanced design approach by [5] was applied, which implies that the surge ratio was equal to the throttling ratio for efficient design and it can be expressed as,

$$\frac{Z_{dn}}{h_{f1}} = \frac{h_{or}}{h_{f1}}$$ …Eq. (10)

where,

Z_{dn} = lowest down surge depth above the orifice top;

h_{f1} = head losses in head race tunnel up to the Surge Tank;

h_{or} = throttling loss for flow into and out via orifice in the Surge tank;

Z_{dn}/h_{f1} = surge ratio, and

h_{or}/h_{f1} = throttling ratio.

The surge ratio and throttle ratio relationship, established by Parmakian, 1955 was used for determining the maximum down surge. Annex B to Part A3 Chapter 2 of this report provides the published plot “Maximum Surge in Tank Due to Instantaneous Starting of Flow”.

The factor b_0 was computed using the relation,

$$b_0 = \frac{h_{f1}}{Q_1}\sqrt{\frac{A_S\, g}{L/A_t}} \qquad \text{...Eq. (11)}$$

The estimated value of b0 defines the factor to be applied at balanced condition to determine the surge ratio. The maximum head loss has been accounted to derive the maximum down surge.

6.2.3.4 Transient Analysis

Although the maximum upsurge and down surge scenario can be analyzed from the equations used in the previous sections, it is in the interest of the study to examine the transient behavior of the system in various time lags and at different load conditions. These conditions can be modelled developing different cases. In the present study, the software, WHAMO by the US Army Corps of Engineers (USACE), was used to model the various cases, which are extensively explained in the subsequent sub-sections. The upstream and the downstream water conveying systems were modelled separately using WHAMO.

Software Description: WHAMO

The design of the pressure shafts must consider the related issues of water hammer and speed control. In general, the pipeline / turbine systems the meet the following conditions do not require additional protective devices:

The acronym, WHAMO indicates “Water Hammer and Mass Oscillations”. The simplified, one-dimensional continuity and momentum equations, utilized in WHAMO model development, are shown below:

Momentum: $$\frac{1}{gA}\frac{\partial Q}{\partial t} + \frac{\partial H}{\partial x} + \frac{f}{D}\frac{Q|Q|}{2gA^2} = 0 \qquad \text{...Eq. (12)}$$

Continuity: $$\frac{\partial H}{\partial t} + \frac{c^2}{gA} + \frac{\partial Q}{\partial x} = 0 \qquad \text{...Eq. (13)}$$

where,

H = total head or energy grade;

Q = discharge;

x = distance along the conduit;

t = time;

g = gravitational constant;

A = cross sectional area of the conduit;

D = diameter of the conduit;

f = Darcy-Weisbach friction factor; and

c = celerity of a compression wave travelling through the conduit.

The WHAMO program is formulated in terms of a four-point implicit finite difference representation of the aforesaid governing differential equations. The programme delivers additional extensive utilities in terms of machine parameters and pumping systems. However, the biggest limitation of this programme is, it works only in FPS units, and hence, conversion of units for input parameters was essential and the model results needed to be converted to SI units.

Load Conditions for Transient Analysis

The following cases were induced:

- Case-1: Load rejection;
- Case-2: Load acceptance
- Case-3: Load acceptance followed by load rejection
- Case-4: Load rejection followed by load acceptance

Load Conditions: Upstream Water Conveyance System (Model-1)

These cases are described in detail below:

Case-1: Load Rejection

Emergency closure of wicket gate results in extreme high pressures upstream of the turbine. This operation will result in the upsurge in the surge shaft and pressure rise in penstock for the design of penstock.

Case-2: Load Acceptance

Sudden opening of the wicket gate results in extreme low pressures upstream of the turbine. This operation of the wicket gates results in the lowest down surge in surge shaft and minimum pressure at turbine end so as to align the penstock below minimum hydraulic gradient to avoid the development of negative pressures in the penstock.

Case-3: Load Acceptance followed by Load Rejection

Maximum upsurge in the Surge Tank may also occur with the acceptance of full load followed by emergency closing of wicket gate.

After load acceptance of all the units, rejection of load at the instant of maximum positive velocity in the Headrace Tunnel results in the maximum upsurge in the Surge Tank. Pressure rise in the penstock was also checked in this case. Upsurge in the Surge Tank and pressure rise in the penstock was adopted as the maximum of case-1 and case-3.

Case-4: Load Rejection followed by Load Acceptance

After emergency closure, if the load is accepted at the instant of maximum negative velocity in the headrace tunnel, then the lowest down surge in the Surge Tank may deepen to much lower levels as compared to the load acceptance case.

Load conditions: Downstream water conveyance system (Model-2)

In the downstream water conveyance system, the maximum upsurge occurs during load acceptance case, whereas, the lowest down surge occurs during load rejection case. All the four cases mentioned above were modelled keeping the same logic behind the loading criteria.

Boundary Conditions of Upstream Water Conveyance System (Model-1)

Upstream Boundary Conditions: Headpond Levels

Since the load rejection will cause upsurge in the Surge Tank, the water level at the upstream reservoir was adopted as the highest regulated water level (HRWL), whereas, in case of load acceptance, to meet the minimum submergence criteria of HRT, it was found that considering a level at HRWL would be ideal. Hence, the same level was considered as the head water level at the upstream reservoir for the lowest down surge simulations in the Surge Shaft. The table below shows the reservoir levels adopted while analyzing load rejection and load acceptance and their combinations i.e. Case-3 and Case-4.

Table 6-7: Reservoir levels for different operating conditions

Case	Operating Condition	Headpond Level (m)
Case-1	Load rejection	1158.00
Case-2	Load acceptance	1155.00
Case-3	Load acceptance followed by load rejection	1158.00
Case-4	Load rejection followed by load acceptance	1155.00

Downstream Boundary Conditions: Time Varying Discharge Operating Rules

The considered rated/allowable discharge of all the units in the transient model study are listed in the table below:

Table 6-8: Rated/maximum discharge through all units

Unit No	Discharge (m^3/s)	Remarks
Unit 1	22	Rated discharge
Unit 2	22	Rated discharge
Unit 3	20.65	Allowable discharge < Rated discharge of $22 m^3/s$
Unit 4	3.35	Rated discharge

Realistically, the transient model would have to be set up for the maximum release of 68 m^3/s being turbined by the three main units at about 3% overload. For the model it was, however, anticipated that Unit 4 will be in operation together with the 3 main units, and that Unit 3 and Unit 4 will be operated such that the design discharge of the system is met. More specifically, for the transient modelling, the maximum release of 68 m^3/s will be achieved with an assumed split into unit discharges as shown in the above table. The above table was thus referenced to set up the models of various cases with time varying discharge operating rules.

In the event of load rejection all the units are closed simultaneously; the closing time was adopted as 19 s from 100% opening to 0% opening of the wicket gates. While accepting the load, all the units except one main unit were loaded simultaneously and the opening time of 30 s was considered for 0% to 100% wicket gate opening. One main unit was delayed by 13 s, which means that 13 s after the start of the simulation, the wicket gates of this unit begin to open from 0% to 100% opening within next 30 s. This delay in time was optimally achieved to make the upstream and the downstream water conveyance system synchronous and to make the system free from negative pressure. The same is further discussed in the following sub-chapters.

The following table shows the operating rules adopted for the four cases:

Table 6-9: Operating rules (upstream boundary condition) in Model-1

Operating Conditions		Unit-1		Unit-2		Unit-3		Unit-4	
		Time [s]	Q [m^3/s]	Time [s]	Q [m^3/s]	Time [s]	Q [m^3/s]	Time [s]	Q [m^3/s]
Case-1	Load rejection	0	22	0	22	0	20.65	0	3.35
		19	0	19	0	19	0	19	0
Case-2	Load acceptance	0	0	0	0	0	0	0	0
		13	0	30	22	30	20.65	30	3.35
		43	22	-	-	-	-	-	-
Case-3	Load acceptance followed by load rejection	0	0	0	0	0	0	0	0
		13	0	30	22	30	22	30	3.35
		43	22	237	22	237	22	237	3.35
		237	22	256	0	256	0	256	0
		256	0	1500	0	1500	0	1500	0
		1500	0	-	-	-	-	-	-

Operating Conditions		Unit-1		Unit-2		Unit-3		Unit-4	
		Time [s]	Q [m³/s]	Time [s]	Q [m³/s]	Time [s]	Q [m³/s]	Time [s]	Q [m³/s]
Case-4	Load rejection followed by load acceptance	0	22	0	22	0	20.65	0	3.35
		19	0	19	0	19	0	30	0.00
		249	0	236	0	236	0	237	0.00
		279	22	266	22	266	20.65	256	3.35
		1500	22	1500	22	1500	20.65	1500	3.35

Boundary conditions of downstream water conveyance system (Model-2)

Upstream Boundary Conditions: Time Varying Discharge Operating Rules

In case of load rejection all the units are closed simultaneously. The closing time was adopted as 19 sec from 100% opening to 0% opening of wicket gates, which means that the discharge though a single machine reduces to 0 m^3/s in 19 sec.

While accepting the load, all the units except one main unit were loaded simultaneously and the opening time of 30 sec was considered for 0% to 100% wicket gates opening, which means that the discharge though single machine increases from 0 m^3/s to rated or allowable discharge in 30 sec. One main unit is delayed by 13 sec, which means that at 13th sec from the start of the simulation, the wicket gates of this unit begins to open from 0% to 100% opening in next 30 sec.

The operating rules are the same as listed in **Table 2-10**.

Downstream boundary conditions: Tail water levels (TWL)

The normal tail water level is 984 m, whereas, the maximum tail water level (Max.TWL) was calculate as 986.3 m. The source and the details of the tail water levels are explained in detail in Chapter 5.

Table 6-10: Operating rules (downstream boundary condition)

Case	Operating Condition	Headpond Level (m)
Case-1	Load rejection	984.00
Case-2	Load acceptance	986.30
Case-3	Load acceptance followed by load rejection	984.00
Case-4	Load rejection followed by load acceptance	986.30

6.2.3.5 Friction Parameters

The Darcy – Weisbach friction factor (Ref. USBR) with varying diameter of the components of water conveyance system was derived from Manning's n as given in the equation below:

$$f = \left(\frac{185 n^2}{D^{1/3}} \right) \qquad \text{...Eq. (9)}$$

where,

f = Darcy – Weisbach friction factor,

n = Manning's roughness, and

D = diameter of tunnel/pressure shaft.

<u>Friction Parameters: Upstream Water Conveyance System (Model-1)</u>

Minimum roughness was considered in Case-1 and Case-3 and maximum roughness was considered in Case-2 and Case-4.

The Manning's n values in concrete lined tunnels and penstock for above cases are tabulated below:

Table 6-11: Manning's roughness for different operating conditions (HRT and Pressure Shaft)

Case	Operating Condition	Manning's n	
		HRT	Pressure Shaft
Case-1	Load rejection	0.012	0.010
Case-2	Load acceptance	0.014	0.012
Case-3	Load acceptance followed by load rejection	0.012	0.010
Case-4	Load rejection followed by load acceptance	0.014	0.012

<u>Friction Parameters: Downstream Water Conveyance System (Model-2)</u>

Maximum roughness was considered in Case-1 and Case-3 and minimum roughness was considered in Case-2 and Case-4.

The Manning's n values in concrete lined tailrace tunnel and draft tube as steel conduit for above cases are tabulated below:

Table 6-12: Manning's roughness for different operating conditions (HRT and Pressure Shaft)

Case	Operating Condition	Manning's n	
		TRT	Steel Conduits
Case-1	Load rejection	0.014	0.012
Case-2	Load acceptance	0.012	0.010
Case-3	Load acceptance followed by load rejection	0.014	0.012
Case-4	Load rejection followed by load acceptance	0.012	0.010

<u>Celerity</u>

Wave celerity is the speed with which a disturbance moves through a fluid. The celerity values for the Head Race Tunnel and penstock used in the model are tabulated below:

Table 6-13: Celerity for various hydraulic components

Hydraulic Components	Celerity (m/s)
Head race tunnel (dia 5.6m, modified horse shoe)	969.77
Pressure shaft (dia 4.2m, circular)	1354.94
Tail ace Tunnel (dia 5.6m, circular)	969.77

6.2.4 Results and Discussions

6.2.4.1 Model-1: Upstream Water Conveyance System Results

From the analysis conforming to IS 7396-1 code [5], the maximum upsurge level was computed as EL.1180.2 m. The lowest down surge level of EL.1132.35m was determined for full load acceptance case from the surge ratio, throttling ratio and balanced design curves [5]. Albeit the results are conservative, the targets in the aforesaid methods are only the extreme surges. The above methodologies fail to exhibit the damping away of surges over time due to opening and closure of wicket gates with various operating rules and conditions.

The results from the transient modelling using WHAMO are not as conservative as the aforesaid results. However, the advantage of WHAMO is that it considers the actual geometry from the intake to the exit terminal, the elevations and losses in the water conveyance system, and it simulates the hydraulic system with a mass balanced numerical approach, and therefore, the results are precisely reliable. The maximum upsurge level from the transient model study has been computed to be EL.1179.30 m, whereas, the lowest down surge level has been computed to be EL.1130.50 m.

From the transient analysis, the surge levels were computed for various loads and operating rules. The opening time of 30 sec and greater than 30 sec are sensitive to the model and it leads to a scenario of possible drain out from the surge shaft through orifice. Therefore, the throttle diameter was optimized to be 2.5m with opening time of wicket gate as 30sec. From the model study, it was observed that the hydraulic stability of the water conveyance system, with the given operating conditions, is maintained with the following configurations:

- invert level of HRT at the start is EL 1141.98m;
- invert level of HRT below surge shaft is EL 1112.4 m; and
- the center line level of the pressure shaft just on the downstream of surge shaft is 1114.5 m

The following table illustrates the results obtained from transient analysis of the Model-1 (upstream water conveyance system):

Table 6-14: Extreme surges for various operating load conditions

Operating Load Condition		Max Upsurge Level in Surge Tank (m)	Lowest Down surge Level in Surge Tank (m)	Max Head at Turbine (m)
Full rejection (FLR)		1176.7	1148.5	1176.7
Full acceptance followed by full rejection (FAFR)	f_{max}	1177.3	1136.6	1178.4
	f_{min}	**1179.3**	1137.0	1182.8
Full acceptance (FA)		1155.0	1133.6	1155.0
Full rejection followed by full acceptance (FRFA)	f_{max}	1172.6	1130.8	1172.6
	f_{min}	1173.8	**1130.5**	1173.7

In the above table, the "fmax" and "fmin" are indicative of maximum and minimum friction. The extreme surge levels in the surge are displayed in the figure below.

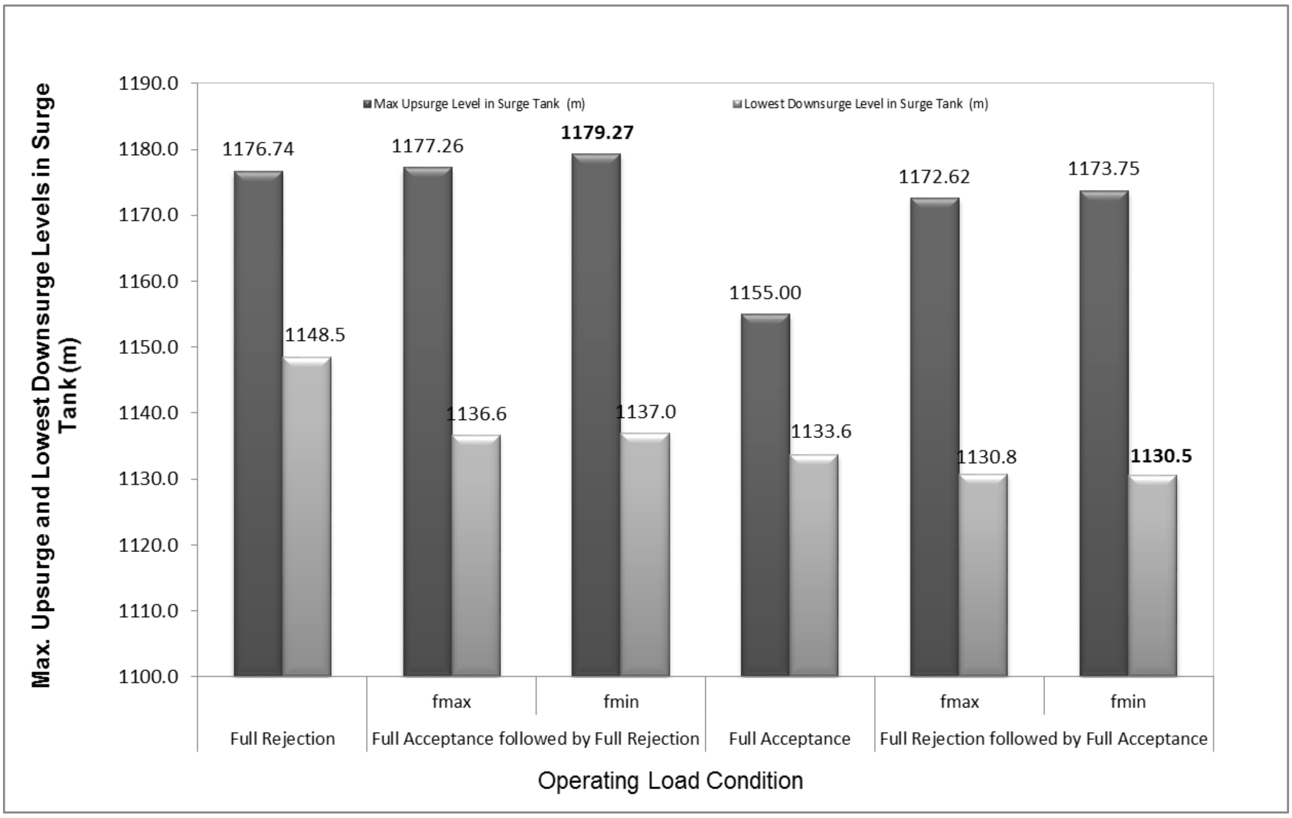

Figure 6-16: Extreme Surges in Surge Tank

The extreme energy levels and the extreme pressures in the upstream water conveyance system for various operating load conditions are tabulated below.

Table 6-15: Extreme energy levels and pressures in the upstream water conveyance system (Model-1)

Operating Load Condition		Maximum Energy Level (m)	Minimum Energy Level (m)	Maximum Pressure (m)	Minimum Pressure (m)
Full rejection (FLR)		1176.7	1148.5	202.74	7.80
Full acceptance followed by full rejection (FAFR)	f_{max}	1178.4	1127.9	204.39	7.56
	f_{min}	1182.8	1128.1	208.81	7.50
Full acceptance (FA)		1155.0	1124.7	181.01	4.57
Full rejection followed by full acceptance (FRFA)	f_{max}	1172.6	1112.8	198.60	1.34
	f_{min}	1173.7	1111.9	199.73	0.21

The plots featuring the above table are displayed below.

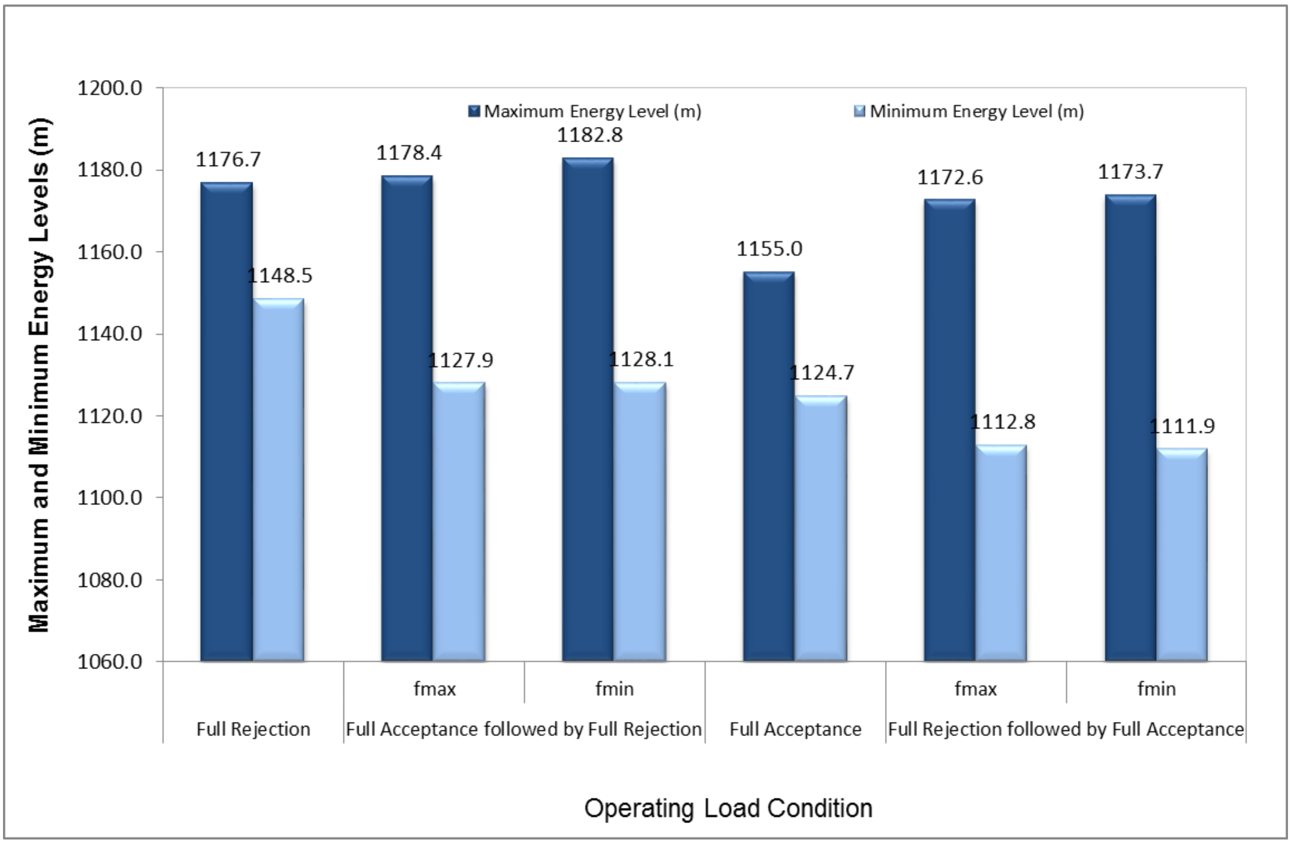

Figure 6-17: Model-1: Extreme energy levels in the upstream water conveyance system

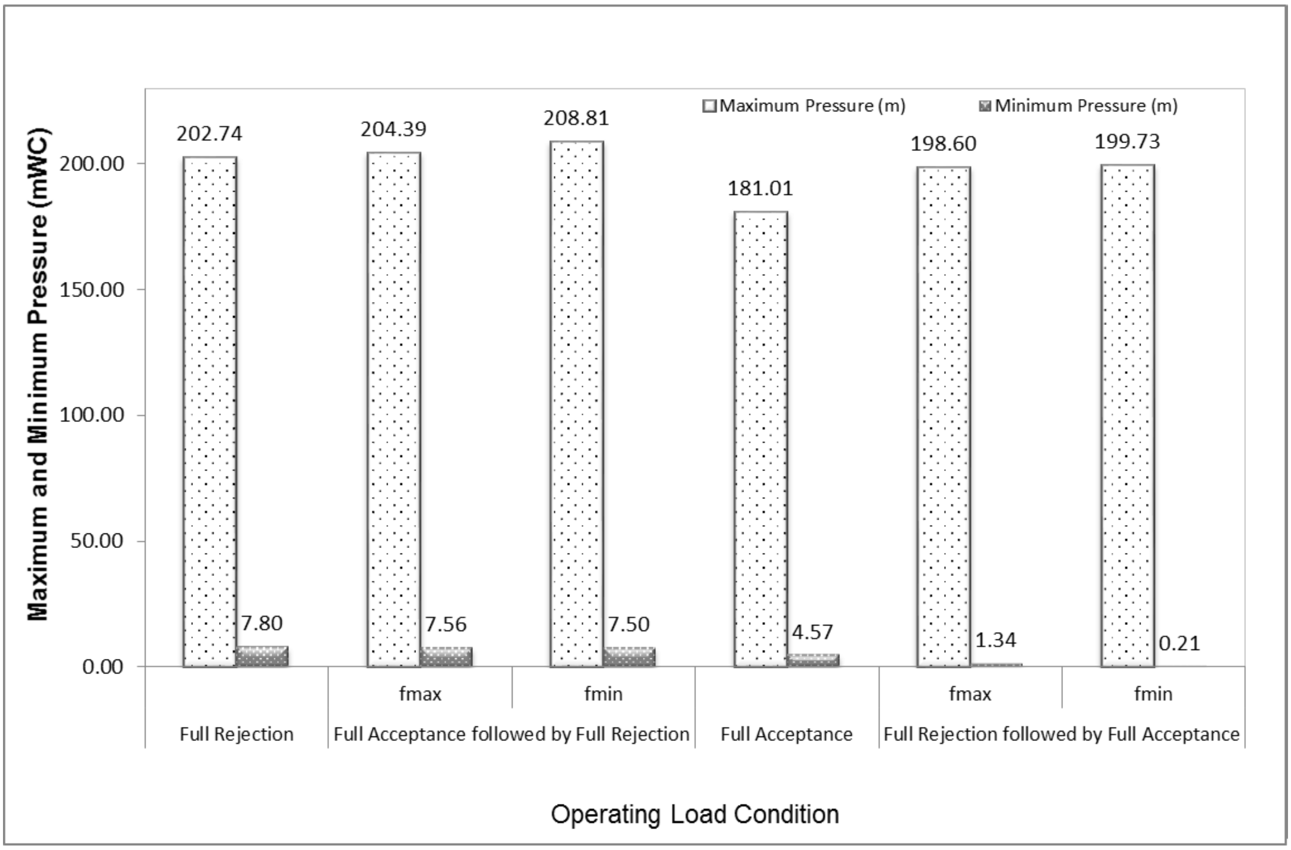

Figure 6-18: Model-1: Extreme transient pressures in the upstream water conveyance system

From the above results, it is concluded that the upstream water conveyance system is free from negative pressures. The static head over the center line of the main turbine is difference between the highest regulating water level (HRWL = 1158 m) and the center line level (C/L = 974 m) of the main turbine. The static head was calculated as 184.0 m, whereas, the maximum transient pressure over the turbine center line was 208.81, which results with a pressure rise of 13.48 %.

The plots of propagation of head and flow at the Surge Tank and at the units and of the maximum and minimum energy levels along the upstream water conveyance system for all considered cases are presented in the Annex to Part A3 Chapter 2 of this report.

6.2.4.2 Model-2: Downstream Water Conveyance System Results

The extreme energy levels and the extreme pressures in the upstream water conveyance system for various operating load conditions have been tabulated below.

Table 6-16: Extreme energy levels and pressures in the upstream water conveyance system (Model-2)

Operating Load Condition		Maximum Energy Level (m)	Minimum Energy Level (m)	Maximum Pressure (m)	Minimum Pressure (m)
Full rejection (FLR)		986.52	972.26	16.80	-3.02
Full acceptance followed by full rejection (FAFR)	f_{max}	992.96	971.43	23.14	-4.60
	f_{min}	992.96	971.37	23.14	-4.88
Full acceptance (FA)		992.87	986.01	22.84	6.49
Full rejection followed by full acceptance (FRFA)	f_{max}	992.90	973.78	22.84	-1.83
	f_{min}	992.84	973.72	22.77	-1.89

The plot featuring the above table has been displayed below.

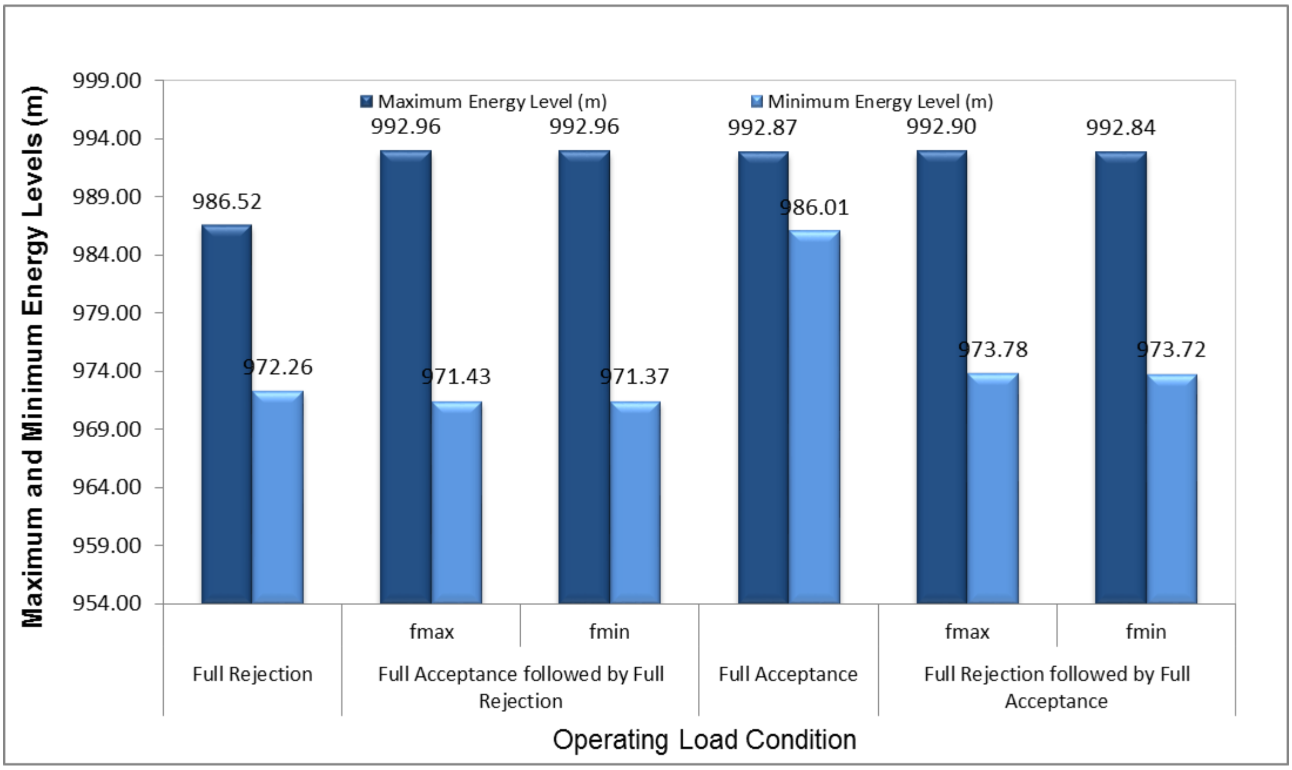

Figure 6-19: Model-2: Extreme energy levels in the downstream water conveyance system

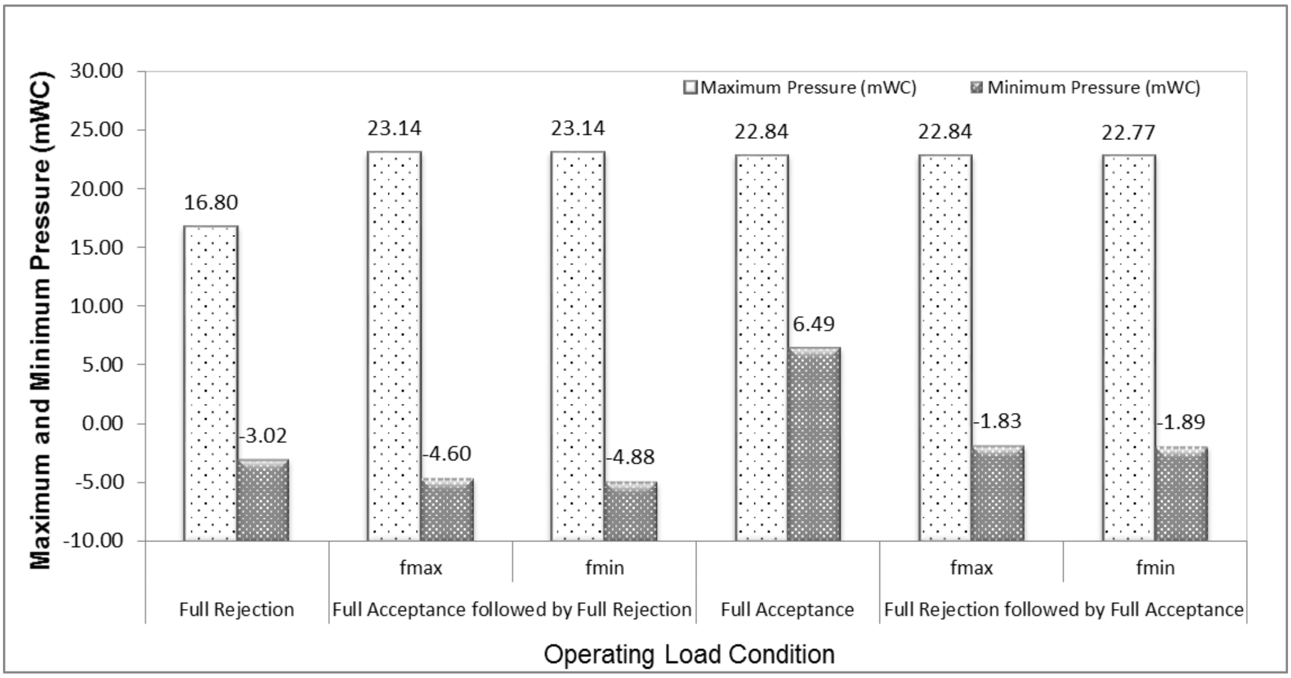

Figure 6-20: Model-2: Extreme transient pressures in the downstream water conveyance system

The negative pressures, as indicated in the above figure, were developed in the drop section (steel conduit) of the draft tube. The Tailrace Tunnel was observed to be free from negative pressure. Several scenarios were simulated by iteratively changing the opening and closing times of the wicket gates and also by delaying on of the main unit/turbine by 13 sec so that negative pressures were mitigated in the system other than the one specified above.

The plots of propagation of head and flow at various nodes and at units and of the maximum and minimum energy levels along downstream water conveyance system for all considered cases are displayed in the Annex to Part A3 Chapter 2 of this report.

6.2.5 Conclusions

From the above study and analysis, the following conclusions were drawn:

- the hoist arrangements in the Surge Tank will be kept in accordance with the maximum upsurge level of 1179.3 m;
- for the purpose of the Surge Tank design in the upstream water conveyance system, the lowest down surge level will be 1130.5 m;
- the diameter of the throttle in the Surge Shaft shall be 2.5 m;
- the maximum pressure rise from the analysis works out to be 13.48 % at turbine;
- the maximum transient discharge towards the Headpond was observed during load acceptance followed by rejection and the magnitude of the discharge was 34.9 m^3/s;
- the Headrace Tunnel, the pressure shaft and the Tailrace Tunnel were free from negative pressure;
- the maximum negative pressure in the draft tube drop section was observed to be (-4.88 mWC); and
- the final modalities of operation rules must be based on the first experience of closing and opening of the wicket gates during operation of the units.

6.3 Outlet Structure

6.3.1 Introduction

The Outlet Structure is the successive structure after the Tailrace Tunnel, which is a part of the exist system. The Outlet Structure was designed such that the exit loss is minimized by inducing a Tailbay at an adverse gradient. The control section is the sill of the Tailbay, which acts as a weir. However, as the sill is in the proximity of the Tamakoshi River bank, the flow depth over the weir may or may not be influenced by the water level in the river. This means that the tailwater level in the system may or may not be independent of the river depending upon the combination of flow in the river and flow through the outlet. To design the Outlet Structure, it was essential to analyze the river tailwater levels under various possible flow conditions.

6.3.2 Objectives

The objectives of the study were:

- to model the river reach near the Outlet Structure and to establish the rating curve at the cross-section of the Tamakoshi River closest to the Outlet Structure;
- to determine the tailwater levels with reference to the energy generation simulations, which are indicative for possible combinations of flow from the Outlet Structure and the corresponding flow in the river; and

- to design the hydraulic tailrace arrangements.

6.3.3 River Modelling using HEC-RAS for the Outlet Structure Reach

The river reach at the Outlet Structure section was based on the survey data and was modelled using HEC-RAS software. A steady state model was developed for the reach. The geometry file of the river reach, cross section station and geometries, as well as the downstream reach lengths of the channel and overbanks for each cross section were utilized. The topography of the area was obtained from the survey drawings. A section of the Tamakoshi River was defined as the study area. The river geometry was defined by the alignment and cross sections of the river which was created with Civil 3-D software and exported as GIS data as *.geo file. The cross sections were extended to ensure that all water from the flood was confined in the cross sectional area.

6.3.3.1 Overbank geometry - topographic data

The overbank geometry was obtained from topographic maps provided by surveyors.

6.3.3.2 Final HEC-RAS cross sections

Final HEC-RAS cross sections were extracted using the field survey data and the topographic data. The schematic alignment of the river and the cross-sections are displayed in the figure below.

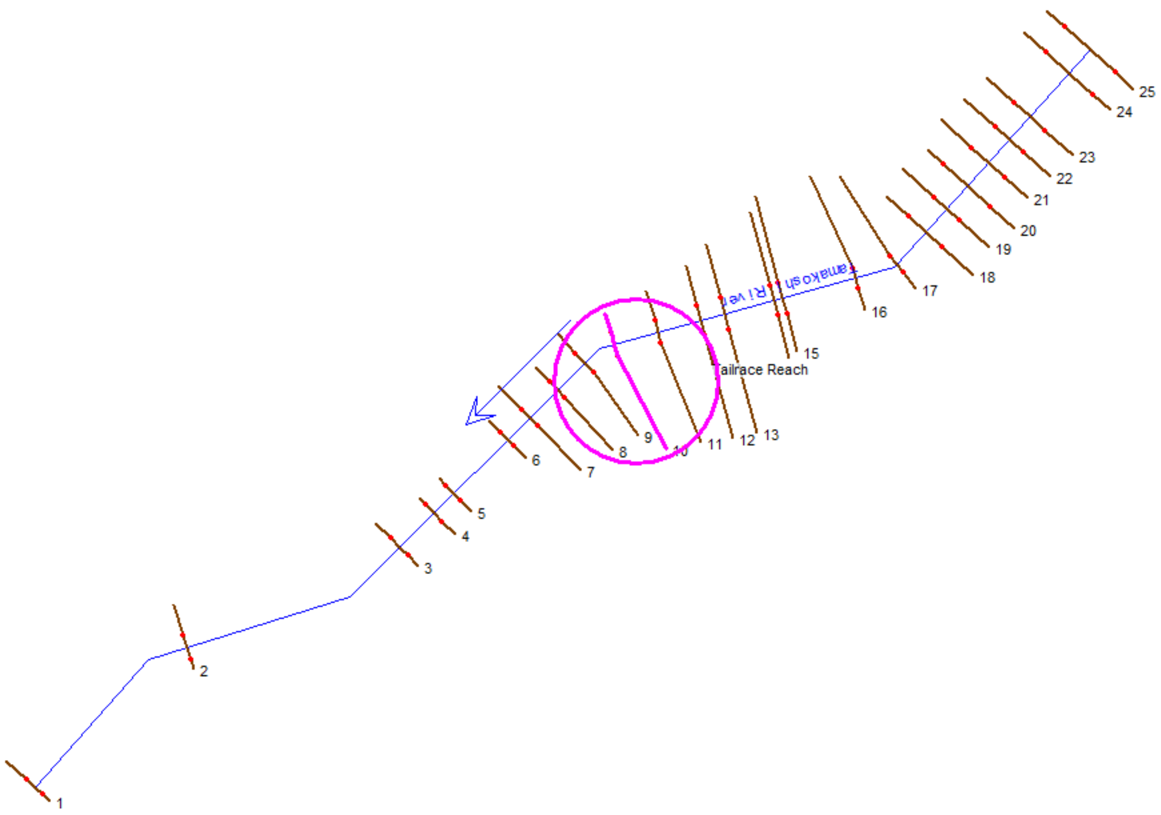

Figure 6-21: Tailrace Outlet Reach (XS-10)

In the above figure, the cross-section number 10 represents the section nearest to the Outlet Structure.

6.3.3.3 Flow Resistance or Roughness

The Manning's n-value was used to help calculate the energy losses between cross sections due to friction. The Manning's n-value depends on several factors which include: surface roughness; vegetation; channel irregularities; degree of meander; obstructions; size and shape of the channel. For the present study, the reach was assigned Manning's n values for the channel and overbank flow areas.

Manning's n values for the HEC-RAS model were determined with reference to the HEC-RAS User's Manual (USACE, 2008). The 'n' values of .045 for the overbanks and 0.06 for the channel as recommended by the User's Manual is consistent with the 'n' values adopted in previous studies [7].

6.3.3.4 Boundary conditions

The HEC-RAS models were executed under the assumption of subcritical flow. Normal depth was chosen as the downstream boundary condition. Normal depth computations were based on energy slope which was approximated by the channel slope of the downstream reach as determined from topographic data.

The river slope near the project area was determined as follows.

Table 6-17: River slope near the tailrace outlet of TK-V HEP

Location	Distance between upstream and downstream contours (m)	u/s contour (m)	d/s contour (m)	Slope	Slope (%)
Outlet Structure	1,710	993.00	970.79	0.01298	1.29

6.3.3.5 Steady Flow Data

Steady flow analysis was used to route five flows of 66, 300, 1,000, 1,700, and 2,500 m^3/s through the river. This range was chosen because it represents design flow, flow when Upper Tamakoshi HEP is shut down, and other flows close to Q_{2yr}, Q_{25yr} and Q_{1000yr} floods.

6.3.3.6 Results

<u>Tailwater Rating Curve</u>

The rating curve of cross-section number 10 represents the section near the Outlet Structure is displayed in the figure below. Water surface values computed with the help of the shown formula refer to a reference elevation of 979.66 masl.

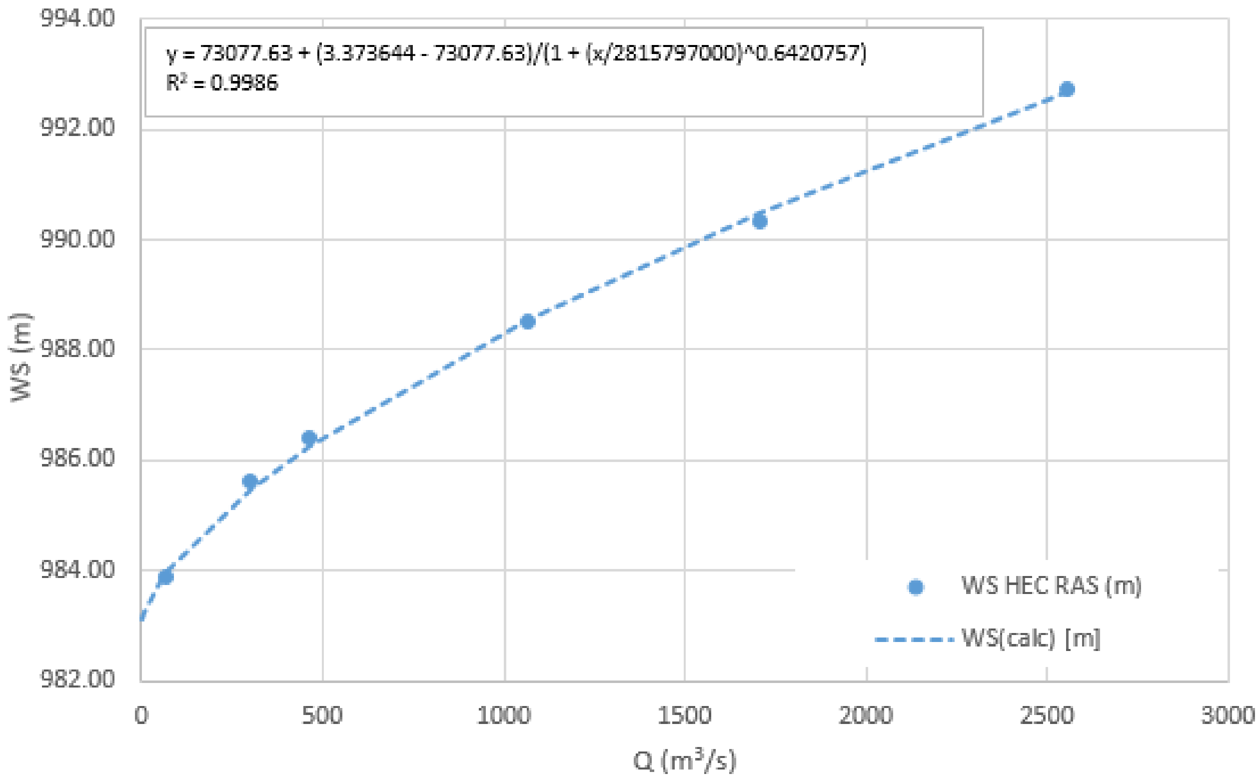

Figure 6-22: Rating curve at the river cross-section near the Outlet Structure

Safety of Outlet Structure Deck against Flooding

The possible flood level near the Outlet Structure was determined from the rating curve shown above. Since the safety level for the passage of a hydrological flood alone past the permanent structure, the flood with 10,000 years return period was selected. The design flood for this return period is available from Part F2 of the Detailed Design Report.

As per Part F2 of the Detailed Design Report, Table 13-2, the flood discharge considered for the location of the Power Station and Outlet Structure is $Q_{10,000}$ = 2,478 m³/s. According to that report the most unfavourable estimate is $Q_{10,000}$ = 3,470 m³/s when taking a 40% increase for climate change effects into account.

The distance from Tsho Rolpa to the Power Station area is approximately 42 km (see Part F5 of the Detailed Design Report). For this distance ICIMOD [3] states, for the passage of a GLOF alone, a peak GLOF discharge of about Q_{GLOF} = 3,000 m³/s was assurmed (see below figure).

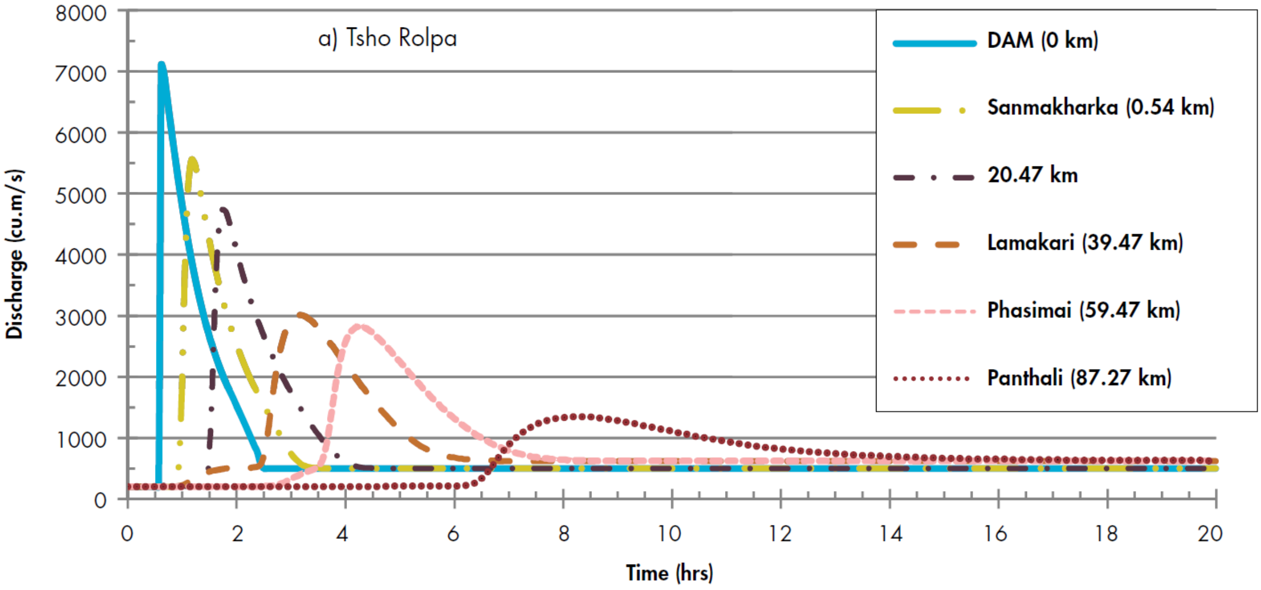

Figure 6-23: GLOF Hydrograph routed from Tsho Rolpa (according to [3])

Following the above data for stand-alone events, the relevant design flood for the Outlet Structure was the 10,000 yrs. flood which is greater than the GLOF peak discharge.

The corresponding water level for the $Q_{10,000}$ flood was derived from the rating curve:

$$H = 73077 + \frac{3.374 - 73077}{\left[1 + \left[\frac{Q}{2.816 \times 10^9}\right]^{0.642}\right]} + 979.66$$

= 994.77 m asl.

This level was significantly lower than the level for the combined flood event of a GLOF coinciding with the Q_{100} flood, which was estimated in Part F2 of the Detailed Design Report as Q = 6,000 m³/s. For the latter discharge the water level was computed from the same rating curve as 999.7 m asl. For this water level the Outlet Structure, with a deck level at 1014 m asl, is safe.

6.3.4 Hydraulic Design of Tailrace Arrangements

The tailrace arrangement was established and designed based on the river flows and corresponding water levels, the topography and the cross-sections of the river.

6.3.4.1 River Flow at the Outlet Structure

The flow condition in the river at the Outlet Structure is governed by the combined discharge from the Tamakoshi V power waterways and from the river directly upstream of the structure. The water level corresponds to the elevation given by the above rating curve, which has also been adopted for the energy generation simulations (peak and non-peak operation). A separate rating curve for very small plant releases, which would reflect the tailwater backup effect by the Tailbay end-sill, was not considered in the simulations since the influence was found negligible.

6.3.4.2 Tailwater Levels for Project Planning

Input Parameters

Table 6-18: Input Parameters for Tailwater Levels

Input Parameters	Magnitude	Unit
Design Discharge Qd =	66.00	m^3/s
3 Main Units Discharge Q_{3unit} =	66.00	m^3/s
1 Main Unit Discharge Q_{1unit} =	22.00	m^3/s
Q_{normal} (full load) =	66.00	m^3/s
Q (40% load when 1 unit is running) =	8.80	m^3/s
River Bed Level =	979.66	m
Height of Sill from river bed =	3.1	m
Proposed Sill Level =	982.8	m

Design Computations for Tailwater Level

The tailwater levels in the Tailbay were determined considering the above design discharges. Subsequently, these design discharges were added to the river flow to examine the tailwater levels in the downstream river bed. The tailwater levels from these two approached were compared and the highest of the two, for a given design condition, was adopted as the tailwater level.

Table 6-19: Design Computations for Tailwater Levels

Particulars		All Units	100% (1-Unit)	40% (1-Unit)
Discharge through Weir Qw (m^3/s) =	-	66.00	22.00	8.80
Coefficient of discharge Cd =	1.7	1.7	1.7	1.7
Weir Span, B (m) =	27.8	27.8	27.8	27.8
Upstream head of the Weir, H (m) =	-	1.2	0.6	0.3
Proposed Weir Crest Level (m) =	982.8	982.8	982.8	982.8
Water level above the crest (m) =	-	984.0	983.4	983.1
River flow including the discharge though weir (m^3/s) =	-	72.08	44.61	23.90
Water level in the river (m) =	-	984.0	983.8	983.5
Tailwater level (m) =	-	984.0	983.8	983.5

From the above computations, it was noted that, for the given discharge conditions from the unit/units, the water level in the river dominates the water level above the sill/crest that was computed excluding the river inflow. It was observed that, for all units running conditions, the tailwater levels from both approaches merge at 984.0 m, whereas, for smaller discharges the river tailwater dominates.

Upper Limit of Tailwater Level for Tamakoshi V HEP

The inflow at UTK HEP, when the plant shuts down, is 250 m³/s [2]. The catchment areas of the UTK HEP dam site and the Tamakoshi River section near the Tamakoshi V Outlet Structure are 1,587 km² [7] and 2,460 km² (source: satellite images and GIS). The corresponding flow near the Tamakoshi V Outlet Structure was derived from the catchment area ratio of UTK HEP to Tamakoshi V, which was determined as 1.6. The same factor was applied and the resultant flow at river section near Tamakoshi V Outlet Structure was computed as 465 m³/s. When, on the other hand, analyzing the peak river discharges for return periods in the range between 10 and 0.05 years, the discharge of 250 m³/s at UTK HEP dam was found to be associated with an annual exceedance probability of 3.1. For the location of the Tamakoshi V Outlet Structure, the same annual exceedance probability leads to a river discharge of 483 m³/s.

Given the above river flow estimates, the limiting river flow, up to which the Tamakoshi V plant can operate, was taken as the tailwater level corresponding to the river flow of 485 m³/s (conservatively selected as the greater of the two estimated discharges). The upper limit of the tailwater level corresponding to 482 m³/s is derived from the rating curve as 986.35 m.

6.3.4.3 Hydraulic Design of the Tailbay

The hydraulic design of Tailbay are tabulated below:

Table 6-20: Input Parameters for Tailwater Levels

Particulars	Magnitude	Unit
Design sill level/ crest level at the end of Tailbay	982.8	m
Design slope of Tailbay	0.44	(2.2H:1V)
Plan length of the Tailbay bed	19.70	m
Invert level at the Tailrace Tunnel end just before the Tailbay	974.0	m
Depth of the Tailbay from the crest level	8.8	m
Actual length of the Tailbay bed	21.6	m
Diameter of the Tailrace Tunnel	5.6	m
Design Velocity of flow through the Tailrace Tunnel	2.7	m
Required water seal at the exit of Tailrace Tunnel above the obvert	0.37	m
Design minimum water seal level at the exit of Tailrace Tunnel	982.8	m
Design water seal at the exit of Tailrace Tunnel above the obvert	3.2	m

6.3.4.4 Head Loss in the Outlet Structure

The details of the head losses are discussed in Subchapter 2.3 Water Conveying Tunnels and Surge Tank. The head loss in the Outlet Structure arrangement are summarized in the table below.

Table 6-21: Head Loss in Outlet Structure

S.N.	Components of Head Loss	Head Loss	Unit
A.	**Head Loss in Outlet Structure**		
	Head Losses in Transitions (from Circular to Rectangular)	0.0000	m
	Frictional Loss in Rectangular Conduit	0.003	m
	Gate Groove Losses	0.021	m
	Exit Losses	0.0014	m
	Head Loss in Transitions and Rectangular Conduit including the Exit Loss	**0.026**	m

6.3.4.5 Conclusion

The following points were concluded:

- the tailwater level with 1 main unit running with 40% load was calculated as 983.5 m;
- the sill or the crest of weir must be 27.8 m;
- the inclined length of the Tailbay bed must be 21.6 m;
- the invert level of the lowest bed of the Tailbay must be 974.0 m;
- the present design proposes the merging of Tailrace Tunnel invert at the exit and the lowest bed of the Tailbay; this arrangement may be altered by keeping a distance between the said two points, but at the same level of 974.0 m, if such change is found suitable without affecting the overall arrangements.

6.4 Power Station Area

6.4.1 Objectives

The objectives of the hydraulic analyses are to model the river reach close to the Power Station with its tunnel portals (Main Access Tunnel and Cable & Ventilation Tunnel) to establish the rating curve at the cross-section of the Tamakoshi River vis-à-vis these portal structures. The rating curve will thereby serve as basic input for the setting of the tunnel portals resp. design of flood protection structures.

The location of the river cross section of primary interest essentially coincides with the location of the Test Adit portal.

6.4.2 River Modelling using HEC-RAS for the Powerhouse Adit Reach

6.4.2.1 General

The river reach vis-a-vis the above referred tunnel portals has been modelled based on the survey data using HEC-RAS. The steady state model has been developed for the river reach. The geometry file of the river reach, cross section station and geometries, as well as the downstream reach lengths of the channel and overbanks for each cross section have been utilized. The topography of the area has been referred from the survey drawings. A section of Tamakoshi River has been defined as the study area. The river geometry was defined by the alignment and cross sections of the river, created with Civil 3D and exported as GIS data as *.geo file. The cross sections have been extended to ensure that all water from the flood is confined in the cross sectional area.

6.4.2.2 Overbank geometry - topographic data

For the overbank geometry, the topographic maps provided by surveyors have been used.

6.4.2.3 Final HEC-RAS cross sections

Final HEC-RAS cross sections have been extracted using the field survey data and the topographic data. The schematic alignment of the river and the cross-section have been displayed in the figure below.

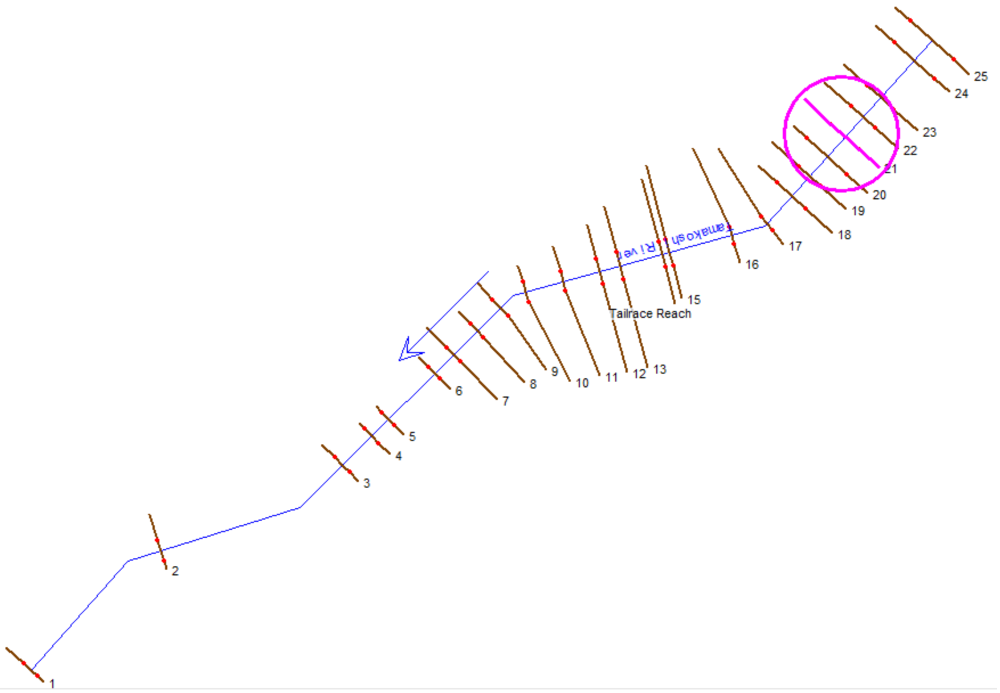

Figure 6-24: River Reach with Cross Section 21 (XS-21) vis-à-vis the Test Adit Portal

In the above figure, the cross-section number 21 represents the section nearest to the Test Adit portal.

6.4.2.4 Flow Resistance or Roughness

The Manning's n-value has been used to help calculate the energy losses between cross sections due to friction. The Manning's n-value depends on a number of factors which include: surface roughness; vegetation; channel irregularities; degree of meander; obstructions; size and shape of the channel. For the present study, the reach was assigned Manning's n values for the channel and overbank flow areas.

Manning's n values for the HEC-RAS model have been determined in reference to the HEC-RAS User's Manual (USACE, 2008). The 'n' values of .045 for the overbanks and 0.06 for the channel as recommended by the User's manual is consistent with the 'n' values adopted in previous studies [1].

6.4.2.5 Boundary conditions

The HEC-RAS models have been executed under the assumption of subcritical flow. Normal depth has been chosen as the downstream boundary condition. Normal depth computations are based on energy slope which was approximated by channel slope of the downstream reach as determined from topographic data.

The river slope in the vicinity of the project area was determined as follows.

Table 6-22: River slope in the vicinity of the tailrace outlet of TK-V HEP

Location	Distance between upstream and downstream contours	u/s contour (m)	d/s contour (m)	Slope	Slope (%)
Powerhouse Adit structure	1710	993.00	970.79	0.01298	1.29

6.4.2.6 Steady Flow Data

Steady flow analysis was used to route five flows of 66, 300, 1000, 1700, and 2500 m^3/s through the river. This range was chosen because it represents design flow, flow when Upper Tamakoshi HEP will be shut down, and other flows close to Q_2yr, Q_{25} yr and Q_{1000} yr floods.

6.4.2.7 Results

The rating curves of cross-section number 21 representing the sections near the outlet structure has been displayed in the figure below.

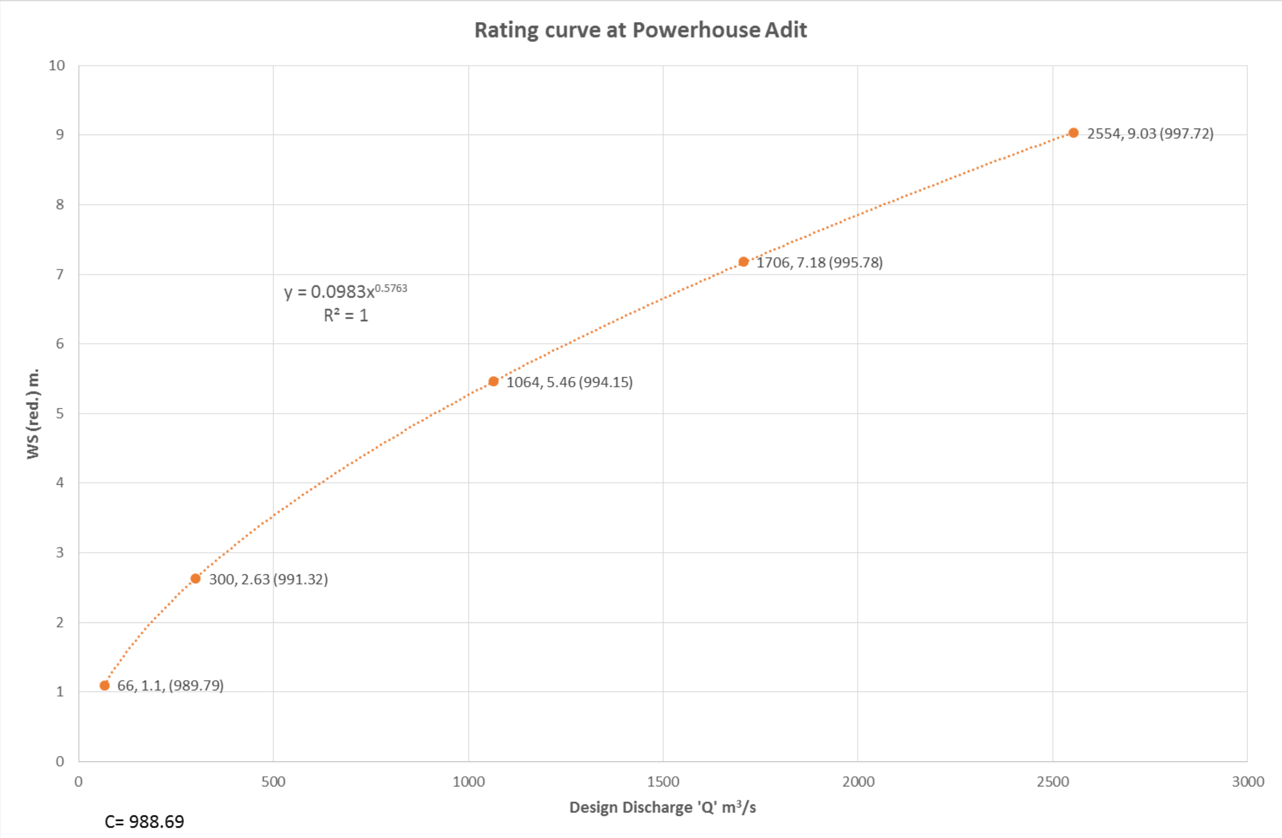

Figure 6-25: Rating Curve at River Cross-section at the Powerhouse Adit

6.4.3 Safety of Tunnel Portals in the Power Station Area against Flooding

The possible flood levels in the area around the portals of the Main Access Tunnel to the Powerhouse Cavern and the Cable & Ventilation Building were determined from the rating curve shown above. As safety level for the passage of a hydrological flood alone past the permanent structures the flood with 10,000 years return period was selected; the design flood for this return period is taken from Part F2 of the Detailed Design Report.

As per Part F2 of the Detailed Design Report, Table 13-2, the flood discharge considered for the location of the Power Station and Outlet Structure is $Q_{10,000}$ = 2,478 m³/s. According to that report the most unfavourable estimate is $Q_{10,000}$ = 3,470 m³/s when taking a 40% increase for climate change effects into account.

The distance from Tsho Rolpa to the Power Station area is approximately 42 km (see Part F5 of the Detailed Design Report). For this distance ICIMOD [3] states for the passage of a GLOF alone a peak GLOF discharge of about Q_{GLOF} = 3,000 m³/s (see Figure 6-23).

Following the above data for stand-alone events, the relevant design flood for the Power Station area is the 10,000 yrs. flood which is greater than the GLOF peak discharge.

The corresponding water level for the $Q_{10,000}$ flood is derived from the rating curve:

$$H = 0.0983\ Q^{0.5763} + 988.69 = 999.48 \text{ m asl}$$

This level is significantly lower than the level for the combined flood event of a GLOF coinciding with the Q_{100} flood, which is estimated as per Part F2 of the Detailed Design Report as Q = 6,000 m³/s. For the latter discharge the water level is computed from the same rating curve as 1003.5 m asl. For this water level the tunnel portals and the Terminal & Ventilation Building are safely protected by the parapet wall with crest elevation at 1005.0 m asl.

6.5 References

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Volume III – Annex

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[3] Glacial Lakes and Glacial Lake Outburst Floods in Nepal

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[5] Bureau of Indian Standards, IS 7936 Part-I,-II,-III, IS 4880-Part 3, and IS 11625,

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Hydraulic Design for Energy Generation, H. Wayne Coleman et al, Harza Company, Chicago, Illinois

[6] WHAMO, Software Manual

[7] Report on Power Station Equipment, 20-31-00053, September 26, 2017

7 CIVIL ENGINEERING DESIGN

7.1 Rock Mechanics & Foundation Design

7.1.1 Headworks - General

The Headworks consists of the project components upstream of the Headrace Tunnel. These include:

- Connecting Tunnel,
- Headpond
- Spillway Weir
- Spillway Tunnel,
- Spillway Terminal Structure.

The layout of the Headworks and the various components are shown in the figure below:

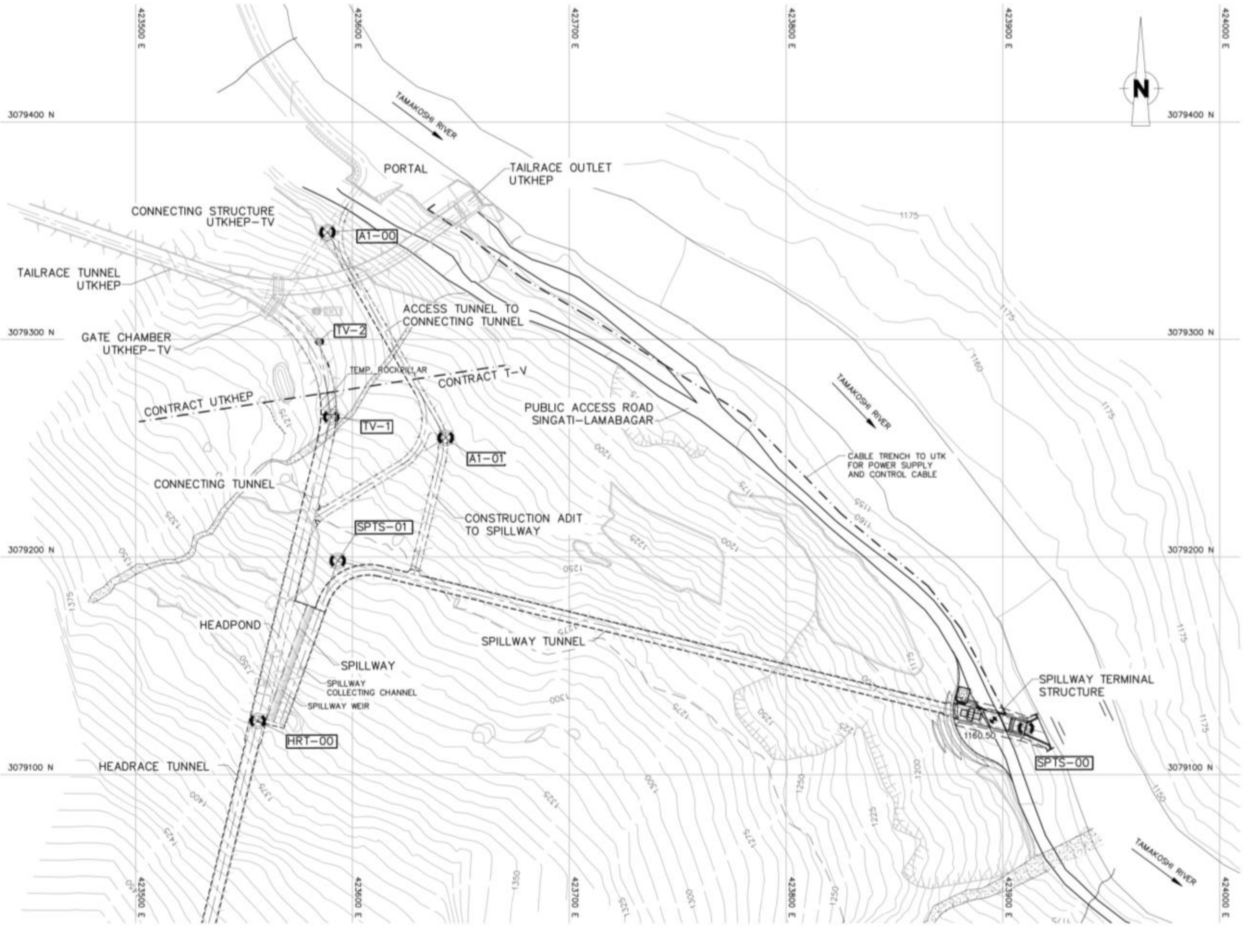

Figure 7-1: Sketch of the Plan of Headpond and Related Structures

In this section the support systems for the 1) Connecting Tunnel, 2) Headpond, 3) Spillway and 4) Spillway Terminal Structure are presented. Design of the concrete lining of the Head Pond Cavern and Spillway Tunnel are also presented in their respective sections. The concrete linings were designed to resist only hydrostatic loads. No rock loads were applied to the design of the concrete linings.

7.1.2 Headworks - Connecting Tunnel

Design of Support System for Connecting Tunnel

The design of the rock support system in the Connecting Tunnel to the Headpond requires a detailed analysis of the in situ rock stresses, rock strength parameters, application of rock bolts and shotcrete for rock support. The deformation modelling of the Connecting Tunnel, subjected to in situ stresses, were studied for the analysis and design of the rock support. A 2-D numerical analysis was carried out with the Phase 2 finite element programme for deformation and rock support modelling. The analyses and their results are discussed in this report. Initially, the rock support system was estimated based on empirical relations. Subsequently, a numerical analysis was performed. Details are presented below.

Geology of the Connecting Tunnel

The rock mass in this area is composed of banded gneiss. The rock mass contains quartz, feldspar, biotite, garnet, amphibole, and kyanite. Phenocrysts of quartz, feldspar and garnet are prominent.

The rock mass is coarse to very coarse grained, strong to very strong in strength, have closely to widely spaced foliation joints. The foliation joints (JS0) & some other joints are filled with sandy silty and sometimes clayey material up to 5mm in thickness. The rock mass is fresh to slightly weathered.

Four well developed main joint sets, including foliation joints (JS0, JS1, JS2 & JS3), were observed above the Headpond area. Among them, the foliation joints (JS0) are the prominent sets. The dip angles & dip directions of the said joints are as follows: JS0 (Foliation joints): 55-65/020-045, JS1: 50-60/080-085, JS2: 30-45/235 & JS3: 80-85/345-350. The rock mass is blocky in general. The said rock bodies are well interlocked.

The rock mass is dry at the outcrop. However, damp to dripping groundwater condition can be anticipated during excavation.

7.1.2.1 Specific Design Criteria

For the design of the support system for underground works e.g. tunnels, shafts and caverns, an initial support system was estimated based on empirical correlations and, thereafter, the model was analysed with a 2-D numerical analysis from Phase-2 software.

Two methods to determine preliminary length of rock bolts based on empirical correlation method and support design based on numeric analysis are explained below:

Empirical Method 1:

Design of rock support by using Barton et al. (1974) Empirical formula

Rock bolted rock mass forms a reinforced rock around an opening, which together with shotcrete and other kinds of supports are designed to resist the in situ stress conditions.

Barton et al. (1974) has suggested an empirical formula to estimate the bolt length as follows:

Lb = 2 + (0.15 B or H/ESR)

Where, Lb = Length of rock bolt in meters

B or H = Span or height of the tunnel in meters for designing bolt length for crown and wall respectively.

ESR = Excavation support ratio (indicated in the table below)

Table 7-1: Values of Excavation Support Ratio (ESR) - BARTON ET AL. 1974

S.N.	Type of Excavation	ESR
1	Temporary mine openings, etc.	3 – 5
2	Vertical shafts;	
	(a) Circular section	2.5
	(b) Rectangular / square section	2.0
3	Permanent mine openings, water tunnels for hydro power (excluding high pressure penstocks), pilot tunnels, drifts and headings for large excavations, etc.	1.6
4	Storage rooms, water treatment plants, minor road and railway tunnels, surge chambers, access tunnels, etc.	1.3
5	Oil storage caverns, power stations, major road and railway tunnels, civil defence chambers, portals, intersections, etc.	1.0
6	Underground nuclear power stations, railway stations, sports and public facilities, factories, etc.	0.8

For a particular structure, the applicable Excavation Support Ratio (ESR) and length of the rock bolts was determined by using the above empirical formula suggested by Barton el at. (1974).

The rock bolt lengths were additionally checked using the "rule of thumb" recommendation of the US Army Corps of engineers (USACE) as follows:

L = 0.5 B (for B<6.6 m),

L = 0.5B<L<0.25B (for 6.6 m<B<20 m)

L = 0.25B (for B>20m)

where b = width or height of the excavated space

L = minimum required length of the rock bolt

Empirical Method 2:

Design of rock support by using Grimstad and Barton (2013) chart

Grimstad and Barton have proposed a chart for the design of rock supports in underground excavations based on rock quality and equivalent dimension of excavation (see Figure below). In this figure equivalent dimension is defined as the ratio of excavation opening to Excavation Support Ratio (ESR).

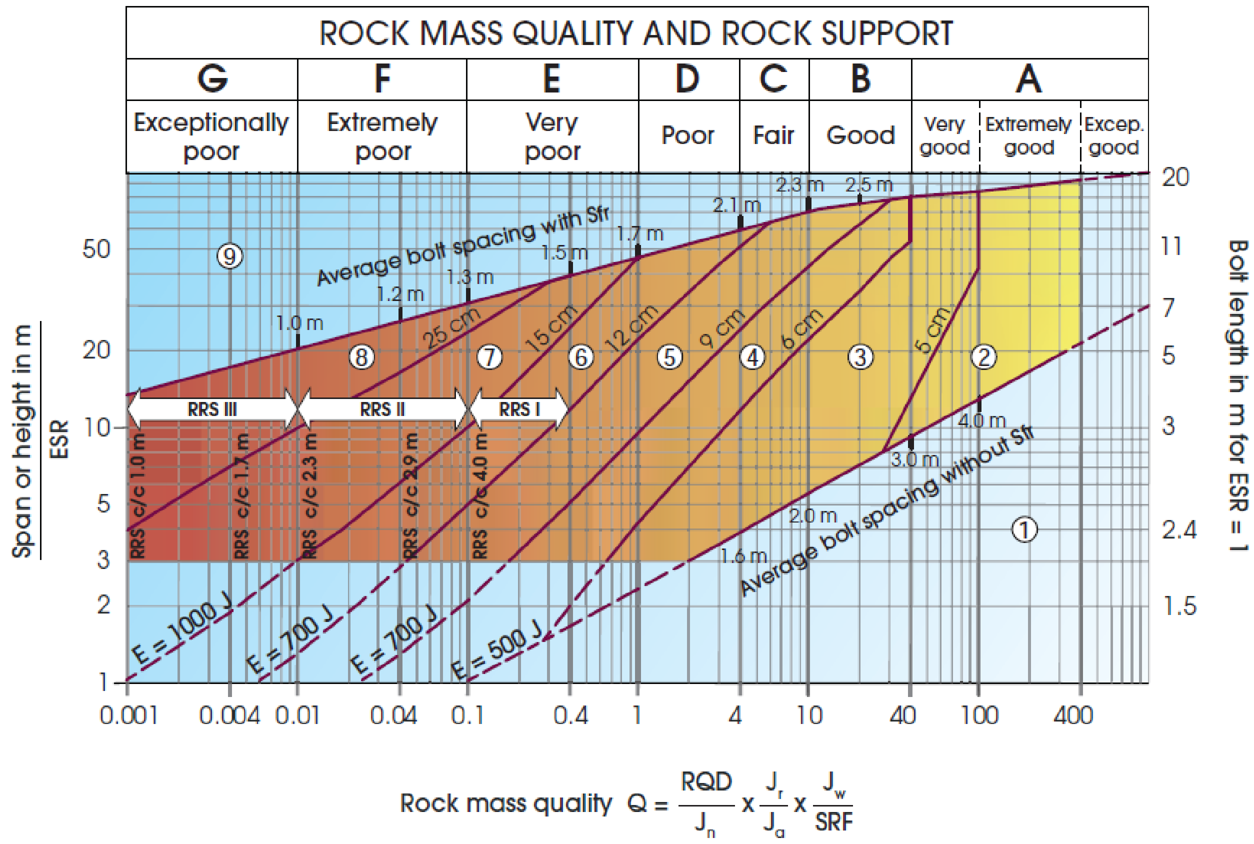

Figure 7-2: Tunnel Support Chart (Grimstad and Barton 2013)

Support Design Based on a Numerical Analysis

Input Parameters for the Analysis

A) Geo-mechanical Properties

To estimate the geo-mechanical parameters for different rock mass conditions, the rock mass parameters was derived using intact rock parameters with GSI, intact rock modulus and material index.

Intact rock parameters was obtained either from lab test reports, where applicable, or based on published results for similar types of rock. Based on the rock mass condition, GSI values were estimated based on a correlation with the RMR value. Based on these values, the parameters for Generalized Hoek-Brown failure criterion of the jointed rock mass will be obtained by using the RocLab software or the inbuilt parameter calculator of the Phase^2 software.

As per Bieniawski 1989, the following correlation exists between RMR and GSI:

GSI = RMR -5 for RMR >23

GSI = 9 x logQ + 44 for RMR <23

Based on above correlation, the GSI values considered for various classes of rocks are given in the table below:

Table 7-2: GSI Values Based on Class of Rock

S.N.	Class of Rock	RMR Value	Q Value	GSI Value Based on Correlation	Adopted GSI Value
1	Class I	80 – 100	40 – 100	75 - 95	75
2	Class II	60 – 80	10 – 40	55 – 75	55
3	Class III	40 – 60	4 – 10	5 – 55	45
4	Class IV	20 – 40	1 – 4	44 – 35	35
5	Class V	10 – 20	0.1 - 1	35 – 44	25
6	Class VI	0 - 10	0.01 – 0.1	26 - 35	20

B) In Situ Stress

In absence of actual test results, the in situ stresses were obtained based on overburden depth at the location of project structure. The ratio of horizontal stresses to vertical stresses will be obtained from empirical correlations based on International Research Journal of Applied and Basic Sciences @ 2013 by Vahid Hosseinitoudeshki. The equation of Sengupta (1998, in Singh and Goel, 1999) used is given as:

$$\sigma_H = 1.5 + 1.2x\ \sigma_v, \text{ where } \sigma_v = \gamma \text{ x } Z$$

The value of vertical stress (σ_v) was calculated considering the average density of the rock mass (i.e. 27 KN/m^3) and then, the values of maximum horizontal stress (σ_H) in each section of the tunnel and cavern was obtained from the above equation as described above., Since the σ_H is highest of all the stress tensors it will be taken as σ_1 and σ_v as σ_3. σ_z which is the out of plane stress in the plain strain analysis of Phase^2 software will be calculated as 75% of σ_1 or equal to σ_3, whichever is higher.

C) Support Properties

Different properties of the support elements in the model will be used as described below:

Rock Bolts: Grade Fe500 rock bolts with a Young's Modulus of 200,000 MPa were used. The maximum tensile capacity of each rock bolt was calculated as 0.87*fy*A, where A is the cross sectional area of the bolt. The residual capacity of the rock bolt was 10% of the tensile capacity.

Thus, the maximum tensile capacity of a 25 mm dia rock bolt is 213 kN and for a 32 mm diameter rock bolt the maximum tensile capacity is 349 kN. In the numerical modelling maximum tensile capacity of rock bolt was used and a minimum factor of safety will be required for most of the rock bolts in the excavated periphery.

In Powerhouse wall portion, a high capacity Fe1050 DCP (Double Corrosion Protection) monobars of 36 mm dia with maximum tensile capacity of 960 kN are also proposed

Shotcrete: Grade M35 (C30/37) shotcrete of different thicknesses was proposed. The Young's Modulus of elasticity is 5000√fck = 50000√35 = 29580 MPa, Poisson's Ratio is 0.20, the compressive stress was 35 MPa and the tensile stress was 0.7√fck = 0.7√35 = 4.14 MPa.

Steel Rib: For Rock Class Support V and VI, Indian Standard I Sections (ISMB / ISHB) was used as steel ribs.

<u>Model Setup</u>

The 2-D plain strain finite element model was prepared as follows:

1. Shape and size of the excavation boundary was modelled as per the AutoCAD drawings.
2. Stages of the excavation was modelled where applicable.
3. Size of the finite element mesh was modelled considering the likely thickness of liner element.

4. The disturbed zone boundary of the rock mass was based on size of the excavation dimensions and it will vary between 0.5m to 2m which will have the same parameters as the rock mass but will consider a disturbance factor of 0.3. The modulus of deformation of this disturbed zone was considered the same as that of the surrounding undisturbed rock mass.

5. The expansion factor for the external boundary was considered as 3 three times the corresponding dimension to the opening and only in exceptional cases, where the effect of the excavation exceeds this boundary, it was increased.

6. Support parameters was modelled as described above.

Design Methodology

The analysis was done with a 2-D plane strain model with the finite element method based software Rocscience Phase[2]. After the tunnel is excavated some rock stresses will be released through deformation of the rock prior to the support application. Consequently, only the remaining amount of stress induced loads will be carried by the support system. If the support system were designed to support the entire in situ stresses the supports system would be unnecessarily over designed. The maximum allowable amount of deformation allowed for stress relaxation is 0.5% of the tunnel's dimensions for brittle rock i.e. rock class II and III and 1% for class IV and V rocks. Slightly higher deformations will be allowed in exceptional cases. Hence, in this analysis, stress relaxation will be realized by appropriate analytical methods (e.g. assume 10 percent of the rock's modulus of elasticity after excavation). The support system will then be designed to prevent further deformation. The use of stress relaxation techniques and analysis methodology is discussed below.

1. Stress relaxation by the core softening method: To simulate the gradual relaxation of the rock mass due to the excavation of the tunnel, different stages were modelled. The initial stage was the initial ground condition (before excavation). Since there are balanced forces in the initial finite element model in this stage, the model is in its initial equilibrium condition. In the next analytical stage, the modulus of deformation of the proposed excavated area was reduced by 90% i.e. core softening of the excavated material, which allows the rock mass to deform and release some the stresses. The application of 90% relaxation of the modulus of deformation was determined from experience in past similar works.

2. Application of the support: In the next stage, the area was excavated, and the supports were applied, which will resist the remaining amount of stress induced loading. Support, consisting of rock bolts, shotcrete, steel rib, etc., was applied at this stage.

3. Total displacement and radius of the plastic zone: After the analysis in the above mentioned stages was complete, the adequacy of the support, total displacement and radius of the plastic zone was checked. The total displacement with supports in place must not exceed a maximum of 0.5% of the tunnel dimensions for brittle rock i.e. rock class II and III or 1% of the tunnel dimensions for class IV and V rocks. Slightly higher deformation was permissible in exceptional cases. If the total displacement after excavation does not exceed 0.1% of the tunnel diameter then no support is required. The radius of the plastic zone after application of support will also be checked for the adequacy of the length of the rock bolts provided. It is expected that normally about 1-2 m of the bolt's anchorage should extend beyond the plastic zone radius. In exceptional cases, a portion of the plastic zone can exceed the elastic limit. In exceptional cases some of the bolts may not meet this criterion.

4. Plot of Yielded Bolts and Axial Stress on Bolts: To further check for the adequacy of the rock bolts, yielded elements on the bolts was plotted together with axial stresses.

5. Support Capacity Plots: Support capacity plots is way of checking the factor of safety of the liner elements (such as shotcrete, steel rib, etc.). This plot includes failure envelopes of the liner support for given parameters, and for different factors of safety. In the support capacity plot the moment vs. thrust and shear force vs. thrust capacity envelopes corresponding to different factors of safety will be plotted and the actual moment vs. thrust and shear force vs. thrust for each finite element of the support was also be plotted. If the point plot of the elements was inside the required FoS envelope, then the support element was safe.

Assumptions

Following major assumptions were made while designing the support system:

1. The rock mass surrounding the excavation was assumed to be homogenous and isotropic in all directions for the generalized Hoek-Brown model. rock mass was modelled as a continuum.

2. Based on the core softening technique, a 90% reduction of the modulus of deformation of the excavated zone will be considered before the application of support.

3. The maximum limit of deformation is 0.5% of the opening's dimensions for brittle rock i.e. rock class II and III and 1% of the opening's dimensions for class IV and V rocks. Slightly higher deformations may be permissible in exceptional cases.

Wedge Analysis

In this analysis, the stability of the excavated profile of the underground tunnel, shaft or cavern was checked against potential wedges that can develop based on identified structural discontinuities in the rock mass. The analysis was carried out considering the excavation geometry of the tunnel, shaft or cavern. The wedge stability and the influence of the proposed support will be evaluated with the Unwedge software from Rocscience, which is based on a limit equilibrium analysis program. The analysis was carried out before and after the application of the support system.

Input Parameters

A) Basic Parameters

The Input parameters adopted for the analysis are as follows:

- Tunnel Axis Orientation
  - Trend in Degree w.r.t. North
  - Plunge in Degrees
- Design Factor of Safety = 1.5

- Unit Weight of Rock = 27 kN/m^3
- Unit Weight of Water = 10 kN/m^3

B) Joint Surface Properties

Given the absence of laboratory investigations, the following joint parameters were considered in the analysis based on field assessments and characteristic values for the joint plane such as roughness, alteration, etc.

Table 7-3: Parameters for the Joint Plane

S.N.	Parameter	Value
1	Angle of Internal Friction	30^o for all joint planes
2	Cohesion	10 kN/m^2 for all joint planes
3	Tensile Strength	0.0 for all joint planes

The assumed parameters are conservative.

C) Bolt Properties

The bolt properties are the same as proposed in the numerical analysis.

D) Shotcrete Properties

The shotcrete properties was the same as adopted in the numerical analysis.

E) Bolting Pattern

The bolting pattern was the same as adopted in the numerical analysis.

Results of Wedge Analysis

Based on combinations of all joints, the results were presented as the calculated factor of safety with and without rock support. The minimum factor of safety of 1.5 was respected. A with minimum support system proposed for any tunnel, shaft and cavern.

7.1.2.2 Design Methods Applied

Two empirical methods to determine the initial rock bolt length and, subsequently, a numerical analysis was performed to complete the analysis and design. The details for these empirical and numerical methods are described in Part B-1 Design Criteria Report of Civil Works.

- Design of the rock support in the Connecting Tunnel crown & wall portions by using Barton et al. (1974) empirical formula and, also, the US Army Corps of Engineers (USACE) “rule of thumb” method based on excavated dimensions to confirm the initial estimate for rock bolt length.

- Design of rock support in Connecting Tunnel crown & wall portion by using Grimstad and Barton (2013) charts

- Numerical method for the final analysis:

  Numerical Analysis of the Connecting Tunnel to the Headpond using the Phase 2 FE programme

7.1.2.3 Design Calculations

Numerical Analysis of the Connecting Tunnel (to Headpond)

Model Generation

A 2-D numerical analysis was carried out with the Phase 2 finite element programme. The proposed section of the Connecting Tunnel was modelled as per its excavated profile. The external fixed boundary was considered as three times its dimension in each direction. A full face excavation of the tunnel is proposed.

For generation of the model, an excavated boundary was created by importing a “DXF” File from AutoCAD. The external boundary was created by considering a box section with an expansion factor of 3 i.e. the external boundary was created at three times its excavated width on both sides, top and bottom. The model was analysed considering three stages of excavation and corresponding support systems.

A close-up view of the 2-D Connecting Tunnel Model is shown in the figure below.

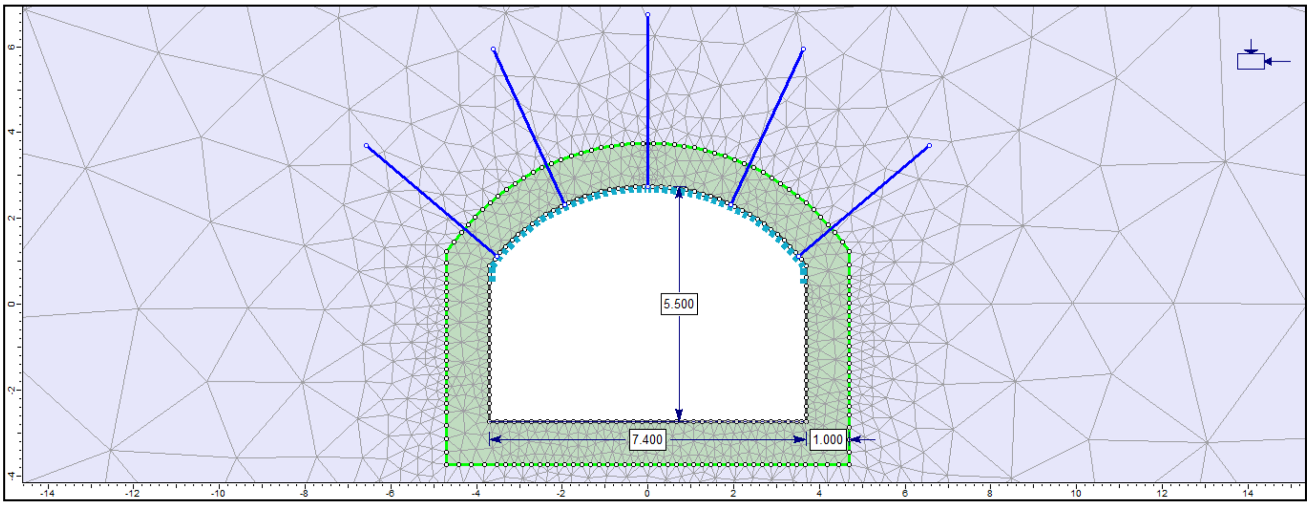

Figure 7-3: Close-up View of a 2-D Model of the Connecting Tunnel

The assumptions entering the model included the stages of excavation, the properties of the rock material, the loading details and the support system. These assumptions are presented in detail in Part A3 Chapter 1 of the report and are not reiterated here.

Results and Discussions

The type of analysis performed was plain strain analysis. Plain strain is defined to be a state of strain in which the strain normal to the x-y plane (< z) and shear strain (< xz and < yz) are assumed to be zero. The details of the various parameters determined after the analysis are given below for each Rock Class.

Conclusion of the Rock Support System

Based on results below, it can be concluded that the deformed shape of the model was as expected. As per this 2-D numerical analysis of the Connecting Tunnel, with the proposed support system, the maximum deformation was about 2.09 mm for Class II rock, 3.53 mm for Class III rock, 7.39 mm for class IV rock and 18.44 mm for Class V rock, which is within the allowable limits. The details are shown in the annexes to Part A3 Chapter 1 of this report.

The maximum limit of the yielded elements from the excavated face was about 1.5 m. and the proposed rock bolt length was 4.0 m, which is safe. The details are shown in the annexes to Part A3 Chapter 1 of this report.

The number of yielded bolts was very small and hence the design is safe. The details are shown in the annexes to Part A3 Chapter 1 of this report.

As per support capacity plots, only a limited portion of the shotcrete has a factor of safety less than 1.0, which is acceptable, and the shotcrete provided is safe. The details are shown in the annexes to Part A3 Chapter 1 of this report.

The recommended support system based on above analysis is given in the following Subsection Results of Calculations.

Wedge Analysis of the Connecting Tunnel

In this study the stability of the excavation profile of the underground Connecting Tunnel to Headpond was checked against potential wedges that can develop based on identified structural discontinuities in the rock mass. The analysis for the Connecting Tunnel was carried out considering the excavation geometry. The wedge stability and the influence of the proposed support was evaluated with the Unwedge software of Rocscience, which is based on a limit equilibrium analysis program. The analysis was carried out before and after the application of the support system.

The assumptions entering the model included the general input parameters, the joint properties and the support system. These assumptions are presented in detail in Part A3 Chapter 1 of the report and are not reiterated here.

Results of the Wedge Analysis

Based on combinations of all joints, a summary of results for wedge stability analysis, before and after support installation, is presented below.

Table 7-4: Summary of wedge stability analysis

S.N.	Joint Combinations	Critical Failure Wedges	Wedge Weight (kN)	Excavated Face Area (m^2)	Factor of Safety (Without support)	Factor of Safety (With Support)
1	J2-J3-J4	Lower Right Wedge (4)	102.13	7.35	1.074	3.776
2	J2-J3-J4	Upper Left Wedge (7)	1.98	0.89	0.867	115.125
3	J2-J3-J4	Roof Wedge (8)	11.97	2.5	0	54.858
4	J1-J3-J4	Upper Left Wedge (7)	28.16	2.84	0.784	24.736

5	J1-J3-J4	Upper Right Wedge (8)	0.07	0.11	0	744.124
6	J1-J2-J3	Upper Right Wedge (6)	218.75	10.66	1.088	5.426
7	J1-J2-J3	Roof Wedge (8)	1.10	0.62	0	216.49

Conclusion Wedge Failure Analysis

The analysis reveals the presence of 7 critical failure wedges. The factor of safety (FOS) at various locations of the cavern assumes greater importance in consideration of the size of this Connecting Tunnel. Any wedge failure in the crown portion, in particular, failure of unconfined wedges around it, could create crown instability with catastrophic consequences. Hence, a very conservative approach to stabilization was adopted. Based on this analysis, based on known joint sets and based on the assumed rock properties, 4.0 m rock bolts achieve the required FOS.

The calculated factor of safety before and after installation of the supports shows that with the applied support, the potential wedges are safe. The critical wedges formed due to various joint combinations are shown in the annexes to Part A3 Chapter 1 of this report.

7.1.2.4 Results of Calculations (Connecting Tunnel rock support)

The recommended support system based on above analysis is given in the table below:

Table 7-5: Summary of Proposed Rock Support for Connecting Tunnel

S.N.	Class of Rock	Proposed Support System
1	Class II	25 mm diameter fully grouted rock bolts @ 2 m c/c in crown only 50 mm thick plain shotcrete in Crown only
2	Class III	25 mm diameter fully grouted rock bolts @ 1.75 m c/c in crown only 50 mm thick plain shotcrete in crown and sides
3	Class IV	25 mm diameter fully grouted rock bolts @ 1.5 m c/c in crown and sides 100 mm thick plain shotcrete in crown and sides with one layer of welded wire mesh of 100x100x4 mm
4	Class V	25 mm diameter fully grouted rock bolts @ 1.25 m c/c in crown and sides 50 mm thick plain shotcrete in Crown and sides Steel Ribs of ISMB150 @ 1 m c/c

In addition to the above support system, drainage holes of 50 mm diameter, 4.0 m length will be provided as per requirement. Also, to avoid corrosion of the steel ribs, an additional layer of 50 mm thick plain shotcrete shall be applied on exposed area of Steel Ribs i.e. for Class V Rock.

A grout with no shrinkage and with the greatest possible strength yet which is still pumpable will be developed by the Contractor and approved by the Engineer. A commercially available mix such as Rescon zinc rock bolt cement mix may also be used

7.1.3 Headworks - Headpond

Design of the Support System for the Headpond

The design of the rock support system for the Headpond requires a detailed analysis of the in situ rock stresses, rock strength parameters in conjunction with the sequence of cavern excavation stages and subsequent application of rock bolt and shotcrete rock support. The deformation modelling of the Headpond against in situ stresses was studied for the analysis and design of the rock support. The 2-D numerical analysis was carried out with the Phase 2 finite element programme for deformation modelling. The analyses and their results are presented below. Initially, the rock support system was estimated based on empirical relations. Subsequently, a numerical analysis was performed. Details are presented below.

Geology of the Headpond

The rock mass at the Headpond area is composed of banded gneiss rock. The rock mass contains quartz, feldspar, biotite, garnet, amphibole, and kyanite. Phenocrysts of quartz, feldspar and garnet are prominent.

The rock mass is coarse to very coarse grained, strong to very strong in strength, have closely to widely spaced foliation joints. The foliation joints (JS0) & some other joints are filled with sandy silty and sometimes clayey material up to 5 mm of thickness. The rock mass is fresh to slightly weathered.

Four well developed main joint sets, including foliation joints (JS0, JS1, JS2 & JS3), were observed above the Headpond area. Among them, the foliation joints (JS0) are the prominent sets. The dip angles & dip directions of the said joints are as follows: JS0 (Foliation joints): 55-65/020-045, JS1: 50-60/080-085, JS2: 30-45/235 & JS3: 80-85/345-350. The rock mass is blocky in general. The said rock bodies are well interlocked.

The rock mass is dry at the outcrop. However, damp to dripping groundwater condition can be anticipated during excavation.

7.1.3.1 Specific Design Criteria

Please refer to above Section on Connecting Tunnel.

7.1.3.2 Design Method Applied

Please refer to above Section on Connecting Tunnel.

7.1.3.3 Design Calculations

<u>Numerical Analysis of the Headpond</u>

<u>Model Generation</u>

A 2-D numerical analysis was carried out with the Phase 2 finite element programme. The proposed section of the Headpond was modelled as per its excavated profile. The external fixed boundary was considered as three times its dimension in each direction.

For generation of the model, an excavated boundary was created by importing a “DXF” File from AutoCAD. The external boundary was created by considering a box section with an expansion factor of 3, i.e. the external boundary was created as three times its excavated width on both sides, top and bottom. The model was analysed considering thirteen stages of excavation and their corresponding support systems. The excavation stages’ boundaries were also created by importing a “DXF” file from AutoCAD.

A close-up view of the 2-D Headpond model is shown in below figure.

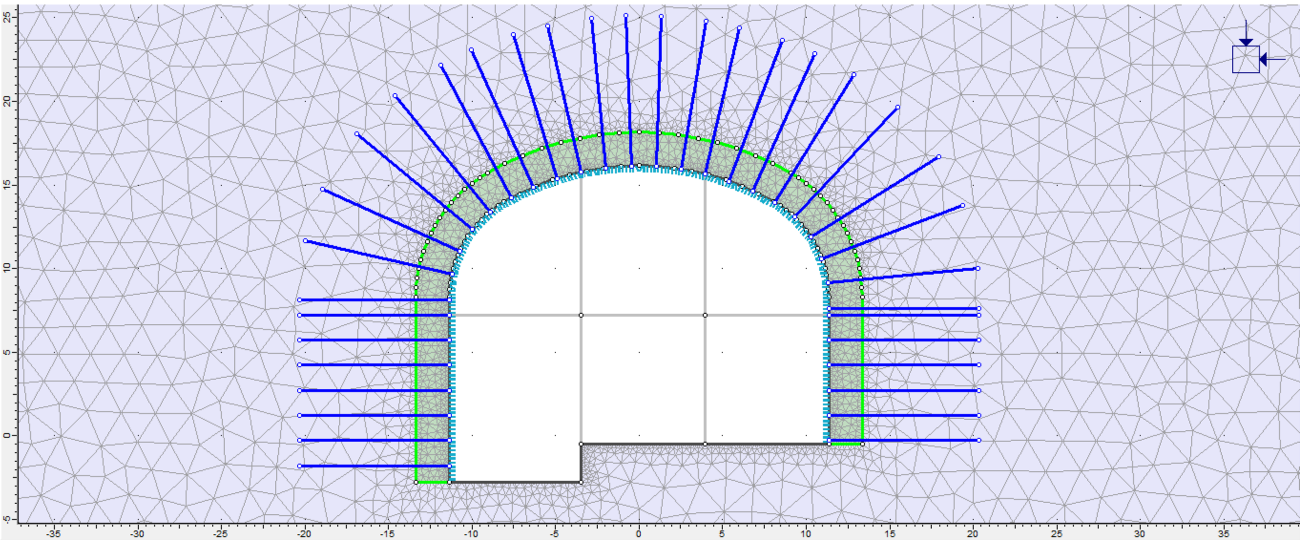

Figure 7-4: Close-up View of 2D Model of Head Pond

The assumptions entering the model included the stages of excavation, the properties of the rock material, the loading details and the support system. These assumptions are presented in detail in Part A3 Chapter 1 of the report and are not reiterated here.

<u>Results and Discussions</u>

The following summary of the various parameters determined after analysis are given below.

Table 7-6: Maximum Displacements

S.N.	Case	Condition	Maximum Displacement (mm)			
			On Left Wall	On Right Wall	At Crown	At Bottom
1	Case-1	Thirteen Stage Excavation with Support	25.6	25.3	17.4	44.8

A) Total displacement

The maximum value of displacement was about 25.6 mm at mid-height of left side wall and 25.3 mm at mid height of the right side wall of the Headpond. At the crown, the displacement was about 17.4 mm and at the bottom, the displacement was about 44.8 mm. Complete details of the model showing displacements is shown in the annexes to Part A3 Chapter 1 of this report.

B) Yielded Zone

The maximum value of the radius of the plastic zone is about 15 m. Thus, the plastic zone is about 3.5 m beyond the excavation boundary. Complete details of the model showing yielded zone is shown in the annexes to Part A3 Chapter 1 of this report.

C) Support Capacity of Rock Bolts

Due to deformation after application of the support system, some rock bolts have yielded. The number of yielded bolts is small and hence safe. The details of the yielded bolts are shown in the annexes to Part A3 Chapter 1 of this report.

D) Support Capacity Plot of Shotcrete

The support capacity plot of the shotcrete is shown the annexes to Part A3 Chapter 1 of this report.

<u>Conclusion</u>

Based on the above results, it was concluded that the deformed shape of the model was as expected. As per this 2-D numerical analysis of the Headpond, with the proposed support system, the maximum deformation is about 44.8 mm, which is about 0.2% of the excavated width and hence within the allowable limit.

The maximum limit of the yielded element from the excavated face is about 3.5 m and the rock bolts are proposed up to a length of 7.5 m, which is safe.

The number of yielded bolts is very small and hence safe.

As per support capacity plots, only limited portions of the shotcrete have a factor of safety less than 1.0, which is acceptable and the shotcrete as designed is safe.

The recommended support system based on the above analysis is given in the following Subsection Results of Calculations.

<u>Wedge Analysis of Headpond</u>

In this study, the stability of the excavation profile of the underground Headpond was checked against potential wedges that can develop based on identified structural discontinuities in the rock mass. The analysis for the Headpond was carried out considering the excavation geometry. The wedge stability and the influence of the proposed support was evaluated by the Unwedge software from Rocscience, which is based on a limit equilibrium analysis program. The analysis was carried out before and after application of the support system.

The assumptions entering the model included the general input parameters, the joint properties and the support system. These assumptions are presented in detail in Part A3 Chapter 1 of the report and are not reiterated here.

Results of Wedge Analysis

Based on combinations of all joints, the summary of results for wedge stability analysis before and after support installation is presented below.

Table 7-7: Summary of Results for Wedge Stability Analysis

S.N.	Joint Combinations	Critical Failure Wedges	Wedge Weight (kN)	Excavated Face Area (m^2)	Factor of Safety (Without support)	Factor of Safety (With Support)
1	J2-J3-J4	Lower Right Wedge (4)	3146.78	72.38	0.615	5.978
2	J2-J3-J4	Upper Left Wedge (7)	11.53	3.13	0.522	95.292
3	J2-J3-J4	Roof Wedge (8)	509.44	30.36	0.000	15.097
4	J1-J3-J4	Upper Right Wedge (4)	1112.19	29.09	1.398	14.586
5	J1-J3-J4	Roof Wedge (7)	671.59	23.99	0.315	9.623
6	J1-J3-J4	Upper Right Wedge (8)	2.56	1.20	0.000	161.292
7	J1-J2-J3	Upper Left Wedge (4)	8.64	2.51	1.029	105.112
8	J1-J2-J3	Upper Right Wedge (6)	6147.39	94.78	0.628	4.513
9	J1-J2-J3	Roof Wedge (8)	47.17	7.63	0.000	67.135

Conclusion regarding wedge failure

The analysis reveals the presence of 9 critical failure wedges. The factor of safety (FOS) at various locations of the cavern assumes greater importance due to the 18m height of the Headpond. Any wedge failure in the crown portion, in particular failure of unconfined wedges around it, could create crown in-stability of catastrophic proportions. Hence, a very conservative approach to stabilization was adopted. Based on this analysis, based on known joint sets and assumed rock properties, 7.5 m rock bolts achieve the required FOS.

The observed factor of safety before and after installation of the rock support shows that with the applied supports, the potential wedges are safe. The critical wedges formed due to various joint and shear zone combinations are shown in the annexes to Part A3 Chapter 1 of this report.

Design of Concrete Lining (40cm thickness) of the Headpond

The objective and scope of this work is the analysis and design of the concrete lining for the Headpond. The concrete lining of open channel flow of the Headpond is subjected to maximum internal pressure during initial filling and maximum external pressure during dewatering / maintenance conditions. The analysis was carried out with Bentley STAAD Pro V8i and the analysis and the results are presented below.

The assumptions entering the model included the general input parameters, the material properties and the loads on the lining. These assumptions are presented in detail in Part A3 Chapter 1 of the report and are not reiterated here.

Based on the STAAD analysis and design, the details of reinforcement proposed for the concrete lining of the Headpond lining are given in the following Subsection Results of Calculations.

7.1.3.4 Results of Calculations

The recommended support system for the Headpond, based on above analysis, is given below:

Table 7-8: Proposed Support System for Headpond

S.N.	Class of Rock	Proposed Support System
1	Class II to III	32 mm diameter, 7.5 m length fully grouted rock bolts @ 1.5 m c/c in crown as well as side walls 200 mm thick plain shotcrete with two layers of wire mesh in Crown as well as side walls

In addition to the above support system, drainage holes of 75 mm diameter, 6.0 m length will be provided as per requirement.

For the reinforcement of the 40cm thick concrete liner of the Headpond, the following reinforcement was chosen:

Table 7-9: Reinforcement for Concrete Lining of Headpond

S.N.	Class of Rock	Concrete Lining Thickness (mm)	Main Reinforcement (Across Flow Direction)	Distribution Reinforcement (Along flow direction)
1	II to III	400	20 mm dia bars @ 200 c/c on inner as well as outer face (water face and rock face)	16 mm dia bars @ 200 c/c on inner as well as outer face (water face and rock face)

7.1.4 Headworks - Spillway

No separate rock support is required for the Spillway Structure. For analysis of the Spillway Structure see "Structural Design" Subchapter 1.2.

7.1.5 Headworks - Spillway Tunnel

Design of Support System for Spillway Tunnel

The design of the rock support system in the Spillway Tunnel requires a detailed analysis of the in situ rock stresses, rock strength parameters, application of rock bolt and shotcrete for rock support. The deformation modelling of the spillway tunnel against in situ stresses have been studied for the analysis and design of the rock support. The 2-D numerical analysis was carried out with the Phase 2 finite element programme for deformation modelling. The analyses and their results are presented below. An Initial, rock support system was estimated based on empirical relations. Subsequently a numerical analysis was performed The analyses and their results are presented below.

Geology of the Spillway Tunnel

The Spillway Tunnel extends from approximately Ch. 20 onward and is composed of bedrock. The section between Ch. 0 and Ch. 20, on the other hand is composed of a colluvial deposit.

The bedrock is composed of banded gneiss. The rock mass contains quartz, feldspar, biotite, garnet, amphibole, and kyanite. Porphyroblasts of quartz, feldspar and garnet are frequently observed. Likewise, blue or green kyanite blades are also observed in biotite rich bands.

The rock mass is coarse to very coarse grained, strong to very strong in strength, has closely to widely spaced foliation joints. The foliation joints (JS0) & some other joints are filled with sandy silty and sometimes clayey material up to 5mm of thickness. The rock mass is slightly weathered along the joint surfaces.

Four well developed main joint sets including foliation joints (JS0, JS1, JS2 & JS3) were observed at the Spillway area. The foliation joints (JS0) are the prominent sets. The dip angles & dip directions of the said joints are as follows: JS0 (Foliation joints): 55-65/020-045, JS1: 50-60/080-085, JS2: 30-45/235 & JS3: 80-85/345-350. The rock mass is blocky in general. The said rock bodies are well interlocked.

The rock mass is dry at the outcrop. However, damp to dripping groundwater conditions can be anticipated during excavation.

7.1.5.1 Specific Design Criteria

Please refer to above Section on Connecting Tunnel.

7.1.5.2 Design Method Applied

Please refer to above Section on Connecting Tunnel.

7.1.5.3 Design Calculations

Numerical Analysis of the Spillway Tunnel

Model Generation

A 2-D numerical analysis was carried out with the Phase 2 finite element programme. The proposed section of the Spillway Tunnel was modelled as per its excavated profile. The external fixed boundary was considered as three times its dimensions in each direction. Full face excavation of the Spillway Tunnel is proposed.

For generation of the model, the excavated boundaries were created by importing a “DXF” File from AutoCAD. The external boundary was created by considering a box section with an expansion factor of 3 i.e. the external boundary was created as three times its excavated width on both sides and top and bottom. The model was analysed considering fourteen stages of excavation and corresponding support systems. The excavation stages’ boundaries were also created by importing “DXF” files from AutoCAD.

A close-up view of the 2-D Spillway Tunnel model is shown in below figure.

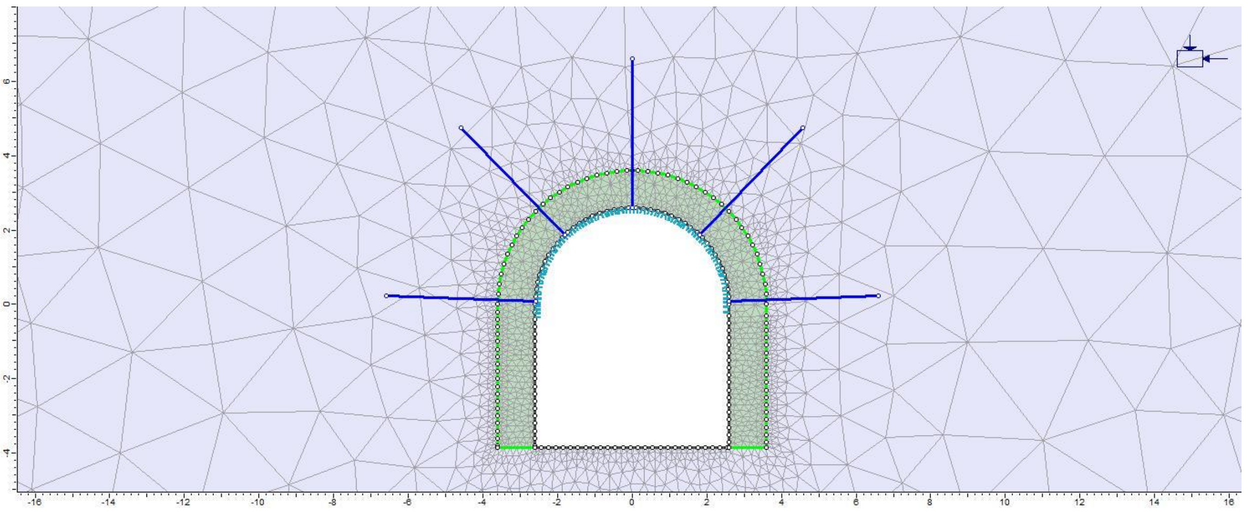

Figure 7-5: Close-up View of 2-D Model of Spillway Tunnel

The assumptions entering the model included the stages of excavation, the properties of the rock material, the loading details and the support system. These assumptions are presented in detail in Part A3 Chapter 1 of the report and are not reiterated here.

Results and Discussions

The type of analysis performed was plain strain analysis. Plain strain is defined to be a state of strain in which the strain normal to the x-y plane (< z) and shear strain (< xz and < yz) are assumed to be zero. The details of the various parameters determined after analysis are given below for each rock class.

Conclusion

Based on above results, it can be concluded that deformed shapes of the model are as expected. As per this 2D numerical analysis of the Spillway Tunnel, with proposed support system, the maximum deformation is about 3.3 mm for Class II rock, 6.51 mm for Class III rock, 13.1 mm for class IV rock, 30.0 mm for Class V rock and 42.1 mm for class VI rock, which are within the allowable limit. The details are shown in the annexes to Part A3 Chapter 1 of this report.

The maximum limit of the yielded element from the excavated face is about 2 m and rock bolts are proposed up to a length of 4.0 m, which is safe. The details are shown in the annexes to Part A3 Chapter 1 of this report.

The number of yielded bolts is very limited and hence safe. The details are shown in the annexes to Part A3 Chapter 1 of this report.

As per support capacity plots, only limited portions of the shotcrete have a factor of safety less than 1.0, which is acceptable and the shotcrete, as designed is safe. The details are shown in the annexes to Part A3 Chapter 1 of this report.

The recommended support system based on above analysis is given in the Subsection Results of Calculations below.

Wedge Analysis of Spillway Tunnel

In this study, the stability of the excavation profile of the underground Spillway Tunnel is checked against potential wedges that can develop based on identified structural discontinuities in the rock mass. The analysis for the Spillway Tunnel is carried out considering the excavation geometry. The wedge stability and the influence of the proposed support is evaluated with the unwedge software from Rocscience, which is based on a limit equilibrium analysis program. The analysis was carried out before and after the application of the support system.

The assumptions entering the model included the general input parameters, the joint properties and the support system. These assumptions are presented in detail in Part A3 Chapter 1 of the report and are not reiterated here.

Results of Wedge Analysis

Based on combinations of all joints, the summary of results for wedge stability analysis before and after support installation is presented below.

Table 7-10: Summary of wedge stability analysis

S.N.	Joint Combinations	Critical Failure Wedges	Wedge Weight (kN)	Excavated Face Area (m^2)	Factor of Safety (Without support)	Factor of Safety (With Support)
1	J2-J3-J4	Roof Wedge (8)	16.20	2.45	0.000	28.183
2	J1-J3-J4	Lower Left Wedge (3)	30.01	5.79	0.723	3.647
3	J1-J3-J4	Roof Wedge (7)	1.82	0.80	0.000	21.981
4	J1-J3-J4	Upper Right Wedge (8)	2.56	1.20	0.000	130.508
5	J1-J2-J3	Upper Left Wedge (4)	114.47	7.57	1.27	7.101
6	J1-J2-J3	Roof Wedge (8)	0.12	0.15	0.000	1351.47

<u>Conclusion Wedge Failure Analysis</u>

The analysis reveals the presence of 6 critical failure wedges. Due to the size of the Spillway Tunnel at various locations, the factor of safety (FOS) has increased importance. of Any wedge failure in the crown portion, in particular failure of unconfined wedges around it, could create crown instability and would be catastrophic. Hence, a very conservative approach to stabilization was adopted. Based on this analysis, known joint sets and assumed rock properties, 4.0 m rock bolts achieve the required FOS.

The observed factor of safety before and after the installation of support shows that with the applied support, the potential wedges are safe. The critical wedges formed due to various joint combinations are shown in the annexes to Part A3 Chapter 1 of this report.

Design of Concrete Lining of Spillway Tunnel

The objective and scope of work is the analysis and design of a 30cm thick concrete lining for the Spillway Tunnel. The concrete lining of free flow Spillway Tunnel is subjected to a maximum internal pressure during flow of water through the spillway and a maximum external pressure during maintenance conditions. The analysis has been carried out with the Bentley STAAD Pro V8i program and analysis.

The assumptions entering the model included the general input parameters, the material properties and the loads on the lining. These assumptions are presented in detail in Part A3 Chapter 1 of the report and are not reiterated here.

Based on STAAD software analysis and design, the details of reinforcement proposed for the concrete lining of the Spillway Tunnel are given in below Subsection Results of Calculations.

7.1.5.4 Results of Calculations

The recommended support system for the Spillway Tunnel, based on above analysis, is given below:

Table 7-11: Proposed Support System for Spillway Tunnel

S.N.	Class of Rock	Proposed Support System
1	Class II	25 mm diameter fully grouted rock bolts @ 2 m c/c in crown only 50 mm thick plain shotcrete in crown only
2	Class III	25 mm diameter fully grouted rock bolts @ 1.75 m c/c in-crown only 50 mm thick plain shotcrete in crown and sides
3	Class IV	25 mm diameter fully grouted rock bolts @ 1.5 m c/c in crown and sides 100 mm thick plain shotcrete in crown and sides with one layer of welded wire mesh of 100x100x4 mm
4	Class V	25 mm diameter fully grouted rock bolts @ 1.25 m c/c in crown and sides 50 mm thick plain shotcrete in crown and sides Steel Rib of ISHB150 @ 1 m c/c
5	Class VI	25 mm diameter fully grouted rock bolts @ 1.5 m c/c in crown and sides 50 mm thick plain shotcrete in crown and sides Steel Rib of ISHB150 @ 0.5m c/c

In addition to the above support system, drainage holes of 50 mm diameter, 4.0 m length will be provided as per requirement. Also, to avoid corrosion of steel ribs, an additional layer of 50 mm thick plain shotcrete shall be applied on exposed area of steel ribs i.e. for Class V rock.

The following table summarizes the reinforcement for the liner of the Spillway Tunnel liner:

Table 7-12: Summary of Reinforcement

S.N.	Class of Rock	Concrete Lining Thickness (mm)	Main Reinforcement (Across Flow Direction)	Distribution Reinforcement (Along flow direction)
1	II to III	300	16 mm dia bars @ 200mm c/c on inner as well as outer face (water face and rock face)	12 mm dia bars @ 200mm c/c on inner as well as outer face (water face and rock face)

7.1.6 Headworks - Spillway Terminal Structure

This section addresses the design of the support system for the Spillway Tunnel Portal.

7.1.6.1 Specific Design Criteria

Please refer to above Section on Connecting Tunnel.

7.1.6.2 Design Method Applied

A detailed slope stability analysis was carried out for the proposed slope at the highest cut at the portal location to estimate the factor of safety for this slope. Excavation was planned in two phases, the lowest bench will have a height of 18 m.

A 2-D slope stability analysis was carried out with Phase2 software using the shear strength reduction approach. The proposed excavated section of the Spillway Tunnel Portal was modelled at the location of the portal. The stability calculations were carried out with and without support systems in place and including / excluding seismic loads.

To determine properties of rock mass, the Roclab software was used.

7.1.6.3 Design Calculations

For generation of the model, an external boundary was created by importing an AutoCAD DXF file. The material boundary was also created by importing an AutoCAD DXF file.

2-D slope stability analysis was carried out with Phase2 software using the shear strength reduction approach. The proposed excavated section of the Spillway Tunnel Portal is modelled at the location of the portal. The stability calculations were carried out with and without support systems in place and including / excluding seismic loads. The excavated slope modelled with the Phase2 program is given in below figure.

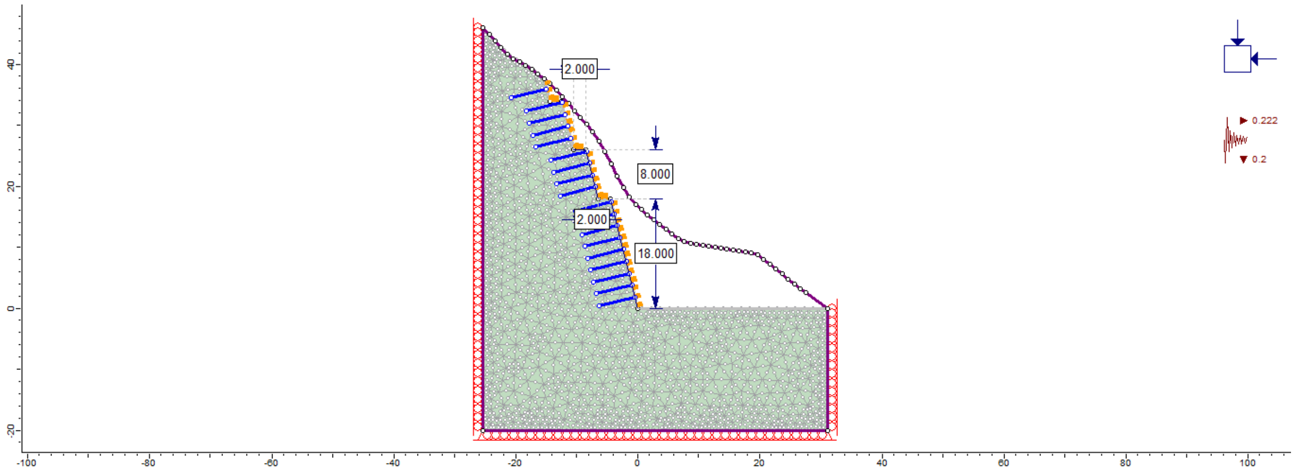

Figure 7-6: Excavated Slope of Spillway Tunnel Portal Modelled in Phase2 Software

The assumptions entering the model included the general input parameters, the material properties and the loads on the lining. These assumptions are presented in detail in Part A3 Chapter 1 of the report and are not reiterated here.

7.1.6.4 Results of Calculations

The proposed excavated slopes is 4V:1H with 2m bench at every 8 m in height of heading with 100 mm thick shotcrete with one layer of wire mesh of 100x100x4 mm and 6m long rock bolts at @ 2 m c/c staggered. The excavated section for slope stability analysis have been considered accordingly. Based on the results of slope stability analysis, the summary of factor of safety obtained under various scenario is given below. The factor of safety obtained under various conditions are shown below in the annexes to Part A3 Chapter 1 of this report.

Table 7-13: Factor of safety under different conditions

S.N.	Case	Condition	Factor of Safety (Without Support System)	Factor of Safety (With Support System)
1	Case-1	Normal Case	1.52	1.62
2	Case-2	Seismic Case	1.1	1.2

Based on above results, it can be concluded that proposed slopes are stable along portal excavation with required support system. The minimum factor of safety with support system is 1.5 under normal condition and 1.2 under seismic condition, which reflects a safe design.

7.1.7 Power Waterways - General

This chapter covers the analysis and design of the rock support systems for the following water carrying tunnels:

- Headrace Tunnel
- Surge Tank
- U/S Valve Chamber
- Pressure Shaft
- High Pressure Tunnel & Upstream Manifolds
- Tailrace Tunnel & Downstream Manifolds

The concrete linings of the hydraulic tunnels i.e. Headrace Tunnel, Surge Tank and Tailrace Tunnel are also presented.

7.1.8 Power Waterways - Headrace Tunnel

The design of rock support system for Headrace Tunnel requires a detailed analysis of the in situ rock stresses, rock strength parameters, application of rock bolts and shotcrete for the rock support. The deformation modelling of the headrace tunnel corresponding to in situ stresses have analyzed and the corresponding rock support designed. The 2-D numerical analysis was carried out with the Phase 2 finite element program based on deformation modelling. Initially, the rock support systems were estimated based on empirical relations. Subsequently, a numerical analysis was performed. The analysis and design are presented below.

7.1.8.1 Specific Design Criteria

Geology

The geology of the Headrace Tunnel was divided into 8 different reaches. Each reach is described separately below:

Headrace Tunnel from Adit 1 towards upstream

The Headrace Tunnel from Adit 1 extending upstream is composed of banded Gneiss. The rock mass contains quartz, feldspar, biotite, garnet, amphibole, and kyanite. Phenocrysts of quartz, feldspar and garnet are prominent.

The rock mass is coarse to very coarse grained, strong to very strong in strength having closely to widely spaced foliation joints. The foliation joints (JS0) & some other joints are filled with sandy silt and sometimes clayey material up to 5mm of thickness. The rock mass is fresh to slightly weathered.

Four well developed main joint sets, including foliation joints (JS0, JS1, JS2 & JS3), are observed above the HRT area. Among them, the foliation joints (JS0) are the prominent sets. The dip angles & dip directions of the said joints are as follows: JS0 (Foliation joints): 55-65/020-045, JS1: 50-60/080-085, JS2: 30-45/235 & JS3: 80-85/345-350. The rock mass is blocky in general. The said rock bodies are well interlocked.

The rock mass is dry at the outcrop. However, damp to wet groundwater condition can be anticipated during excavation.

Headrace Tunnel from Adit 1 towards downstream

The Headrace Tunnel from Adit 1 extending downstream is composed of banded gneiss, garnet schist, amphibolite and quartzite. The banded gneiss and the rest of the rocks are delineated by the Main Central Thrust (MCT). The MCT zone is presumed to be encountered at around Ch. 1077.

The rock mass tentatively from Ch. 0.0 to Ch. 1077 m is presumed to be banded gneiss. The rock mass contains quartz, feldspar, biotite, garnet, amphibole, and kyanite. Phenocrysts of quartz, feldspar and garnet are prominent.

The rock mass is coarse to very coarse grained, strong to very strong in strength having closely to widely spaced foliation joints. The foliation joints (JS0) & some other joints are filled with sandy silty and sometimes clayey material up to 5mm of thickness. The rock mass is fresh to slightly weathered.

Two well developed main joint sets including foliation joints (JS0 & JS1) and some random joints were observed above the HRT area. Among them, the foliation joints (JS0) are the prominent sets. The dip angles & dip directions of the said joints are as follows: JS0 (Foliation joints): 55-65/020-045, JS1: 55-75/080-095. The rock mass is blocky in general. The said rock bodies are well interlocked.

The rock mass is dry at the outcrop. However, damp to wet groundwater condition can be anticipated during excavation.

The rock mass tentatively from Ch. 1077 m to Ch. 1785 m is presumed to be garnet schist, amphibolite and quartzite.

The rock mass is coarse to very coarse grained, strong to very strong in strength having closely to widely spaced foliation joints. The foliation joints (JS0) & some other joints are filled with sandy silty and sometimes clayey material up to 5mm of thickness. The rock mass is fresh to slightly weathered.

Four well developed main joint sets including foliation joints (JS0, JS1, JS2 & JS3) joints are observed above the HRT area. Among them, the foliation joints (JS0) are the prominent sets. The dip angles & dip directions of the said joints are as follows: JS0 (Foliation joints): 40-50/020-035, JS1: 55-75/080-095, JS2: 60-65/170-190 & JS3: 80-85/290. The rock mass is blocky in general. The said rock bodies are well interlocked.

The rock mass is dry at the outcrop. However, damp to wet groundwater condition can be anticipated during excavation.

Headrace Tunnel from Adit 2 towards upstream

The Headrace Tunnel from Adit 2 extending upstream is composed of metacarbonate, graphitic schist, biotite schist, garnet schist, amphibolite and quartzite.

Rock mass approximately from Ch. 1785 m to Ch. 2059 m, Ch. 2390 m to Ch. 2679 m and Ch. 3138 m to Ch. 3880 m is presumed to be metacarbonate with graphitic schist bands. Likewise, the Headrace Tunnel, approximately, from Ch. 2059 m to Ch. 2390 m, Ch. 2679 m to Ch. 3138 m and Ch. 3880 m to Ch. 4101 m is composed of laminate graphitic schist & biotite schist bands

Metacarbonates are primarily rolomites. They are bluish gray, fine to medium grained, strong in strength, having closely to moderately spaced foliation joints. Graphitic schist & biotite schist are gray to dark gray, weak to medium strong in strength, having very closely spaced foliation joints. The foliation joints (JS0) & some other joints are filled with silty clayey material up to 1mm of thickness. The rock mass is fresh to slightly weathered along the joint surfaces.

Three well developed main joint sets including foliation joints (JS0, JS1 & JS2) were observed at the said area of HRT. Among them, the foliation joints (JS0) are the prominent sets. The dip angles & dip directions of the said joints are as follows**:** JS0 (Foliation joints): 30-40/040-050, JS1: 75/145 & JS2: 45-60/220-240. Metacarbonate rocks are blocky in general, whereas, graphitic schists & biotite schists are slabby. The said rock bodies are well interlocked.

Rock mass is dry at the outcrop. However, damp to wet groundwater condition can be anticipated during excavation.

<u>HRT from Adit 2 towards downstream</u>

The HRT from Adit 2 extending downstream is composed of metacarbonate, graphitic schist & biotite schist.

Rock mass approximately from Ch. 2059 m to Ch. 2390 m, Ch. 2679 m to Ch. 3138 m and Ch. 3880 m to Ch. 4101 m is composed of laminate graphitic schist & biotite schist bands. Likewise, The tunnel from approximately from Ch. 1785 m to Ch. 2059 m, Ch. 2390 m to Ch. 2679 m and Ch. 3138 m to Ch. 3880 m is presumed to be metacarbonates with graphitic schist bands.

Graphitic schist & biotite schist are gray to dark gray, weak to medium strong in strength, having very closely spaced foliation joints. The foliation joints (JS0) & some other joints are filled with silty clayey material up to 1mm of thickness. The rock mass is fresh to slightly weathered along the joint surfaces.

Metacarbonates are primarily dolomites. They are bluish gray, fine to medium grained, strong in strength and have closely to moderately spaced foliation joints. The foliation joints (JS0) & some other joints are filled with silty clayey material up to 1mm of thickness. The rock mass is fresh to slightly weathered along the joint surfaces.

Metacarbonates also contain talc and magnesite especially at the Tatopani area. Talc dominant outcrops are not visible on the surface. However, layers of talc is likely found along foliation joints of metacarbonates. Few outcrops of magnesite have been observed around the tunnel alignment.

Three well developed main joint sets including foliation joints (JS0, JS1 & JS2) were observed at the said area of HRT. Among them, the foliation joints (JS0) are the prominent sets. The dip angles & dip directions of the said joints are as follows: JS0 (Foliation joints): 30-40/020-040, JS1: 65-75/145-160 & JS2: 45-60/220-

240. Metacarbonate rocks are blocky in general, graphitic schists & biotite schists are slabby. Generally, the rock mass at the Tatopani area is sheared and fractured.

The rock mass is dry at the outcrop. However, wet to slowly flowing groundwater conditions can be anticipated during excavation.

<u>Headrace Tunnel from Adit 3 towards upstream</u>

The Headrace Tunnel from Adit 3 towards upstream is composed of chlorite schist, metasandstone, quartzite, graphitic Schist & Biotite Schist.

Rock mass approximately from Ch. 4306 m to Ch. 4864 m is composed of chlorite schist and metasandstone. Rock mass approximately from Ch. 4101 m to Ch. 4306 m is composed of pale grey to yellow quartzite with schist bands.

Chlorite schist and meta sandstones are fine to medium grained, have very closely to closely spaced foliation joints, fresh to slightly weathered and medium strong to strong in strength.

Joints are tight in general. However, the foliation joints (JS0) & some other joints are sometimes filled with silty & clayey material up to 2mm of thickness.

Four well developed main joint sets including foliation joints (JS0, JS1, JS2 & JS3) joints were observed around the HRT area. Among them, the foliation joints (JS0) are the prominent sets. The dip angles & dip directions of the said joints are as follows: JS0 (Foliation joints): 30-40/030-040, JS1: 65-70/050-060, JS2: 30-35/170 & JS3: 80-85/280-285. The rock mass is slabby in general. The said rock bodies are well interlocked.

The rock mass is dry at the outcrop. However, damp to dripping groundwater condition can be anticipated during excavation.

The rock mass is followed from the north by pale gray to yellow Quartzite with Schist bands. The rock mass is fine to medium grained, have very closely to moderately spaced foliation joints, fresh to slightly weathered and strong to very strong in strength.

Joints are tight in general. However, the foliation joints (JS0) & some other joints are sometimes filled with talc & silty, clayey material up to 5mm of thickness.

Three well developed main joint sets including foliation joints (JS0, JS1 & JS2) joints are observed around the HRT area. Among them, the foliation joints (JS0) are the prominent sets. The dip angles & dip directions of the said joints are as follows: JS0 (Foliation joints): 40-50/025-035, JS1: 65-80/130-140 & JS2: 70-80/230-240. The rock mass is blocky & slabby. The said rock bodies are well interlocked.

Quartzite is followed to the north by Graphitic schist and Biotite Schist bands. The rock mass is gray to dark gray, have very closely spaced foliation joints, fresh to slightly weathered and weak to medium strong in strength.

Joints are tight in general. However, the foliation joints (JS0) & some other joints are filled with silty clayey material up to 2mm of thickness.

Three well developed main joint sets including foliation joints (JS0, JS1 & JS2) were observed around HRT. Among them, the foliation joints (JS0) are the prominent sets. The dip angles & dip directions of the said joints are as follows: JS0 (Foliation joints): 30-40/040-050, JS1: 65-75/145-160 & JS2: 45-60/220-240. The rock mass is slabby in general. The said rock bodies are well interlocked.

Metacarbonates are primarily dolomites. They are bluish gray, fine to medium grained, strong in strength, have closely to moderately spaced foliation joints. The foliation joints (JS0) & some other joints are filled with silty clayey material up to 1mm of thickness. The rock mass is fresh to slightly weathered along the joint surfaces.

Metacarbonates also contain talc and magnesite especially at Tatopani area. Talc dominant outcrops are not visible on the surface. However, layers of Talc are likely found along foliation joints of metacarbonates. Few outcrops of magnesite have been observed around the tunnel alignment.

Three well developed main joint sets including foliation joints (JS0, JS1 & JS2) are observed at the said area of HRT. Among them the foliation joints (JS0) are the prominent sets. The dip angles & dip directions of the said joints are as follows: JS0 (Foliation joints): 30-40/020-040, JS1: 65-75/145-160 & JS2: 45-60/220-240. Metacarbonate rocks are blocky in general whereas graphitic schists & biotite schists are slabby. Apart from that, the rock mass at Tatopani area is sheared and fractured.

The rock mass is dry at the outcrop. However, wet to slowly flowing groundwater condition can be anticipated during excavation.

<u>HRT from Adit 3 towards downstream</u>

The HRT from Adit 3 extending downstream is composed of chlorite schist & metasandstone, Augen gneiss and chlorite schist & phyllite.

Rock mass approximately from Ch. 4306 m to Ch. 4864 m is composed of chlorite schist and metasandstone. Likewise, rock mass approximately from Ch. 4864 m to Ch. 5255 m is composed of Augen gneiss with chlorite schist partings. Similarly, rock mass approximately from Ch. 5255 m to Ch. 8186 m is composed of Chlorite schist and Phyllite.

Chlorite schist and meta sandstones are fine to medium grained, have very closely to closely spaced foliation joints, fresh to slightly weathered and medium strong to strong in strength.

Joints are tight in general. However, the foliation joints (JS0) & some other joints are sometimes filled with silty & clayey material up to 2mm of thickness.

Four well developed main joint sets including foliation joints (JS0, JS1, JS2 & JS3) joints are observed around the HRT area. Among them, the foliation joints (JS0) are the prominent sets. The dip angles & dip directions of the said joints are as follows: JS0 (Foliation joints): 30-40/030-040, JS1: 65-70/050-060, JS2: 30-35/170 & JS3: 80-85/280-285. The rock mass is slabby in general. The said rock bodies are well interlocked.

The rock mass is dry at the outcrop. However, damp to dripping groundwater condition can be anticipated during excavation.

Likewise, Augen gneiss has a gneissic texture. The rock mass contains quartz, feldspar, muscovite and biotite. Phenocrysts of quartz and feldspar are frequently observed. The rock mass is coarse to very coarse grained, having closely to widely spaced foliation joints, fresh to slightly weathered and strong to very strong in strength.

Joints are tight in general. However, the foliation joints (JS0) & some other joints are sometimes filled with sandy, silty & clayey material up to 5mm of thickness.

Three well developed main joint sets including foliation joints (JS0, JS1 & JS2) joints are observed around the HRT area. Among them, the foliation joints (JS0) are the prominent sets. The dip angles & dip directions of the said joints are as follows: JS0 (Foliation joints): 40-45/040-045, JS1: 45-60/230-245 & JS2: 80-85/290-295. The rock mass is blocky in general. The said rock bodies are well interlocked.

The rock mass is dry at the outcrop. However, damp to dripping groundwater condition can be anticipated during excavation.

Similarly, Chlorite schist and phyllite has very closely to closely spaced foliation joints, fresh to slightly weathered and weak to strong in strength.

The foliation joints (JS0) & some other joints are filled with silty, clayey material up to 5mm of thickness.

Three well developed main joint sets including foliation joints (JS0, JS1 & JS2) joints are observed around the HRT area. Among them, the foliation joints (JS0) are the prominent sets. The dip angles & dip directions of the said joints are as follows: JS0 (Foliation joints): 25-35/020-040, JS1: 80-85/280-290 & JS2: 80-85/350-355. The rock mass is slabby in general. The said rock bodies are well interlocked.

The rock mass is dry at the outcrop. However, damp to wet groundwater conditions can be anticipated during excavation.

<u>HRT from Adit 4 extending upstream</u>

The HRT from Adit 4 extending upstream is composed of chlorite schist and phyllite. The rock mass has very closely to closely spaced foliation joints, fresh to slightly weathered and weak to strong in strength.

The foliation joints (JS0) & some other joints are filled with silty, clayey material up to 5mm of thickness.

Three well developed main joint sets including foliation joints (JS0, JS1 & JS2) joints are observed around the HRT area. Among them, the foliation joints (JS0) are the prominent sets. The dip angles & dip directions of the said joints are as follows: JS0 (Foliation joints): 25-35/020-040, JS1: 80-85/280-290 & JS2: 80-85/350-355. The rock mass is slabby in general. The said rock bodies are well interlocked.

The rock mass is dry at the outcrop. However, damp to wet groundwater condition can be anticipated during excavation.

HRT from Adit 4 towards downstream

The HRT from Adit 4 extending upstream is composed of chlorite schist and phyllite. The rock mass has very closely to closely spaced foliation joints, fresh to slightly weathered and weak to strong in strength.

The foliation joints (JS0) & some other joints are filled with silty, clayey material up to 5mm of thickness.

Three well developed main joint sets including foliation joints (JS0, JS1 & JS2) joints are observed around the HRT area. Among them, the foliation joints (JS0) are the prominent sets. The dip angles & dip directions of the said joints are as follows: JS0 (Foliation joints): 25-35/020-040, JS1: 80-85/280-290 & JS2: 80-85/350-355**.** The rock mass is slabby in general. The said rock bodies are well interlocked.

Design Loads

The concrete lining for the underground hydraulic tunnel is subjected to the following loads:

Self-Weight of Lining

The self-weight of lining is considered in the design considering unit weight of concrete about 24 kN/m^3.

Rock Load

No rock load from excavation will be assumed to be supported by the concrete lining, as adequate rock support provisions have been made to support these loads. However, for squeezing rock, some percentage of rock load will be supported by concrete lining.

Contact Grout Pressure

For free flow (open channel) hydraulic tunnels, no contact grouting was proposed. For pressure flow tunnels, the contact grouting will be considered and a maximum pressure of 250 kN/m^2 will be applied in the external boundary for lining design. The lining will be designed to resist safely this grout pressure. Contact grouting is proposed for all classes of rocks for the pressure flow tunnels.

Consolidation Grout Pressure

For free flow hydraulic tunnels, no consolidation grouting is proposed. For the pressure flow tunnels, the maximum design consolidation grouting pressure will be 500 kN/m^2. However, the concrete lining will be designed for 50% of the consolidation grout pressure. The consideration of 50% grout pressure on concrete lining is as per para 6.1.2.2 (C) of CBIP manual on the planning and design of Hydraulic Tunnels. Thus, the consolidation grout pressure that will be considered is 50% of 500 kN/m^2 = 250 kN/m^2.

According to EM 1110-2-2901 (“Engineering Design of Tunnels and Shafts in Rocks”), the maximum influence area for localized grout pressure from a single grout hole is 1.5 m x 1.5 m. Hence, for the structural analysis of the concrete lining, the grout pressure will be applied locally to 1.5m lengths of the tunnel lining. This will simulate the actual grouting sequence and promote a better understanding of the behaviour of the lining elements under local grout pressure. The lining design will be designed to safely resist this grout pressure.

Consolidation grouting is proposed for rock class support IV, V and VI. In addition to these rock support classes, the consolidation grouting is proposed for the following additional areas:

- Seepage zones along water the conductor system.
- Near all Adit junctions extending about 50 m from both sides of the junction.
- Extend about 5 m beyond either side of a Class IV, V and VI rock sections, when these zones are adjacent to Rock Class II and III sections.

Internal Water Pressure

During normal operation, there will be a build-up of external water pressure in the surrounding rock mass facilitated by the arrangement of drainage holes in the lining and also by radial and longitudinal cracks in the lining. For these reasons, internal and external water pressure will balance each other, therefore, internal water pressure need not be considered in the concrete liner design.

However, during the initial filling operation of the hydraulic tunnel, the surrounding rock mass may not have time to fully charge the surrounding rock mass. Due to presence of adequate rock confinement, some percentage of internal water pressure will be transmitted to the surrounding rock mass and residual internal pressure will act on the lining during filling up operation. Therefore, to be on conservative side, 100% of the internal pressure will be assumed to act on the concrete lining.

During transient conditions, the time period of hydraulic transient being very small, the maximum internal pressure would not have sufficient time to balance out and the differential pressure will be exerted on the lining.

Hence, the maximum internal pressure will be the higher of the following:

- Difference between the maximum static water level (FRL) and the tunnel invert level
- Difference between the maximum upsurge level and the steady state water level

However, for locations, where rock confinement is not sufficient (low rock cover zones), the maximum internal pressure will be considered as the difference between the maximum upsurge level and the tunnel invert level.

External Water Pressure

During dewatering of the tunnel, the internal pressure on lining reduces to almost nil. In this case, the concrete lining will be subjected to external pressure which will tend to reduce due to the inflow of water through drainage holes and cracks. The maximum external water level will be considered the maximum static water level (FRL) or the maximum ground water level, whichever is higher. In case of a higher ground water level, the external pressure will be reduced by 50% because of the permeability of concrete lining (drainage holes and cracks) and the permeability of the rock mass.

During transient conditions, (dewatering of the tunnel) the time period of hydraulic transients is rather long, therefore, the maximum external pressure will not have sufficient time to dissipate and, therefore, the external water pressure will be exerted on the lining.

Hence, the maximum external pressure will be the higher of the following:

- Difference between the maximum static water level (FRL) / ground water level and the tunnel excavated invert level
- Difference between the steady state water level and the minimum down surge level.

Seismic Loads

The seismic loads may also be considered by using seismic coefficients corresponding to Operation Basic Earthquake (OBE). According to the Design Basis Report, the value of the horizontal and the vertical seismic coefficients for the OBE case is 0.222 and 0.2 respectively.

However, for underground hydraulic tunnels with rock cover more than 50 m, the seismic load need not be considered.

Load Combinations

During the service life of the tunnel, the secondary lining is subjected to the following loading conditions:

Table 7-14: Loading Conditions

S.N.	Loading Condition	Design Loads
1	Tunnel Under Construction	Self-Weight of Lining + Maximum Grout Pressure
2	Tunnel Initial Filling Condition	Self-Weight of Lining + Maximum Internal Pressure
3	Tunnel Dewatering Condition	Self-Weight of Lining + Maximum External Pressure

Geotechnical Data

The values of the modulus of deformation and Poisson's ratio of rock mass will be used for determining of spring coefficients. Concrete linings will be done adjoining the rock surface. After contact grouting, surrounding rock and concrete lining will be monolithic. During analysis and design, the lining is assumed to be supported by rock springs, where a equivalent spring stiffness will be calculated. In some cases, it will be required to anchor the concrete lining directly to the rock. Appropriate rock anchors will be designed for this situation.

Material Properties

The properties of the material to be used for concrete lining are given below:

- Grade of Concrete : M30 (C25/30)
- Characteristic compressive Strength of concrete (fck) : 30 N/mm^2
- Unit weight of Concrete Lining : 24 kN/m^3
- Permissible Stresses in Concrete (As per IS 456: 2000)
  - In Direct Compression : 8.0 N/mm^2
  - In Flexural compression : 10.0 N/mm^2
- Permissible Flexural tensile stress of Concrete (With factor of safety 1.5): 0.7xfck^0.5/1.5 = 2.556 N/mm^2
- For extreme / severe loading conditions, permissible stresses will be increased by 33.33% according to Clause 6.1.1 of IS 4880 (Part IV) - 1971

7.1.8.2 Design Method Applied

Two empirical methods to determine the initial rock bolt length and, subsequently, a numerical analysis was performed to complete the analysis and design. The details for these empirical and numerical methods are described in Part B-1 Design Criteria Report of Civil Works.

- Design of the rock support in the Headrace Tunnel crown & wall portions by using Barton et al. (1974) empirical formula and, also, the US Army Corps of Engineers (USACE) “rule of thumb” method based on excavated dimensions to confirm the initial estimate for rock bolt length.
- Design of rock support in Headrace Tunnel crown & wall portion by using Grimstad and Barton (2013) charts

Numerical method for the final analysis:

- Numerical Analysis of the Headrace Tunnel to the Headpond using the Phase 2 FE programme

7.1.8.3 Design Calculations

The design calculations included the numerical analysis of the headrace tunnel and the wedge analysis.

Numerical Analysis of the Headrace Tunnel

Model Generation

A 2-D numerical analysis was carried out with the Phase 2 finite element programme. The proposed section of the headrace tunnel was modelled as per its excavated profile. The external fixed boundary was considered as three times its dimension in each direction. A full face excavation of the tunnel is proposed.

For generation of the model, an excavated boundary was created by importing a “DXF” File from AutoCAD. The external boundary was created by considering a box section with an expansion factor of 3 i.e. the external boundary was created at three times its excavated width on both sides, top and bottom. The model was analysed considering three stages of excavation and corresponding support systems.

A close- up view of 2-D Headrace Tunnel model is shown in the below figure.

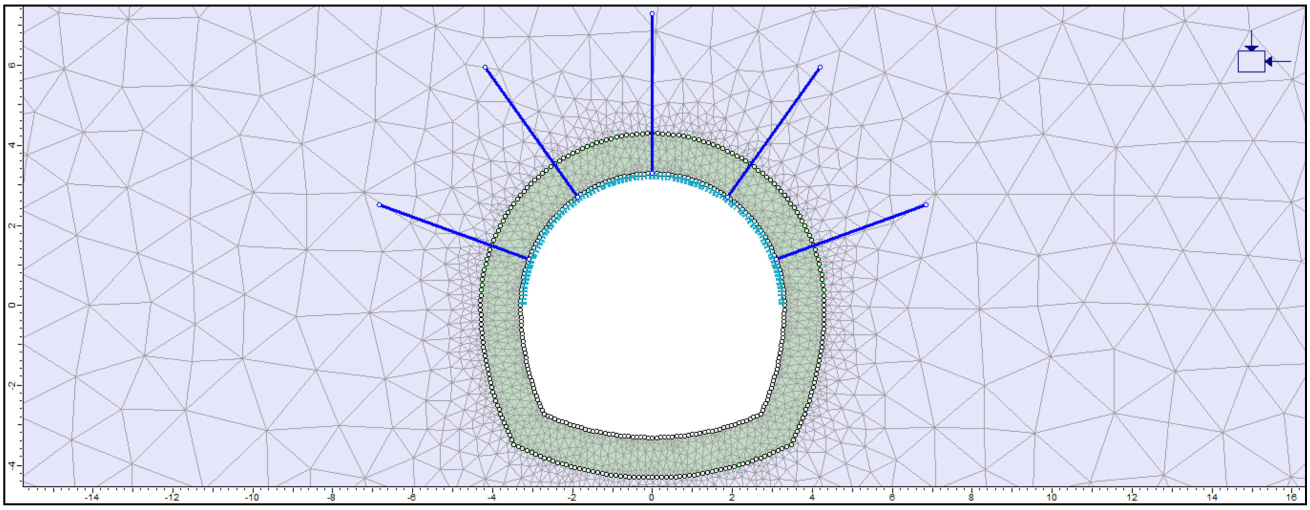

Figure 7-7: Close-up View of 2-D Model of Headrace Tunnel

The assumptions entering the model included the stages of excavation, the properties of the rock material, the loading details and the support system. These assumptions are presented in detail in Part A3 Chapter 2 of the report and are not reiterated here.

Results and Discussions

The type of analysis performed is plain strain analysis. Plain strain is defined as a state of strain in which the strain normal to the x-y plane (< z) and shear strain (< xz and < yz) are assumed to be zero.

Based on variations in rock type, rock class and rock cover (overburden cover), several analyses were performed and checked for the adequacy of different support classes. The condition which best fits the given support class was proposed. Based on these large number of trials and analyses, a graph has been plotted showing the required support class for a given combination of rock type, GSI values and rock/overburden cover. Based on the conversion of GSI to Q*, values like the chart for Q* were also prepared. The Q* used in this chart is a normalized Q which is the Q value without the SRF Parameter. The charts for gneiss and schist are given in Part A3 Chapter 2 of this report.

Based on the referred chart for the selection of the support class for any combination of rock type, GSI / Q* and rock cover / overburden depth appropriate chart for each rock type must be referenced and the GSI / Q* is marked on the X axis and rock cover / overburden depth on the Y axis and the support class will be the one on which zone this point lies. The extreme end of each support class is same as that marked in the legend i.e. the line on the left of each zone. For any point outside of the range of the extreme boundaries care must be applied in the selection of the support class. It is not expected for any point to be outside of this plot for the Headrace Tunnel.

Conclusion of the Rock Support System

Based on above results, it is concluded that shape of the model was as expected. As per this 2-D numerical analysis of the Headrace Tunnel, with the proposed support systems, the maximum deformation is about 4.66 mm for support Class II, 9.94 mm for support Class III, 46.6 mm for support class IV, 83.0 mm for support Class V and 246 mm for support class VI. The details are shown in the annexes to Part A3 Chapter 2 of this report**.**

The maximum limit of yielded elements from excavated face is about 3.0 m except for a few cases. The proposed 4.0m rock bolts reflect a safe design. The details are shown in the annexes to Part A3 Chapter 2 of this report**.**

The number of yielded bolts is very small and hence reflects a safe design. The details of the critically yielded rock bolts are shown in the annexes to Part A3 Chapter 2 of this report**.**

As per support capacity plots, only a limited portion of the shotcrete has a factor of safety less than 1.0 and only a limited portion of steel rib are less than factor of safety less than 1.5, which is acceptable. The shotcrete as designed is safe. The details of critical support capacity curves are shown in the annexes to Part A3 Chapter 2 of this report**.**

The recommended support systems based on above analysis is given below in the Subsection Results of Calculations.

Wedge Analysis of Headrace Tunnel

In this study, the stability of the excavated profile of the Headrace tunnel was checked against potential wedges that can develop based on identified structural discontinuities in the rock mass. The analysis for the Headrace Tunnel was carried out considering the excavation geometry. The wedge stability and the influence of the proposed support was evaluated by the Unwedge software of Rocscience, which is based on a limit equilibrium analysis program. The analysis was carried out before and after the application of the support systems.

The assumptions entering the model included the general input parameters, the joint properties and the support system. These assumptions are presented in detail in Part A3 Chapter 2 of the report and are not reiterated here.

Results of Wedge Analysis

There are slight differences in joint properties over the entire reach of the HRT. As per the geology, the maximum number of joints expected are four (including bedding joint). To be on the safe side, the maximum four of joints are considered for the wedge analysis for the entire length of the HRT. The summary of results for wedge stability analysis, before and after support installations, are presented below.

Table 7-15: Result of Unwedge Analysis

S.N.	Joint Combinations	Critical Failure Wedges	Wedge Weight (kN)	Excavated Face Area (m^2)	Factor of Safety (Without support)	Factor of Safety (With Support)
1	J2-J3-J4	Roof Wedge (8)	196.95	12.59	0.000	5.457
2	J1-J3-J4	Upper Left Wedge (7)	45.22	6.32	1.074	25.993
3	J1-J3-J4	Upper Right Wedge (8)	81.55	8.65	0.000	12.877

Conclusion of the wedge analysis

The analysis reveals the presence of 3 critical failure wedges. The details of the wedge formations are the same for trend angle towards upstream and trend angle towards downstream. The factor of safety (FOS) at various locations of the tunnel assumes greater importance in consideration of the Headrace Tunnel's. Any wedge failure in the crown portion, in particular failure of unconfined wedges around it, could create crown instability of catastrophic proportions. Hence, a very conservative approach to stabilization was adopted. Based on this analysis of known joint sets and assumed rock properties, 4.0 m rock bolts achieve the required FOS.

The calculated factor of safety, before and after installation of the supports, show that, with the applied supports, the potential wedges are safe. The critical wedges formed due to various joint combinations are shown in the annexes to Part A3 Chapter 2 of this report.

Design of Concrete Lining of Headrace Tunnel

A concrete lining is proposed for the entire periphery of the HRT, except initial 1.0 km length, where concrete lining is proposed in invert only. The concrete lining of the pressurized HRT is subjected to grout pressure during construction, maximum internal pressure during initial filling and maximum external pressure during dewatering / maintenance conditions.

The flow system is pressurized flow with FRL at EL 1158.0 m. The maximum water level in the Surge Tank is at EL 1179.3 m. Since the concrete lining is pervious during steady state conditions external and internal pressure will be the same. However, the concrete lining will be subjected to differential external and internal water pressure under different scenarios, which are explained below.

The objective and scope of work is the analysis and design of the concrete lining for Headrace Tunnel (HRT). The analysis was carried out with the Bentley STAAD Pro V8i software. The analysis and results are presented below.

The assumptions entering the model included the general input parameters, the material properties and the loads on the lining. These assumptions are presented in detail in Part A3 Chapter 2 of the report and are not reiterated here.

Based on the STAAD software analysis and design, the details of the reinforcement proposed for concrete lining of HRT are given below in the Section Results of Calculations.

7.1.8.4 Results of Calculations

The recommended support system for the Headrace Tunnel based on above analysis is given in the table below:

Table 7-16: Recommended Support System for Headrace Tunnel

S.N.	Rock Support Class	Proposed Support System
1	Class II	25 mm diameter fully grouted rock bolts @ 2 m c/c in crown only 50 mm thick plain shotcrete in Crown only
2	Class III	25 mm diameter fully grouted rock bolts @ 1.75 m c/c in crown only 50 mm thick plain shotcrete in crown and sides
3	Class IV	25 mm diameter fully grouted rock bolts @ 1.5 m c/c in crown and sides 100 mm thick plain shotcrete in crown and sides with one layer of welded wire mesh of 100x100x4 mm (=1.38cm^2/m)
4	Class V	25 mm diameter fully grouted rock bolts @ 1.25 m c/c in crown and sides 100 mm thick plain shotcrete in crown and sides with one layer of welded wire mesh of 100x100x4 mm Steel Rib of ISMB150 @ 1.0 m c/c
5	Class VI	25 mm diameter fully grouted rock bolts @ 1.0 m c/c in crown and sides 50 mm thick plain shotcrete in crown and sides Steel Rib of ISMB150 @ 0.5m c/c Secondary Concrete lining 400 mm thick with hoop reinforcement of 20 mm dia @ 200 mm c/c

In addition to the above support system, drainage holes of 50 mm diameter, 4.0 m length will be provided as per requirement. For initial 1.0 km length of Headrace Tunnel, modified horse shoe shaped tunnel of 7.5 m diameter (after shotcrete lining) is proposed. The support system for this portion of Headrace Tunnel is same as other portion of Headrace Tunnel.

The summary of reinforcement for the 40 cm thick Headrace Tunnel concrete liner are given the table below.

Table 7-17: Summary of Reinforcement

S.N.	Class of Rock	Concrete Lining Thickness (mm)	Main Reinforcement (Across Flow Direction)	Distribution Reinforcement (Along flow direction)
1	II, III & IV	400	No Reinforcement	No Reinforcement
2	V	400	16 mm dia bars @ 200 c/c on both faces (water face and rock face)	12 mm dia bars @ 200 mm c/c on both faces (water face and rock face)
3	VI	400	20 mm dia bars @ 200 mm c/c both faces (water face and rock face)	12 mm ϕ bars @ 200 mm c/c both faces (water face and rock face)

Headrace Tunnel in the Initial 1.0 km length is shotcrete lined with invert concrete. The excavated as well as finished size of Headrace Tunnel in this portion is increased. The finished size of Headrace Tunnel after shotcrete lining is 7.5 m diameter modified horse shoe shaped. The Invert lining thickness of maximum 400 mm at center, reducing to 200 mm at ends is proposed.

7.1.9 Power Waterways - Surge Tank

The design of rock support system in the surge tank requires a detailed analysis of the in situ rock stresses, rock strength parameters in conjunction with the sequence of cavern excavation and subsequent application of rock bolts and shotcrete for the rock support. The deformation modelling of the surge tank according to the in situ stresses were studied for the analysis and design of the rock support. The 2-D numerical analysis was carried out in a Phase 2 finite element program for deformation modelling. The analyses and the results have been presented in this report. Initially, the rock support system was estimated based on empirical relations.

7.1.9.1 Specific Design Criteria

Geology

The Surge Tank is composed of chlorite schist and phyllite. They are light gray to green-gray, having very closely to closely spaced foliation joints (spacing >20cm) and very weak to medium strong in strength. The foliation joints (JS0) & some other joints are filled with silty, clayey material up to 10mm of thickness. The rock mass is slightly to moderately weathered.

Three well-developed main joint sets including foliation joints (JS0, JS1 & JS2) were observed near the Surge Tank area. Among them, the foliation joints (JS0) are the prominent sets. The dip angles & dip directions of the said joints are as follows: JS0 (Foliation joints): 10-20/330-350, JS1: 55-70/065-085 & JS2: 70-80 / 155-170. The rock mass is slabby in general. The said rock bodies are well interlocked.

The rock mass is dry to damp in general. However, damp to wet groundwater condition can be anticipated during excavation.

Please refer to above Section on Connecting Tunnel.

7.1.9.2 Design Methods Applied

Please refer to above Section on Connecting Tunnel.

7.1.9.3 Design Calculations

The design calculations included the numerical analysis of the Surge Tank and the wedge analysis.

Numerical Analysis for Surge Tank

Model Generation

A 2-D numerical analysis was carried out in a Phase 2 finite element program. Since the Surge Tank is circular, two separate models were considered; A) an axisymmetric model and B) a circular excavated model. The model generated for each analysis is described below;

A) Axisymmetric Model

The centre line of the surge tank was considered as a rotational axis and an excavated half section of the surge tank was modelled. The external boundary was considered as three times its excavated diameter in all directions.

For the generation of the model, excavation stage boundaries were created by importing “DXF” files from AutoCAD. The external boundaries were also created by importing “DXF” files from AutoCAD. The model was analysed considering fifteen stages of excavation and the corresponding support systems. The material boundaries were also created by importing a “DXF” files from AutoCAD.

A close-up view of the axisymmetric 2-D Surge Tank Model is shown in the following figure.

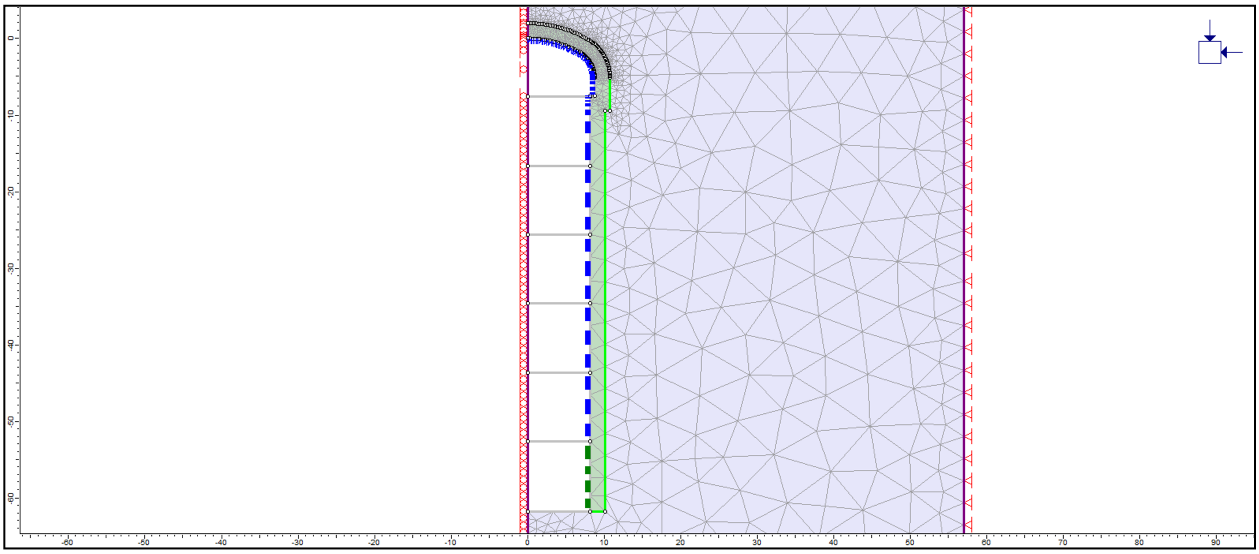

Figure 7-8: Close-up View of the Axisymmetric 2-D Model of the Surge Tank

The assumptions entering the model included the stages of excavation, the properties of the rock material, the loading details and the support system. These assumptions are presented in detail in Part A3 Chapter 2 of the report and are not reiterated here.

Results and Discussions

The results and discussions for the axisymmetric analysis and the plain strain analysis are given below:

A) Axisymmetric Analysis

The circular Surge Tank was modelled about its centreline as a axisymmetric model. Axisymmetric modelling allows for the analysis of a 3-D excavation, which is rotationally symmetric about an axis. The input provided is 2-dimensional, but the analysis results apply to a 3-dimensional problem. The following summary of various parameters determined after analysis are given below.

Table 7-18: Summary of Parameters after Axisymmetric Analysis

S.N.	Case	Condition	Maximum Displacement (mm)		
			On Wall	At Crown	At Bottom
1	Case-1	Fifteen Stage Excavation with Support	23.7	11.6	33.5

a) Total displacements

The maximum displacements were: about 23.7 mm on the side wall. At the crown, the displacement was about 11.6 mm and at the bottom the displacement was about 33.6 mm. Complete details of the model showing displacement is shown in the annexes to Part A3 Chapter 2 of this report.

b) Yielded Zone

The maximum radius of the plastic zone was about 10.5 m. Thus, the plastic zone is about 2.0 m beyond the excavation boundary. Complete details of the model showing yielded zone is shown in the annexes to Part A3 Chapter 2 of this report.

c) Support Capacity Plot of Shotcrete with Wire mesh

The support capacity plot of shotcrete and wire mesh is shown in the annexes to Part A3 Chapter 2 of this report.

B) Plain Strain Analysis

The circular excavated section of the Surge Tank was analysed in plain strain analysis. Plain strain is defined to be a state of strain in which the strain normal to the x-y plane (< z) and shear strain (< xz and < yz) are assumed to be zero. The following summary of the various parameters determined after the analysis are given below.

Table 7-19: Summary of Parameters after Plain Strain Analysis

S.N.	Case	Condition	Maximum Displacement Entire Periphery
1	Case-1	Three Stage Excavation with Support	50.7 mm

a) Total displacement

The maximum value of displacement in the entire periphery is about 50.7 mm. Complete details of the model showing displacement is shown in the annexes to Part A3 Chapter 2 of this report**.**

b) Yielded Zone

The maximum value of radius of plastic zone is about 12.0 m. Thus, the plastic zone is about 3.5 m beyond excavation boundary. Complete details of the model showing yielded zone is shown in the annexes to Part A3 Chapter 2 of this report**.**

c) Plot of Yielded Bolts

Due to deformation after application of the support system, some rock bolts yielded. The number of the yielded bolts are small and, hence, the design is safe. The details of the yielded bolts are shown in the annexes to Part A3 Chapter 2 of this report**.**

d) Support Capacity Plot of Shotcrete with wire-mesh

The support capacity plot of shotcrete and wire mesh is shown in the annexes to Part A3 Chapter 2 of this report**.**

<u>Conclusion</u>

Based on the above results, it can be concluded that the deformed shape of model was as expected. As per this 2-D numerical analysis of the Surge Tank (axisymmetric as well as plain stress analysis),, with the proposed support system, the maximum deformation is about 50.7 mm, which is about 0.3% of excavated diameter and hence within acceptable limits.

The maximum limit of yielded element from excavated face is about 3.5 m and rock bolts are proposed up to a length of 9 m, which reflects a safe design.

The number of yielded bolts is limited and hence safe.

As per support capacity plots, only limited portions of the shotcrete have a factor of safety less than 1.0, which is acceptable and the shotcrete as designed is safe.

The recommended support systems based on above analysis is given below in the Subsection Results of Calculations.

Wedge Analysis of Surge Tank

In this study the stability of the excavation profile of the underground Surge Tank was checked against potential wedges that can develop based on identified structural discontinuities in the rock mass. The analysis for the Surge Tank was carried out considering the excavation geometry. The wedge stability and the influence of the proposed support is evaluated by the Unwedge software from Rocscience, which is based on a limit equilibrium analysis program. The analysis was carried out before and after the application of support systems.

The assumptions entering the model included the general input parameters, the joint properties and the support system. These assumptions are presented in detail in Part A3 Chapter 2 of the report and are not reiterated here.

Conclusion of the Wedge Analysis

The analysis reveals the presence of only 2 critical failure wedges. The factor of safety (FOS) at various locations of the shaft assumes greater importance given the large (17m) excavated diameter of Surge Tank. Any wedge failure in the crown portion, in particular failure of unconfined wedges around it, could create crown instability of catastrophic proportion. Hence, a very conservative approach to stabilization was adopted. On the basis of this analysis based on known joint sets and assumed rock properties, 9m length of rock bolts @ 1m c/c achieves the required FOS.

The calculated factors of safety before and after installation of the support shows that with the applied supports, the potential wedges are safe. The critical wedges formed due to various joint and shear zone combinations are shown in the annexes to Part A3 Chapter 2 of this report.

Design of Concrete Lining for the Surge Tank

The Surge Tank positioned at the end of HRT to provide system stability and address surge defence requirements. A concrete lining is proposed for the entire periphery of the Surge Tank. The concrete lining is subjected to grout pressure during construction, maximum internal pressure during initial filling and maximum external pressure during dewatering / maintenance conditions.

The analysis has been carried out as per provisions of IS 7357-1974 and analysis and their results are discussed here.

The objective and scope of work is the analysis and design of the concrete lining for the Surge Tank. The analysis was carried out with the Bentley STAAD Pro V8i software. The analysis and results are presented below in the Subsection Results and Conclusions.

The assumptions entering the model included the general input parameters, the material properties and the loads on the lining. These assumptions are presented in detail in Part A3 Chapter 2 of the report and are not reiterated here.

Based on the analysis and design, the details of the proposed reinforcement for the concrete lining of the Surge Tank are given below in the Subsection Results of Calculations.

7.1.9.4 Results of Calculations

The recommended support system for the Surge Tank based on the above analysis is given in the table below:

Table 7-20: Recommended Support System for Surge Tank

S.N.	Class of Rock	Proposed Support System
1	Class IV & V	32 mm diameter, 9.0 m length fully grouted rock bolts @ 1.0 m c/c in crown as well as side walls 200 mm thick plain shotcrete with two layers of 100x100x4 mm wire mesh in Crown as well as side walls

Proposed reinforced concrete lining for the Surge Tank:

Table 7-21: Summary of Reinforcement and Concrete Lining Thickness

S.N.	EL (m)	Concrete Lining Thickness (mm)	Main Reinforcement (Hoop Direction)	Distribution Reinforcement (Longitudinal Reinforcement)
1	1123.9 m to 1140 m	900	32 mm dia bars @ 150 c/c in two rows	25 mm dia bars @ 200 c/c in two rows
2	1140 m to 1160 m	600	25 mm dia bars @ 130 c/c in two rows	20 mm dia bars @ 200 c/c in two rows
3	1160 m to 1180 m	500	20 mm dia bars @ 150 c/c in two rows	16 mm dia bars @ 200 c/c in two rows

7.1.10 Power Waterways - U/S Valve Chamber

The design of the rock support system in the Valve Chamber requires a detailed analysis of the in situ rock stresses, rock strength parameters in conjunction with the stages of the cavern excavation and subsequent application of rock bolts and shotcrete for rock support. The deformation modelling of the Valve Chamber subjected to in situ stresses have been analyzed and a rock support system designed. Initially, the rock support system was estimated based on empirical relations. Subsequently, a numerical analysis was performed. The 2-D numerical analysis was carried out in a Phase 2 finite element program for deformation modelling. The analyses and their results are presented below in this report.

7.1.10.1 Specific Design Criteria

Geology of the Valve Chamber

The Valve Chamber is proposed in chlorite schist and phyllite rock. They are light gray to green-gray, have very closely to closely spaced foliation joints (spacing >20cm) and very weak to medium strong in strength. The foliation joints (JS0) Other joints are filled with silty, clayey material up to 10mm of thickness. The rock mass is slightly to moderately weathered.

Three well-developed main joint sets including foliation joints (JS0, JS1 & JS2) were observed near the Valve Chamber area. Among them, the foliation joints (JS0) are the prominent sets. The dip angles & dip directions of the said joints are as follows: JS0 (Foliation joints): 10-20/330-350, JS1: 55-70/065-085 & JS2: 70-80 / 155-170. The rock mass is slabby in general. The said rock bodies are well interlocked.

The rock mass is dry to damp in general. However, damp to wet groundwater condition can be anticipated during excavation.

For design loads, load combinations, geotechnical data and material properties please refer to 2.1.1.1.

7.1.10.2 Design Methods Applied

Please refer to above Section on Connecting Tunnel.

7.1.10.3 Design Calculations

The design calculations included the numerical analysis of the Valve Chamber and the wedge analysis.

Numerical Analysis for Valve Chamber

Model Generation

The 2-D numerical analysis was carried out in a Phase 2 finite element program. The proposed section of the Valve Chamber was modelled as per its excavated profile. The external fixed boundary was considered as three times its dimension in each direction.

For generation of the model, an excavated boundary was created by importing a “DXF” file from AutoCAD. The external boundary was created by considering a box section with an expansion factor of 3 i.e. external boundary was created at three times its excavated width, both sides, top and bottom. The model was analysed considering fourteen stages of excavation and support systems. The stage boundary was also created by importing a “DXF” file from AutoCAD.

A close- up view of the 2-D Valve Chamber model is shown in the following figure.

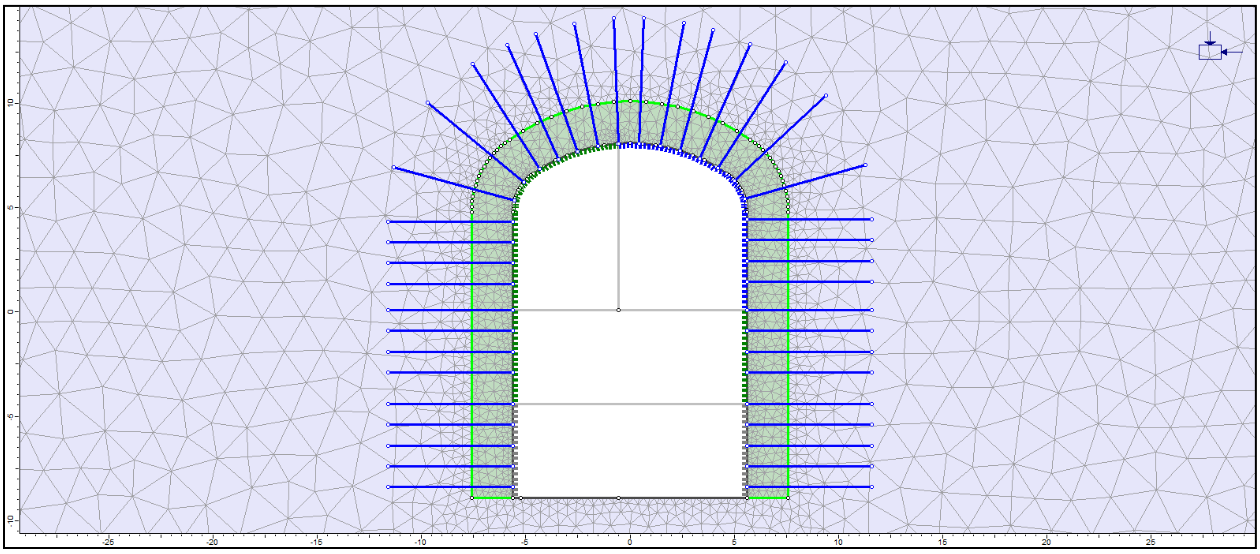

Figure 7-9: Close-up View of 2D Model of Valve Chamber

The assumptions entering the model included the stages of excavation, the properties of the rock material, the loading details and the support system. These assumptions are presented in detail in Part A3 Chapter 2 of the report and are not reiterated here.

Results and Discussions

The type of analysis performed is plain strain analysis. Plain strain is defined as a state of strain in which the strain normal to the x-y plane (< z) and shear strain (< xz and < yz) are assumed to be zero. The following summary of various parameters determined after analysis are given below.

Table 7-22: Summary of Parameters after Numerical Analysis

S.N.	Case	Condition	Maximum Displacement (mm)			
			On Left Wall	On Right Wall	At Crown	At Bottom
1	Case-1	Nine Stage Excavation with Support	94.8	100.8	35.3	72.8

A) Total displacement

The maximum value of displacement is about 94.8 mm on left side wall and 100.8 mm on right side wall of Valve Chamber. At crown, the displacement is about 35.3 mm and at the bottom, displacement is about 72.8 mm. Complete details of the model showing displacement is shown in the annexes to Part A3 Chapter 2 of this report.

B) Yielded Zone

The maximum radius of the plastic zone is about 10 m. Thus, the plastic zone is about 4.0 m beyond excavation boundary. Complete details of the model showing the yielded zone is shown in the annexes to Part A3 Chapter 2 of this report**.**

C) Plot of Yielded Bolts

Due to deformation after application of the support system, some rock bolts have yielded. The details of yielded bolts are limited and hence safe. The details of the yielded bolts are shown in the annexes to Part A3 Chapter 2 of this report**.**

D) Support Capacity Plot of Shotcrete with Wire-mesh

The support capacity plot of the shotcrete and wire mesh is shown in the annexes to Part A3 Chapter 2 of this report**.**

<u>Conclusion</u>

Based on above results, it can be concluded that deformation shape of the model is as expected. As per this 2-D numerical analysis of the Valve Chamber, with the proposed support system the maximum deformation is about 100.8 mm, which is about 0.88% of excavated width and hence within the allowable limit.

The maximum limit of the yielded elements from excavated face is about 4.0 m and the proposed rock bolts are up to a length of 6 m, which is safe.

The number of yielded bolts was limited and hence safe.

As per support capacity plots, only a limited portion of the shotcrete has a factor of safety less than 1.0, which is acceptable and the shotcrete as designed is safe.

The recommended support system based on above analysis is given below in the Subsection Results and Conclusions.

<u>Wedge Analysis of Valve Chamber</u>

In this study the stability of the excavation profile of the underground Valve Chamber was checked against potential wedges that can develop based on identified structural discontinuities in the rock mass. The analysis for the Valve Chamber was carried out considering the excavation geometry. The wedge stability and the influence of the proposed support was evaluated by Unwedge software of Rocscience, which is based on a limit equilibrium analysis program. The analysis was carried out before and after the application of a support system.

The assumptions entering the model included the general input parameters, the joint properties and the support system. These assumptions are presented in detail in Part A3 Chapter 2 of the report and are not reiterated here.

Results of Wedge Analysis

Based on combinations of all joints, the summary of results for wedge stability analysis before and after support installation is presented below.

Table 7-23: Summary of Results for Wedge Analysis

S.N.	Joint Combinations	Critical Failure Wedges	Wedge Weight (kN)	Excavated Face Area (m^2)	Factor of Safety (Without support)	Factor of Safety (With Support)
1	J1-J2-J3	Upper Right Wedge (5)	6768	113.97	0.69	9.597
2	J1-J2-J3	Roof Wedge (8)	49.23	6.51	0	52.708

Conclusion of Wedge Analysis

The analysis reveals presence of only 2 critical failure wedges. The factor of safety (FOS) at various locations of the cavern assumes greater importance in consideration of height of Valve Chamber of about 17 m. Any wedge failure in the crown portion, in particular failure of unconfined wedges around it, could create crown instability of catastrophic proportion. Hence, a very conservative approach to stabilization was adopted. On the basis of this analysis, based on known joint sets and assumed rock properties, 6m length of rock bolts @ 1m c/c achieve the required FOS.

The calculated factor of safety before and after installation of support shows that with the applied support, the potential wedges are safe. The critical wedges formed due to various joint and shear zone combinations are shown in the annexes to Part A3 Chapter 2 of this report.

7.1.10.4 Results of Calculation

The recommended support system for the **U/S Valve Chamber**, based on above analysis, is given below:

Table 7-24: Support System for U/S Valve Chamber

S.N.	Class of Rock	Proposed Support System
1	Class IV & V	32 mm diameter, 6.0 m length fully grouted rock bolts @ 1.0 m c/c in crown as well as side walls, 200 mm thick plain shotcrete with two layers of wire mesh in crown as well as side walls

In addition to the above support system, drainage holes of 75 mm diameter, 6.0 m length will be provided as per requirement.

7.1.11 Power Waterways - Pressure Shaft

The design of rock support system for the Pressure Shaft requires a detailed analysis of the in situ rock stresses, rock strength parameters and application of rock bolts and shotcrete for rock support. The deformation modelling of the Pressure Shaft subjected to in situ stresses has been studied for the analysis and design of the rock support. The 2-D numerical analysis has been carried out with the Phase 2 finite element program for deformation and rock support modelling. The analyses and their results are discussed below in this report. Initially, the rock support system was estimated based on two empirical methods. Subsequently, a numerical analysis was performed The and details are presented below in this report.

7.1.11.1 Specific Design Criteria

Geology of the Pressure Shaft

The Pressure Shaft over its the entire reach is composed of Augen gneiss rock with partings of chlorite schist. The rock mass has a gneissic texture. It contains quartz, feldspar, muscovite & biotite including phenocrysts of quartz and feldspar.

Augen gneiss rock is coarse to very coarse grained, has very closely to moderately spaced foliation joints (spacing >6cm to 60cm) and medium strong to strong in strength. The foliation joints (JS0) & some other joints are filled with sandy, silty and sometimes clayey material up to 5mm of thickness. The rock mass is slightly to moderately weathered.

Four well-developed main joint sets including foliation joints (JS0, JS1, JS2 & JS3) were observed near the Pressure Shaft area. Among them, the foliation joints (JS0) are the prominent sets. The dip angles & dip directions of the said joints are as follows: JS0 (Foliation joints): 05-15/330-350, JS1: 75-85 / 010-030, JS2: 65-75/065-085 & JS3: 70-85 / 170-210. The rock mass is blocky and slabby. The said rock bodies are well interlocked.

The rock mass is dry to damp in general. However, damp to dripping groundwater condition can be anticipated during excavation.

For design loads, load combinations, geotechnical data and material properties please refer to 2.1.1.1.

7.1.11.2 Design Methods Applied

Please refer to above Section on Connecting Tunnel.

7.1.11.3 Design Calculations

The design calculations included the numerical analysis of the pressure shaft and the wedge analysis.

<u>Numerical Analysis for Pressure Shaft</u>

<u>Model Generation</u>

A 2-D numerical analysis was carried out with the Phase 2 finite element program. The proposed cross section of the Pressure Shaft is modelled as per its excavated profile. The external fixed boundary was considered as three times its dimension in each direction.

For generation of the model, the excavated boundary was created by importing a “DXF” file from AutoCAD. The external boundary was created by considering a box section with an expansion factor of 3 i.e. the external boundary dimensions were chosen to be three times its excavated width on both sides, top and bottom. The model was analysed considering three stages of excavation and the corresponding support systems.

A close- up view of the 2-D Pressure Shaft model is shown in the figure below.

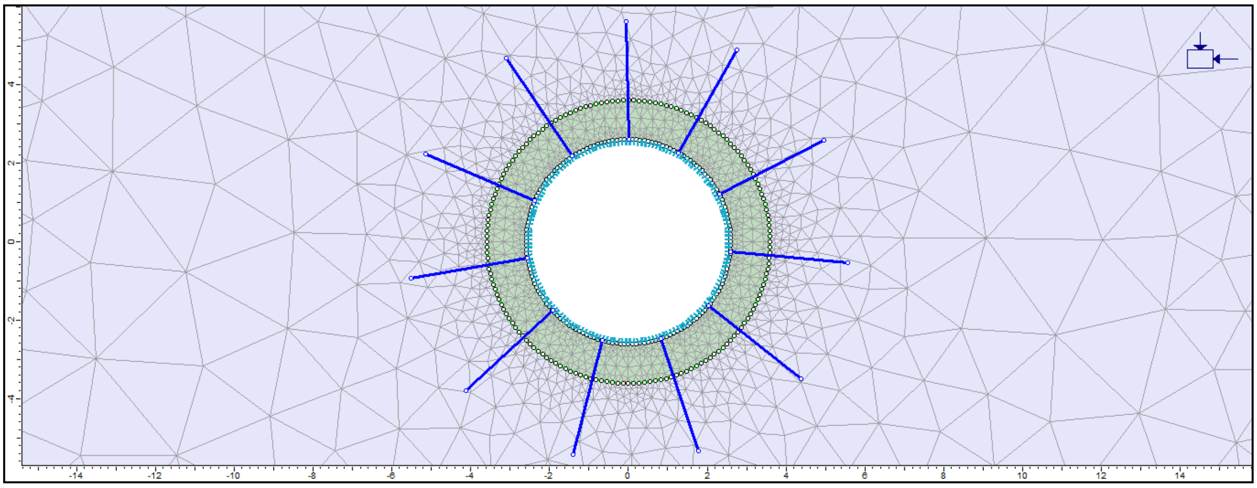

Figure 7-10: Close-up View of Pressure Shaft Model

The assumptions entering the model included the stages of excavation, the properties of the rock material, the loading details and the support system. These assumptions are presented in detail in Part A3 Chapter 2 of the report and are not reiterated here.

Results and Discussions

The type of analysis performed is plain strain analysis. Plain strain is defined as a state of strain in which the strain normal to the x-y plane (< z) and shear strain (< xz and < yz) are assumed to be zero.

Conclusion of Numerical Analysis

Based on above results, it is concluded that the deformed shape of the model was as expected. As per this 2-D numerical analysis of the Pressure Shaft, with the proposed support system, the maximum deformation was about 18.1 mm. The details are shown in the annexes to Part A3 Chapter 2 of this report**.**

The maximum limit of the yielded elements from the excavated face was about 1.0 m and the rock bolts are proposed up to a length of 3.0 m, which is safe. The details are shown in the annexes to Part A3 Chapter 2 of this report**.**

No yielded bolts were generated in this model and hence the design is safe. The details are shown in the annexes to Part A3 Chapter 2 of this report.

As per support capacity plots, none of the shotcrete elements had a factor of safety less than 1.0, which again reflects a safe design. The details are shown in the annexes to Part A3 Chapter 2 of this report.

The recommended support system based on above analysis is given below in the Subsection Results and Conclusions.

Wedge Analysis of Pressure Shaft

In this study, the stability of excavation profile of the underground Pressure Shaft is checked against potential wedges that can develop based on identified structural discontinuities in the rock mass. The analysis for the Pressure Shaft was carried out considering the excavation geometry. The wedge stability and the influence of the proposed support was evaluated by Unwedge software of Rocscience, which is based on a limit equilibrium analysis program. The analysis was carried out before and after application of the support system.

The assumptions entering the model included the general input parameters, the joint properties and the support system. These assumptions are presented in detail in Part A3 Chapter 2 of the report and are not reiterated here.

Results of Wedge Analysis

Based on combinations of all joints, the summary of results for wedge stability analysis, before and after support installation, is presented below.

Table 7-25: Summary of Results for Wedge Analysis

S.N.	Joint Combinations	Critical Failure Wedges	Wedge Weight (kN)	Excavated Face Area (m^2)	Factor of Safety (Without support)	Factor of Safety (With Support)
1	J2-J3-J4	West Wedge (6)	293.89	11.19	1.404	17.657

Conclusion of the Wedge Analysis

The analysis reveals the presence of only 1 critical failure wedge. The factor of safety (FOS) at various locations of the tunnel assumes greater importance given the large diameter (about 5.2 m) of the Pressure Shaft. Hence, a very conservative approach to stabilization was adopted. Based on this analysis which is based on known joint sets and assumed rock properties, 3m length of rock bolts @ 1.5m c/c achieve the required FOS.

The calculated factor of safety before and after installation of the support shows that with the applied support, the potential wedges are safe. The critical wedges formed due to various joint and shear zone combinations are shown in the annexes to Part A3 Chapter 2 of this report.

7.1.11.4 Results of Calculation

The recommended support system for the **Pressure Shaft** based on above analysis is given below.

Table 7-26: Support System for Pressure Shaft

S.N.	Class of Rock	Proposed Support System
1	Class V	25 mm diameter, 3 m length fully grouted rock bolts @ 1.5 m c/c over the entire periphery 100 mm thick plain shotcrete over entire periphery with one layer of welded wire mesh of 100x100x4 mm

In addition to the above support system, drainage holes of 50 mm diameter and 4.0 m length will be provided as per requirement.

7.1.12 Power Waterways - High Pressure Tunnel & Upstream Manifolds

The design of the rock support system for the High Pressure Tunnel & Upstream Manifold requires a detailed analysis of the in situ rock stresses, rock strength parameters, application of rock bolts and shotcrete for rock support. The deformation modelling of the High Pressure Tunnel & Upstream Manifold subjected to in situ stresses has been studied for the analysis and design of the rock support. The 2-D numerical analysis was carried out in a Phase2 finite element program for deformation and rock support modelling. The analyses and their results have been discussed in this report. Initially, the rock support system was estimated based on empirical relations and details presented in this report.

7.1.12.1 Specific Design Criteria

Geology of the Horizontal High Pressure Tunnel

The High Pressure Tunnel & Upstream Manifold in its entire reach is composed of Augen gneiss with partings of chlorite schist. The rock mass has a gneissic texture. It contains quartz, feldspar, muscovite & biotite, including phenocrysts of quartz and feldspar.

Augen gneiss is coarse to very coarse grained, having very closely to moderately spaced foliation joints (spacing >6cm to 60cm) and medium strong to strong in strength. The foliation joints (JS0) & some other joints are filled with sandy, silty and sometimes clayey material up to 5mm of thickness. The rock mass is slightly to moderately weathered.

Four well-developed main joint sets including foliation joints (JS0, JS1, JS2 & JS3) were observed near the High Pressure Tunnel & Upstream Manifold area. Among them, the foliation joints (JS0) are the prominent sets. The dip angles & dip directions of the said joints are as follows: JS0 (Foliation joints): 05-15/330-350, JS1: 75-85 / 010-030, JS2: 65-75/065-085 & JS3: 70-85 / 170-210. The rock mass is blocky and slabby. The said rock bodies are well interlocked.

The rock mass is dry to damp in general. However, damp to dripping groundwater condition can be anticipated during excavation.

For design loads, load combinations, geotechnical data and material properties please refer to **2.1.1.1.**

7.1.12.2 Design Methods Applied

Please refer to above Section on Connecting Tunnel.

7.1.12.3 Design Calculations

The design calculations included the numerical analysis of the tunnels & manifolds and the wedge analysis.

Numerical Analysis for Horizontal High Pressure Tunnel

Model Generation

The 2-D numerical Analysis was carried out in a Phase 2 finite element program. The proposed section of High Pressure Tunnel & Upstream Manifolds is modelled as per its excavated profile. The external fixed boundary was considered as three times its dimension in each direction.

For generation of the model, an excavated boundary was created by Importing a "DXF" file from AutoCAD. The external boundary was created by considering a box section with an expansion factor of 3 i.e. an external boundary was created as three times the excavated width on both sides and top and bottom. The model was analysed considering three stages of excavation and corresponding support system.

A close- up view of 2-D High Pressure Tunnel & Upstream Manifold model is shown in the figure below.

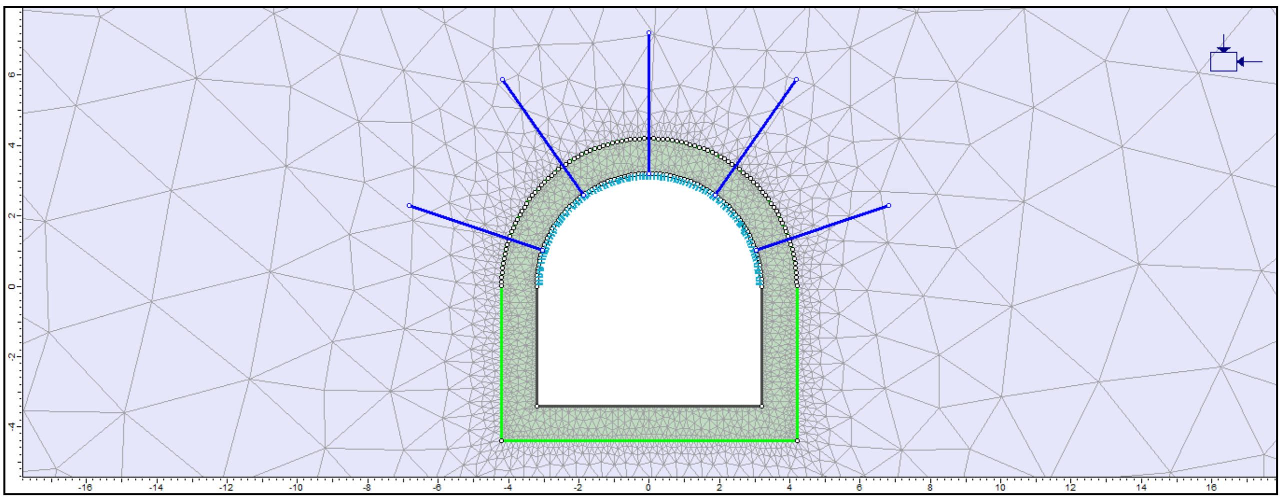

Figure 7-11: Close-up View of Horizontal Pressure Tunnel & Manifold Model

The assumptions entering the model included the stages of excavation, the properties of the rock material, the loading details and the support system. These assumptions are presented in detail in Part A3 Chapter 2 of the report and are not reiterated here.

Results and Discussions

The type of analysis performed is Plain Strain Analysis. Plain strain is defined to be a state of strain in which the strain normal to the x-y plane (< z) and shear strain (< xz and < yz) are assumed to be zero.

Conclusion

Based on above results, it was concluded that the deformed shape of model was as expected. As per this 2-D numerical analysis of Horizontal High Pressure Tunnel, with the proposed support system, the maximum deformations are about 4 mm for Class II rock, 7.8 mm for Class III rock, 16.4 mm for class IV rock and 35.5 mm for Class V rock, which are within the allowable limit. The details are shown in the annexes to Part A3 Chapter 2 of this report**.**

The maximum limit of yielded elements from excavated face is about 1 m and the proposed rock bolts are up to a length of 4.0 m, which is safe. The details are shown in the annexes to Part A3 Chapter 2 of this report**.**

The number of yielded bolts is very limited and hence safe. The details are shown in the annexes to Part A3 Chapter 2 of this report**.**

As per support capacity plots, only limited portions of the shotcrete have a factor of safety less than 1.0 and none of the steel rib elements has a factor of safety less than 1.5, which is safe. The details are shown in the annexes to Part A3 Chapter 2 of this report**.**

The recommended support system based on above analysis is given below in the following Subsection Results of Calculation.

Wedge Analysis of Horizontal High Pressure Tunnel

In this study the stability of the excavation profile of the High Pressure Tunnel & Upstream Manifolds was checked against potential wedges that can develop based on identified structural discontinuities in the rock mass. The analysis for High Pressure Tunnel & Upstream Manifold was carried out considering the excavated geometry. The wedge stability and the influence of the proposed support was evaluated with the Unwedge software from Rocscience, which is based on a limit equilibrium analysis program. The analysis was carried out before and after the application of the support system.

The assumptions entering the model included the general input parameters, the joint properties and the support system. These assumptions are presented in detail in Part A3 Chapter 2 of the report and are not reiterated here.

Results of Wedge Analysis

Based on combinations of all joints, the summary of results for wedge stability analysis, before and after support installation, is presented below.

Table 7-27: Summary of Results for Wedge Analysis

S.N.	Joint Combinations	Critical Failure Wedges	Wedge Weight (kN)	Excavated Face Area (m^2)	Factor of Safety (Without support)	Factor of Safety (With Support)
1	J1-J3-J4	Upper Right Wedge (5)	386.12	16.54	1.281	6.290
2	J1-J3-J4	Roof Wedge (8)	9.58	2.04	0.000	44.204
3	J1-J2-J3	Roof Wedge (6)	21.77	3.03	0.746	17.971
4	J1-J2-J3	Upper Right Wedge (7)	153.59	10.05	1.327	12.154
5	J2-J3-J4	Upper Right Wedge (6)	26.86	7.52	1.383	6.686
6	J2-J3-J4	Upper Left Wedge (8)	0.19	0.12	0.000	430.526
7	J1-J2-J4	Roof Wedge (8)	0.03	0.04	0.000	1380

Conclusion of Wedge Analysis

The analysis reveals the presence of only 7 critical failure wedges. The factor of safety (FOS) at various locations of the cavern assumes greater importance in consideration of size of High Pressure Tunnel & Upstream Manifold of about 6.6 m. Any wedge failure in the crown portion, in particular failure of unconfined wedges around it, could create crown instability of catastrophic proportion. Hence, a very conservative approach to stabilization was adopted. On the basis of this analysis and based on known joint sets and assumed rock properties, 6m length of rock bolts @ 1.25 m c/c achieve the required FOS.

The observed Factor of Safety before and after installation of support shows that, with the applied support, the potential wedges are safe. The critical wedges formed due to various joint and shear zone combinations are shown in the annexes to Part A3 Chapter 2 of this report.

7.1.12.4 Results of Calculations

The recommended support system for the High Pressure Tunnel & Upstream Manifold based on the above analysis is given below:

Table 7-28: Recommended Support System

S.N.	Class of Rock	Proposed Support System
1	Class II	25 mm diameter, 4 m length fully grouted rock bolts @ 2 m c/c in crown only 50 mm thick plain shotcrete in crown only
2	Class III	25 mm diameter, 4 m length fully grouted rock bolts @ 1.75 m c/c in crown only 50 mm thick plain shotcrete in crown and sides
3	Class IV	25 mm diameter, 4 m length fully grouted rock bolts @ 1.5 m c/c in crown and sides 100 mm thick plain shotcrete in crown and sides with one layer of welded wire mesh of 100x100x4 mm
4	Class V	25 mm diameter, 4 m length fully grouted rock bolts @ 1.25 m c/c in entire periphery 100 mm thick plain shotcrete over the entire periphery with one layer of welded wire mesh of 100x100x4 mm Steel Ribs of ISHB150 @ 0.75 m c/c

In addition to the above support system, drainage holes of 50 mm diameter, and 4.0 m length will be provided as per requirement.

7.1.13 Power Waterways - Tailrace Tunnel and D/S Manifold

The design of the rock support system in the Tailrace Tunnel requires a detailed analysis of the in situ rock stresses, rock strength parameters in conjunction with the stages of the cavern excavation and subsequent application of rock bolts and shotcrete for rock support. The deformation modelling of the Tailrace Tunnel subjected to in situ stresses have been analyzed and a rock support system designed. Initially, the rock support system was estimated based on empirical relations. Subsequently, a numerical analysis was performed. The 2-D numerical analysis was carried out in a Phase 2 finite element program for deformation modelling. The analyses and their results are presented below in this report.

The design and analysis of the concrete lining for Tailrace Tunnel is also presented.

7.1.13.1 Specific Design Criteria

Geology of the Tailrace Tunnel

The Tailrace Tunnel in the entire reach is composed of Augen gneiss with partings of chlorite schist. The rock mass has a gneissic texture. It contains quartz, feldspar, muscovite & biotite including phenocrysts of quartz and feldspar.

Augen gneiss is coarse to very coarse grained, having very closely to moderately spaced foliation joints (spacing >6cm to 60cm) and medium strong to strong in strength. The foliation joints (JS0) & some other joints are filled with sandy, silty and sometimes clayey material up to 5mm of thickness. The rock mass is slightly to moderately weathered.

Four well-developed main joint sets including foliation joints (JS0, JS1, JS2 & JS3) were observed near the Adit 4 area. Among them, the foliation joints (JS0) are the prominent sets. The dip angles & dip directions of the said joints are as follows: JS0 (Foliation joints): 05-15/330-350, JS1: 75-85 / 010-030, JS2: 65-75/065-085 & JS3: 70-85 / 170-210. The rock mass is blocky and slabby. The said rock bodies are well interlocked.

The rock mass is dry to damp in general. However, damp to dripping groundwater condition can be anticipated during excavation.

For design loads, load combinations, geotechnical data and material properties please refer to **2.1.1.1.**

7.1.13.2 Design Methods Applied

Please refer to above Section on Connecting Tunnel.

7.1.13.3 Design Calculations

The design calculations included the numerical analysis of the waterways and the wedge analysis.

Numerical Analysis of Tailrace Tunnel

Model Generation

The 2-D numerical analysis was carried out in a Phase 2 finite element program. The proposed section of the Valve Chamber was modelled as per its excavated profile. The external fixed boundary was considered as three times its dimension in each direction.

For generation of the model, an excavated boundary was created by importing a “DXF” file from AutoCAD. The external boundary was created by considering a box section with an expansion factor of 3 i.e. external boundary was created at three times its excavated width, both sides, top and bottom. The model was analysed considering fourteen stages of excavation and support systems. The stage boundary was also created by importing a “DXF” file from AutoCAD.

A close- up view of the 2-D Tailrace Tunnel Model is shown in the following figure.

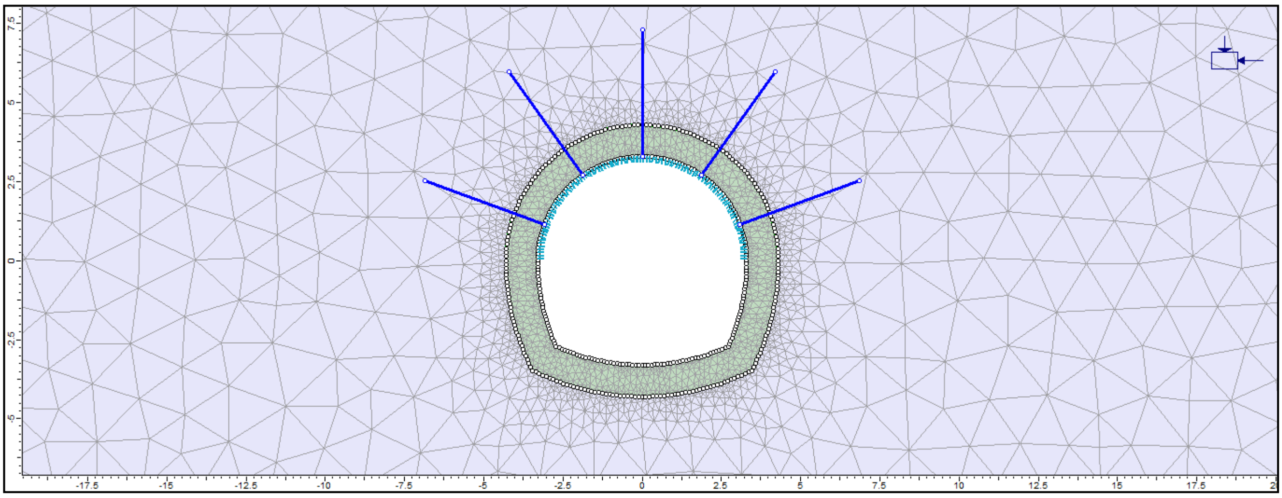

Figure 7-12: Close-up View of 2-D Model of Tailrace Tunnel

The assumptions entering the model included the stages of excavation, the properties of the rock material, the loading details and the support system. These assumptions are presented in detail in Part A3 Chapter 2 of the report and are not reiterated here.

<u>Results and Discussions</u>

The type of analysis performed was plain strain analysis. Plain strain is defined as a state of strain in which the strain normal to the x-y plane (< z) and shear strain (< xz and < yz) are assumed to be zero. The details of the various assumed parameters and the results of the analysis are given below for each rock class.

<u>Conclusion</u>

Based on the above results, it was concluded that the deformed shape of the model was as expected. As per this 2-D numerical analysis of the Tailrace Tunnel, with the proposed support system, the maximum deformation is about 3 mm for Class II rock, 5.7 mm for Class III rock, 11.6 mm for class IV rock and 23.3 mm for Class V rock, which is within the allowable limit. The details are shown in the annexes to Part A3 Chapter 2 of this report**.**

The maximum limit of the yielded elements from the excavated face was about 1.0 m. The rock bolts are proposed up to a length of 4.0 m, which is safe. The details are shown in the annexes to Part A3 Chapter 2 of this report**.**

There were no yielded bolts and hence the design is safe. The details are shown in the annexes to Part A3 Chapter 2 of this report**.**

As per support capacity plots, none of the shotcrete element had a factor of safety less than 1.0 and none of the steel rib elements had a factor of safety less than 1.5, hence the design is safe. The details are shown in the annexes to Part A3 Chapter 2 of this report**.**

The recommended support system based on above analysis is given below in the Subsection Results of Calculations.

Wedge Analysis for Tailrace Tunnel

In this study, the stability of the excavation profile of the underground Tailrace Tunnel was checked against potential wedges that can develop based on identified structural discontinuities in the rock mass. The analysis for the Tailrace Tunnel was carried out considering the excavation geometry. The wedge stability and the influence of the proposed support was evaluated by the Unwedge software from Rocscience, which is based on a limit equilibrium analysis program. The analysis was carried out before and after the application of the support system.

The assumptions entering the model included the general input parameters, the joint properties and the support system. These assumptions are presented in detail in Part A3 Chapter 2 of the report and are not reiterated here.

Results of Wedge Analysis

Based on combinations of all joints, the summary of results for wedge stability analysis before and after support installation is presented below.

Table 7-29: Summary of Results for Wedge Analysis

S.N.	Joint Combinations	Critical Failure Wedges	Wedge Weight (kN)	Excavated Face Area (m^2)	Factor of Safety (Without support)	Factor of Safety (With Support)
1	J2-J3-J4	Upper Left Wedge (8)	0.12	0.09	0.000	654
2	J1-J3-J4	Upper Right Wedge (5)	243.9	12.31	1.316	8.08
3	J1-J3-J4	Roof Wedge (8)	10.29	2.23	0.000	44.953
4	J1-J2-J3	Roof Wedge (6)	30.82	3.65	0.805	20.847
5	J1-J2-J4	Roof Wedge (8)	0.08	0.10	0.000	927

Conclusion of the Wedge Analysis

The analysis reveals the presence of 5 critical failure wedges. The factor of safety (FOS) at various locations of the cavern assumes greater importance in consideration of the size of the Tailrace Tunnel. Any wedge failure in the crown portion, in particular failure of unconfined wedges around it, could create crown instability of catastrophic proportions. Hence, a very conservative approach to stabilization was adopted. Based on this conservative approach and based on known joint sets and assumed rock properties, 4.0 m rock bolts achieve the required FOS.

The calculated factor of safety, before and after the installation of the support, shows that with the applied support, the potential wedges are safe. The critical wedges formed due to various joint combinations are shown in the annexes to Part A3 Chapter 2 of this report.

Design of Concrete Lining of Tailrace Tunnel

The objective and scope of the work is to analysis and design the concrete lining for Tailrace Tunnel. The concrete lining of the pressurized Tailrace Tunnel is subjected to grout pressure during construction, maximum internal pressure during initial filling and maximum external pressure during dewatering / maintenance conditions. The analysis was carried out with Bentley STAAD Pro V8i software.

The assumptions entering the model included the general input parameters, the material properties and the loads on the lining. These assumptions are presented in detail in Part A3 Chapter 2 of the report and are not reiterated here.

Based on STAAD analysis and design, the details of reinforcement proposed for concrete lining of Tailrace Tunnel are given below in the Subsection Results of Calculations.

7.1.13.4 Results of Calculations

The recommended support system for the Tailrace Tunnel based on the above analysis is given in the table below:

Table 7-30: Recommended Support System for Tailrace Tunnel

S.N.	Class of Rock	Proposed Support System
1	Class II	25 mm diameter fully grouted rock bolts @ 2 m c/c in crown only 50 mm thick plain shotcrete in Crown only
2	Class III	25 mm diameter fully grouted rock bolts @ 1.75 m c/c in crown only 50 mm thick plain shotcrete in crown and sides
3	Class IV	25 mm diameter fully grouted rock bolts @ 1.5 m c/c in crown and sides 100 mm thick plain shotcrete in crown and sides with one layer of welded wire mesh of 100x100x4 mm
4	Class V	25 mm diameter fully grouted rock bolts @ 1.25 m c/c in crown and sides 50 mm thick plain shotcrete in Crown and sides Steel Ribs of ISMB150 @ 1 m c/c

In addition to the above support system, drainage holes of 50 mm diameter, 4.0 m length will be provided as per requirement.

Table 7-31: Summary of Reinforcement for Concrete Lining

S.N.	Class of Rock	Concrete Lining Thickness (mm)	Main Reinforcement (Across Flow Direction)	Distribution Reinforcement (Along flow direction)
1	III to V	400	No Reinforcement	No Reinforcement

7.1.14 Outlet Structure - Shaft Structure

The design of the rock support system for the Outlet Structure Shaft requires a detailed analysis of the in situ rock stresses, rock strength parameters and application of rock bolts and shotcrete for rock support. The deformation modelling of the Outlet Structure Shaft subjected to in-situ stresses was studied for the analysis and design of the rock support. The 2-D numerical analysis has been carried out in a Phase2 finite element program for deformation and rock support modelling. The analyses and their results have been discussed in this report. The initial, rock support system was estimated based on empirical relations. The details are presented below in this report.

7.1.14.1 Specific Design Criteria

Geology of the Outlet Structure Shaft

The Outlet Structure Shaft in its entirety is composed of Augen gneiss with partings of chlorite schist. The rock mass has a gneissic texture. It contains quartz, feldspar, muscovite & biotite including phenocrysts of quartz and feldspar.

Augen gneiss is coarse to very coarse grained, having very closely to moderately spaced foliation joints (spacing >6cm to 60cm) and medium strong to strong in strength. The foliation joints (JS0) & some other joints are filled with sandy, silty and sometimes clayey material up to 5mm of thickness. The rock mass is slightly to moderately weathered.

Four well-developed main joint sets, including foliation joints (JS0, JS1, JS2 & JS3), were observed near the Adit 4 area. Among them, the foliation joints (JS0) are the prominent sets. The dip angles & dip directions of the said joints are as follows: JS0 (Foliation joints): 05-15/330-350, JS1: 75-85 / 010-030, JS2: 65-75/065-085 & JS3: 70-85 / 170-210. The rock mass is blocky and slabby. The said rock bodies are well interlocked.

The rock mass is dry to damp in general. However, damp to dripping groundwater condition can be anticipated during excavation.

For design loads, load combinations, geotechnical data and material properties please refer to 1.1.1.1.

7.1.14.2 Design Methods Applied

Please refer to above Section on Connecting Tunnel.

7.1.14.3 Design Calculations

The design calculations included the numerical analysis of the powerhouse cavern and the wedge analysis.

Numerical Analysis for Outlet Structure Shaft

Model Generation

The 2-D numerical analysis was carried out in a Phase 2 finite element program. The proposed cross section of Outlet Structure Shaft was modelled as per its excavated profile. The external fixed boundary was modelled as three times the dimensions of the Outlet Structure Shaft.

For generation of model, the excavated boundary was created by importing a “DXF” File from AutoCAD. The external boundary was created by considering a box section with an expansion factor of 3 i.e. the external boundary was created at three times the excavated width on both sides, top and bottom. The model is analyzed considering three stages of excavation and the corresponding support systems.

A close- up view of the 2-D Outlet Structure Shaft model is shown in the figure below.

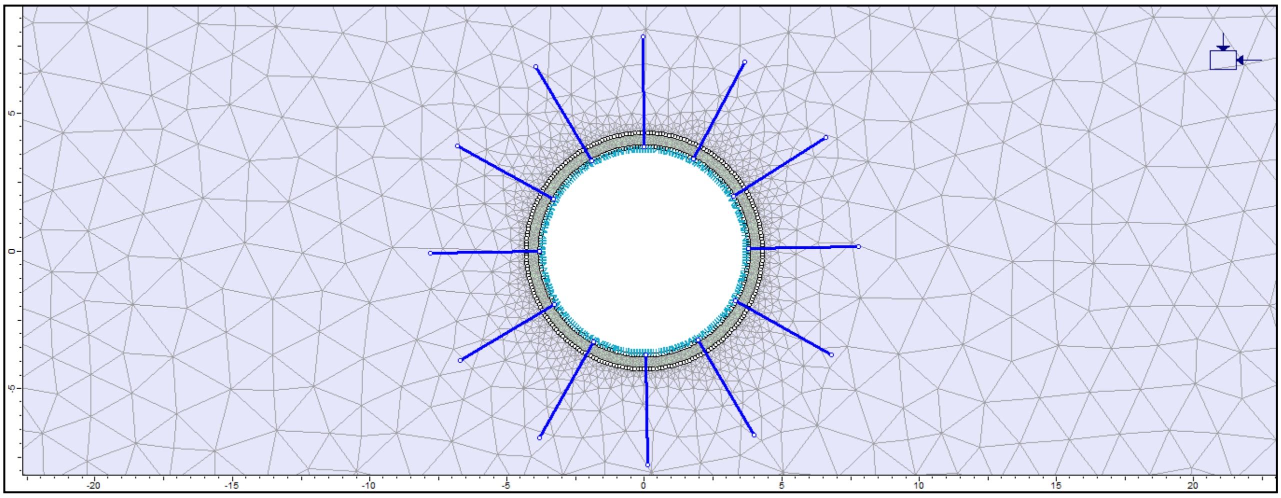

Figure 7-13: Close-up View of 2-D Outlet Structure Shaft Model

The assumptions entering the model included the stages of excavation, the properties of the rock material, the loading details and the support system. These assumptions are presented in detail in Part A3 Chapter 3 of the report and are not reiterated here.

Results and Discussions

The type of analysis performed is plain strain analysis. Plain strain is defined t as a state of strain in which the strain normal to the x-y plane (< z) and shear strain (< xz and < yz) are assumed to be zero.

Conclusion

Based on the above results, it was concluded that the deformed shape of the model was as expected. As per this 2-D numerical analysis of Outlet Structure Shaft, with proposed support system, the maximum deformation is about 7.2 mm. The details are shown in the annexes to Part A3 Chapter 3 of this report.

The maximum limit of yielded elements from the excavated face is about 1.0 m and the proposed rock bolts are up to a length of 4.0 m, which is safe. The details are shown in the annexes to Part A3 Chapter 3 of this report.

There were no yielded bolts and hence the design is safe. The details are shown in the annexes to Part A3 Chapter 3 of this report.

As per support capacity plots, none of the shotcrete elements has a factor of safety less than 1.0, which is safe. The details are shown in the annexes to Part A3 Chapter 3 of this report.

The recommended support system based on above analysis is given below in the Subsection Results of Calculation.

Wedge Analysis of Outlet Structure Shaft

In this study, the stability of the excavation profile of the Outlet Structure Shaft was checked against potential wedges that can develop based on identified structural discontinuities in the rock mass. The analysis for the Outlet Structure Shaft was carried out considering the excavation geometry. The wedge stability and the influence of the proposed support system was evaluated with the Unwedge software from Rocscience, which is based on a limit equilibrium analysis program. The analysis was carried out before and after the application of the support system.

The assumptions entering the model included the general input parameters, the joint properties and the support system. These assumptions are presented in detail in Part A3 Chapter 3 of the report and are not reiterated here.

Results of Wedge Analysis

Based on combinations of all joints, the summary of results for the wedge stability analysis, before and after support installation, is presented below.

Table 7-32: Summary of Results for the Wedge Stability Analysis

S.N.	Joint Combinations	Critical Failure Wedges	Wedge Weight (kN)	Excavated Face Area (m^2)	Factor of Safety (Without support)	Factor of Safety (With Support)
1	J2-J3-J4	West Wedge (6)	1735.84	36.46	0.889	4.018
2	J1-J3-J4	North West Wedge (5)	300.00	13.26	1.445	9.032

Conclusion of the Wedge Analysis

The analysis reveals the presence of only 2 critical failure wedges. The factor of safety (FOS) at various locations of the Outlet Structure Shaft assumes greater importance given the large (7.6m diameter) dimensions of the structure. Hence, a very conservative approach to stabilization was adopted. Based on this analysis and based on known joint sets and assumed rock properties, 4 m length of rock bolts @ 2.0 m c/c achieve the required FOS.

The observed factor of safety before and after the installation of the support shows that with the applied support, the potential wedges are safe. The critical wedges formed due to various joint and shear zone combinations are shown in the annexes to Part A3 Chapter 3 of this report.

7.1.14.4 Results of Calculations

The recommended support system for the Outlet Structure Shaft based on the above analysis is given in the table below:

Table 7-33: Recommended Support System

S.N.	Class of Rock	Proposed Support System
4	Class V	25 mm diameter, 4 m length fully grouted rock bolts @ 2.0 m c/c around entire periphery 50 mm thick plain shotcrete around entire periphery

7.1.15 Outlet Structure - Tailrace Tunnel Portal and Shaft Structure Slope

The Outlet Structure consists of the Outlet Structure Shaft, the Outlet Tunnel, the Tailrace Portal and the Outlet Structure Tailbay.

The objective of this analysis and design of the slope above the Tailrace Tunnel Portal and the slope above the Outlet Shaft Structure, the determination of the required excavated profile and the design of the corresponding support system.

7.1.15.1 Specific Design Criteria

Geology of the slopes above the Tailrace Tunnel Portal and Outlet Structure Shaft

As per geological mapping and geotechnical investigations, the overburden material was determined to be colluvial material and bed rock consisting of Augen gneiss with chlorite schist partings. The portal location of the Tailrace Tunnel and Outlet Structure Shaft lies in Augen gneiss with a thin cover of colluvial material. The properties of the intact rock and rock mass are given below. The properties of intact rock are minimum values derived from various available test results. The test results are attached in the annexes to Part A3 Chapter 3 of this report.

7.1.15.2 Design Method Applied

Please refer to above Section on Connecting Tunnel and Part A3 Chapter 3 of this report.

7.1.15.3 Design Calculations

A detailed slope stability analysis was carried out to estimate the factor of safety for this slope for the proposed slope at the portal location of the Tailrace Tunnel Portal and the slope above the Outlet Structure Shaft and another similar analysis was carried out along the highest cut section in the Outlet Structure Shaft.

2-D slope stability analysis was carried out with Phase 2 software by a shear strength reduction approach. The proposed excavated section of the Portal was modelled at the location of portal for the Tailrace Tunnel along the Outlet Structure Shaft. The stability calculations were carried out with and without a support system and including / excluding seismic loads. The excavated slope modelled in Phase 2 is presented in the following two figures.

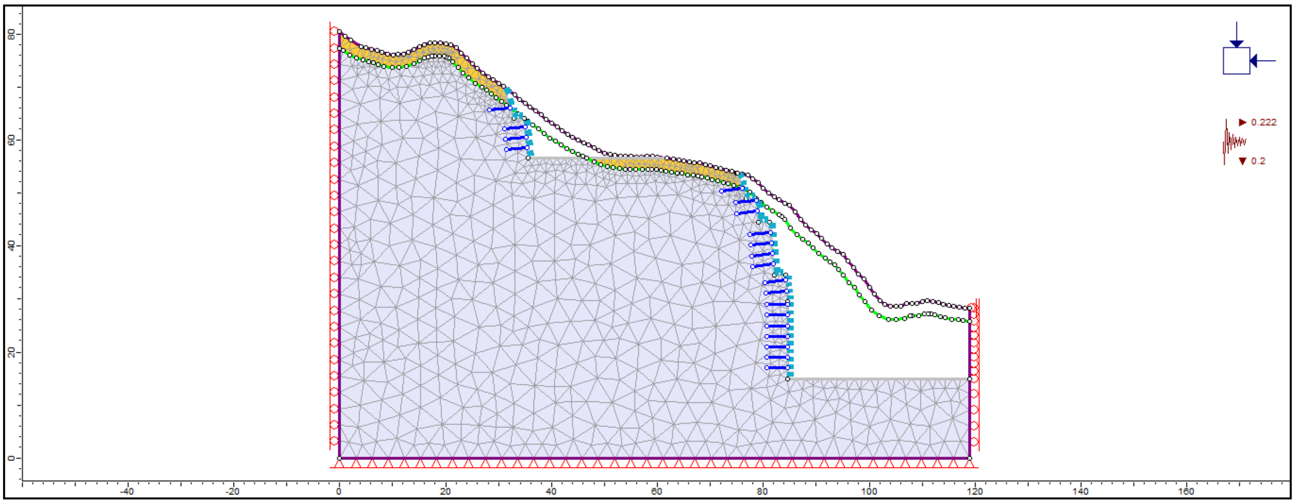

Figure 7-14: Excavated Slope of Tailrace Tunnel Portal and the Outlet Structure Shaft modelled in Phase2 Software

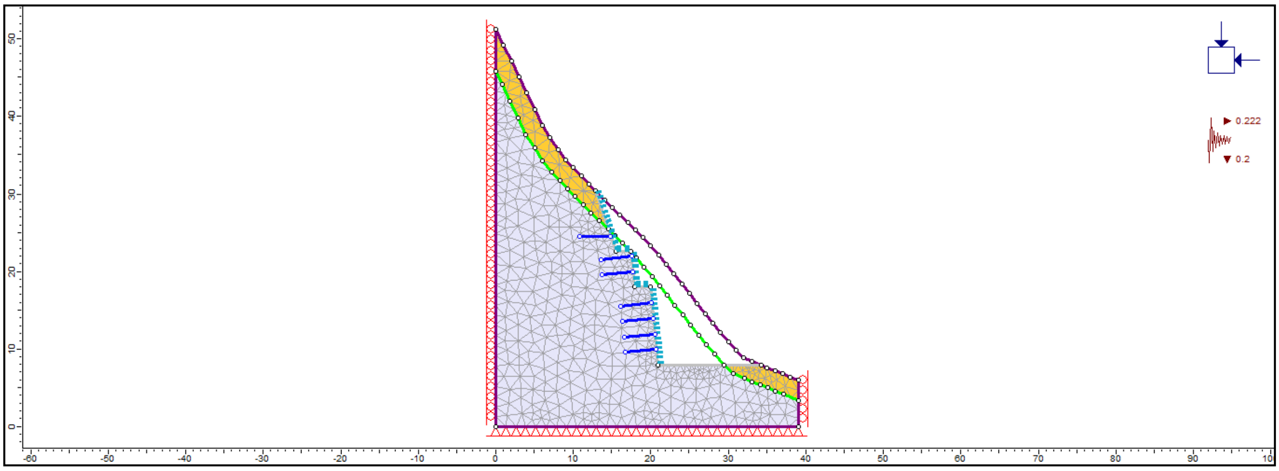

Figure 7-15: Excavated Slope of highest cut slope above the Outlet Structure Shaft modelled in Phase2 Software

Results and Discussions

The type of analysis performed was plain strain analysis. plain strain is defined as state of strain in which the strain normal to the x-y plane (< z) and shear strain (< xz and < yz) are assumed to be zero.

The proposed excavated slopes for rock mass are 10V:1H with a 2m bench at a height of 10 m with 50 mm thick shotcrete and 4m long rock bolts at @ 2 m c/c staggered and 3V:1H for colluvial material with 50 mm thick shotcrete. The excavated section for the slope stability analysis was considered accordingly. Based on results of the slope stability analysis, the summary of factor of safety obtained under various scenarios are given below. The factor of safety obtained under various conditions are shown below.

2. For Tailrace Tunnel Portal and the Outlet Structure Shaft Portal

Table 7-34: Factor of Safety Obtained under Various Conditions

S.N.	Case	Condition	Factor of Safety (Without Support System)	Factor of Safety (With Support System)
1	Case-1	Normal Case	1.85	2.02
2	Case-2	Seismic Case	1.14	1.30

3. For highest cut slope for Outlet Structure Shaft Portal

Table 7-35: Factor of Safety Obtained under Various Conditions

S.N.	Case	Condition	Factor of Safety (Without Support System)	Factor of Safety (With Support System)
1	Case-1	Normal Case	1.77	1.95
2	Case-2	Seismic Case	1.10	1.26

7.1.15.4 Results of Calculations

Based on the above results, it is concluded that the proposed slopes and the slope protection measures are adequate. The minimum required factors of safety for all load cases, with the support systems in place, are met. The designed system is safe. The proposed support systems for the excavated slopes are:

1) Slopes of 10V:1H with a 2m bench at a height of 10 m
2) 50 mm thick shotcrete and 4m long rock bolts at @ 2 m c/c staggered
3) Slopes of 3V:1H for colluvial material
4) 50 mm thick shotcrete.

7.1.16 Power Station Area - General

The Power Station consists of:

- Powerhouse Cavern
- Transformer Cavern,
- Bus duct Tunnels,
- Terminal & Ventilation Building
- Operation Building
- Workshop Building
- Sewage Treatment Building (no specific analysis)
- Take-Off Yards & Switching Station (no specific analysis)

In this design document, the support system of these tunnels and caverns are presented. A combined model of the Powerhouse Cavern and Transformer Cavern was also prepared to judge the adequacy of the rock cover between these two parallel caverns.

The objectives of the study are given below:

- Design of the support system for underground caverns and tunnels
- Determination of the adequacy of the rock cover between the Powerhouse and the Transformer Cavern.

7.1.17 Power Station Area - Powerhouse Cavern

7.1.17.1 General

The design of the rock support system in the Powerhouse Cavern requires a detailed analysis of the in situ rock stresses, rock strength parameters in conjunction with sequence of cavern excavation stages and subsequent application of rock bolt and shotcrete for rock support. The deformation modelling of the Powerhouse Cavern against in situ stresses have been studied for the analysis and design of rock support. The 2-D numerical analysis has been carried out with the Phase 2 Finite Element Program for deformation modelling. The analyses and their results have been discussed in this report. Initially, rock support system was estimated based on empirical relations and details presented in this report. Initially, the rock support system was estimated based on empirical relations. Subsequently, a numerical analysis was performed. Details are presented below.

7.1.17.2 Geology of the Powerhouse Cavern

The Powerhouse Cavern is composed of Augen gneiss with partings of chlorite schist. The rock mass has a gneissic texture. It contains quartz, feldspar, muscovite & biotite including phenocrysts of quartz and feldspar.

Augen gneiss is coarse to very coarse grained, have very closely to moderately spaced foliation joints (spacing >6cm to 60cm) and medium strong to strong in strength. The foliation joints (JS0) & some other joints are filled with sandy, silty and sometimes clayey material up to 5mm of thickness. Rock mass is slightly to moderately weathered.

Four well-developed main joint sets including foliation joints (JS0, JS1, JS2 & JS3) were observed near the Power Station area. Among them, the foliation joints (JS0) are the prominent sets. The dip angles & dip directions of the said joints are as follows: JS0 (Foliation joints): 05-15/330-350, JS1: 75-85 / 010-030, JS2: 65-75/065-085 & JS3: 70-85 / 170-210. The rock mass is blocky and slabby. The said rock bodies are well interlocked.

The rock mass is dry to damp in general. However, damp to slowly flowing groundwater condition can be anticipated during excavation.

7.1.17.3 Specific Design Criteria

Please refer to Section Headworks.

7.1.17.4 Design Method Applied

Please refer to Section Headworks.

7.1.17.5 Design Calculations

The design calculations included the numerical analysis of the powerhouse cavern and the wedge analysis.

Numerical Analysis of Powerhouse Cavern

Model Generation

The 2-D numerical analysis is carried out in a Phase2 finite element programme. The proposed section of the Powerhouse was modelled as per its excavated profile. The external fixed boundary was considered as three times its dimension in each direction.

For generation of the model, an excavated boundary was created by importing a “DXF” file from AutoCAD. The external boundary was created by considered a box section with an expansion factor of 3 i.e. the external boundary was created at three times of its excavated width on both sides, top and bottom. The model was analysed considering sixteen stages of excavation and the corresponding support systems. The stage boundaries were also created by importing a “DXF” file from AutoCAD.

A close-up view of 2-D Powerhouse Cavern model is shown in the figure below.

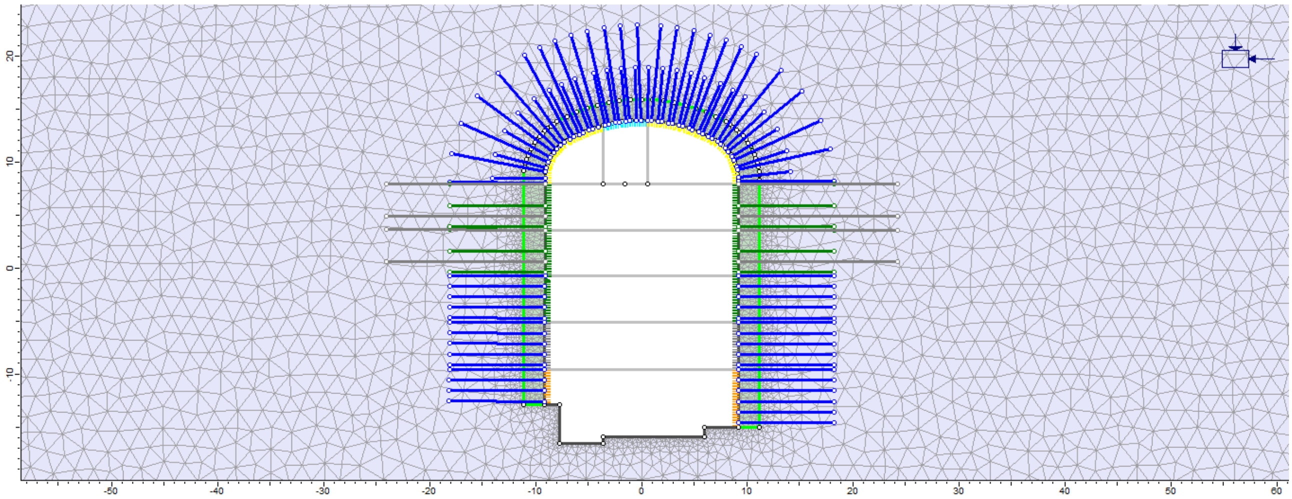

Figure 7-16: Close-up View of 2D Model of Powerhouse Cavern

The assumptions entering the model included the stages of excavation, the properties of the rock material, the loading details and the support system. These assumptions are presented in detail in Part A3 Chapter 4 of the report and are not reiterated here.

<u>Results and Discussions</u>

The type of analysis performed was plain strain analysis. Plain strain is defined to be a state of strain in which the strain normal to the x-y plane (< z) and shear strain (< xz and < yz) are assumed to be zero. The following summary of various parameters determined after analysis are given below.

Table 7-36: Summary of Various Parameters Determined after Analysis

S.N.	Case	Condition	Maximum Displacement (mm)			
			On Left Wall	On Right Wall	At Crown	At Bottom
1	Case-1	Sixteen Stage Excavation with Support	166.9	163.5	44.3	100.9

A) Total displacement

The maximum value of displacement was about 166.9 mm at mid-height of the left side wall and 163.5 mm at mid height of right side wall of the Powerhouse. At the crown, the displacement is about 44.3 mm and at bottom the displacement is about 100.9 mm.

B) Yielded Zone

The maximum radius of the plastic zone was about 15 m. Thus, the plastic zone was about 6 m beyond the excavation boundary.

C) Support Capacity of Rock Bolts

Due to deformation after application of the support system, some rock bolts yielded. The number of yielded rock bolts are small and hence the design is safe.

Conclusion

Based on above results, it can be concluded that the deformed shape of model was as expected. As per this 2-D numerical analysis of the Powerhouse Cavern, with the proposed support system, the deformation was about 166.9 mm, which is about 0.91% of the excavated width and hence within the allowable limit.

The maximum limit of yielded elements from the excavated faces was about 6 m and the rock bolts proposed are 9 m, which reflects a safe design.

The number of yielded bolts was limited and hence safe.

As per support capacity plots, only limited portions of the shotcrete have a factor of safety less than 1.0, which is acceptable and, therefore, the shotcrete design is safe.

The recommended support system based on above analysis is given in the following subsection results of calculations.

Wedge Analysis of Powerhouse Cavern

In this study, the stability of the excavated profile of the underground Powerhouse was checked against potential wedges that can develop based on identified structural discontinuities in the rock mass. The analysis for the Powerhouse was carried out considering the excavated geometry. The wedge stability and the influence of the proposed support was evaluated by Unwedge software from Rocscience, which is based on a limit equilibrium analysis program. The analysis was carried out before and after the application of a support system.

The assumptions entering the model included the general input parameters, the joint properties and the support system. These assumptions are presented in detail in Part A3 Chapter 4 of the report and are not reiterated here.

Results of Wedge Analysis

Based on combinations of all joints, the summary of results for wedge stability analysis before and after support installations presented below.

Table 7-37: Summary of Results for Wedge Stability Analysis

S.N.	Joint Combinations	Critical Failure Wedges	Wedge Weight (kN)	Excavated Face Area (m^2)	Factor of Safety (With-out support)	Factor of Safety (With Support)
1	J1-J2-J3	Upper Left Wedge (5)	18373	306.24	0.297	5.489
2	J1-J2-J3	Roof Wedge (6)	54.29	10.85	0.509	78.091
3	J1-J2-J3	Roof Wedge (8)	39.45	7.30	0.000	55.445
4	J1-J2-J4	Roof Wedge (8)	86.59	9.39	0.000	54.975
5	J2-J3-J4	Roof Wedge (6)	13909	155.41	0.528	2.567
6	J2-J3-J4	Roof Wedge (8)	0.54	0.33	0.000	445.631
7	J1-J3-J4	Upper Left Wedge (6)	2332	107.52	0.481	16.585
8	J1-J3-J4	Upper Right Wedge (8)	15.05	5.21	0.000	142.721

Conclusion of the Wedge Analysis

The analysis reveals the presence of 8 critical failure wedges. The factors of safety (FOS) at various locations of the cavern assumes greater importance given the of height of the Powerhouse of about 30 m. Any wedge failure in the crown portion, in particular a failure of unconfined wedges around it, could create crown instability of catastrophic proportions. Hence, a very conservative approach to stabilization was adopted. Based on this analysis and based on known joint sets and assumed rock properties, rock bolts of 9 m length achieve the required FOS.

The calculated FOS, before and after the installation of the support, shows that, with the applied support, the potential wedges are safe.

7.1.17.6 Results of Calculations

Table 7-38: Proposed Support System

S.N.	Class of Rock	Proposed Support System
1	Class III to V	- 32 mm diameter, 9.0 m length fully grouted rock bolts @ 1.0 m c/c in crown as well as side walls below machine hall level. - 32 mm diameter, 9.0 m length fully grouted rock bolts @ 2.0 m c/c and 36 mm dia DCP monobar anchors @ 3.0 m c/c on side walls above machine hall level. - 300 mm thick plain shotcrete with two layers of welded wire mesh of 100 x 100 x 4 mm on crown and 200 mm thick plain shotcrete with two layers of welded wire mesh of 100x 100 x 4 mm on side walls

In addition to the above support system, a pattern of drainage holes of 75 mm diameter, 6.0 m length @ 3.0 m c/c shall be provided. The flow through the drainage holes will be channelled into surface drains in the Powerhouse.

7.1.18 Power Station Area - Transformer Cavern

The design of the rock support system in the Transformer Cavern requires a detailed analysis of the in situ rock stresses, rock strength parameters in conjunction with the sequence of the cavern excavation stages and subsequent application of rock support measures such as rock bolts and shotcrete. The deformation modelling of the Transformer Cavern against in situ stresses have been studied for the analysis and design of rock support. The 2-D numerical analysis has been carried out in a Phase2 finite element program for deformation modelling. The analyses and their results have been discussed in this report. Initially, the rock support systems were estimated based on empirical relations. The details are presented below in this report.

7.1.18.1 Geology of the Transformer Cavern

The Transformer Cavern is composed of Augen gneiss with partings of chlorite schist. The rock mass has a gneissic texture. It contains quartz, feldspar, muscovite & biotite including phenocrysts of quartz and feldspar.

Augen gneiss is coarse to very coarse grained, has very closely to moderately spaced foliation joints (spacing >6cm to 60cm) and medium strong to strong in strength. The foliation joints (JS0) & some other joints are filled with sandy, silty and sometimes clayey material up to 5mm of thickness. The rock mass is slightly to moderately weathered.

Four well-developed main joint sets, including foliation joints (JS0, JS1, JS2 & JS3), were observed near the Power Station area. Among them, the foliation joint (JS0) are the prominent set. The dip angles & dip directions of the said joints are as follows: JS0 (Foliation joints): 05-15/330-350, JS1: 75-85 / 010-030, JS2: 65-

75/065-085 & JS3: 70-85 / 170-210. The rock mass is blocky and slabby. The said rock bodies are well interlocked.

The rock mass is dry to damp in general. However, damp to slowly flowing groundwater conditions can be anticipated during excavation.

7.1.18.2 Specific Design Criteria

Please refer to Section Headworks.

7.1.18.3 Design Method Applied

Please refer to Section Headworks.

7.1.18.4 Design Calculations

The design calculations included the numerical analysis of the transformer cavern and the wedge analysis.

Numerical Analysis of Transformer Cavern

Model Generation

A 2-D numerical analysis was carried out in a Phase2 finite element programme. The proposed section of the Transformer Cavern was modelled as per its excavated profile. The external fixed boundaries were considered as three times its dimensions in each direction.

For generation of the model, an excavated boundary was created by importing a “DXF” File from AutoCAD. The external boundaries were created by considering a box section with an expansion factor of 3 i.e. the external boundaries were created at three times the excavated width, top and bottom. The model was analysed considering ten stages of excavation and the corresponding support systems. The excavated stage boundaries were also created by importing “DXF” files from AutoCAD.

A close-up view of the 2-D Transformer Cavern model is shown in below figure.

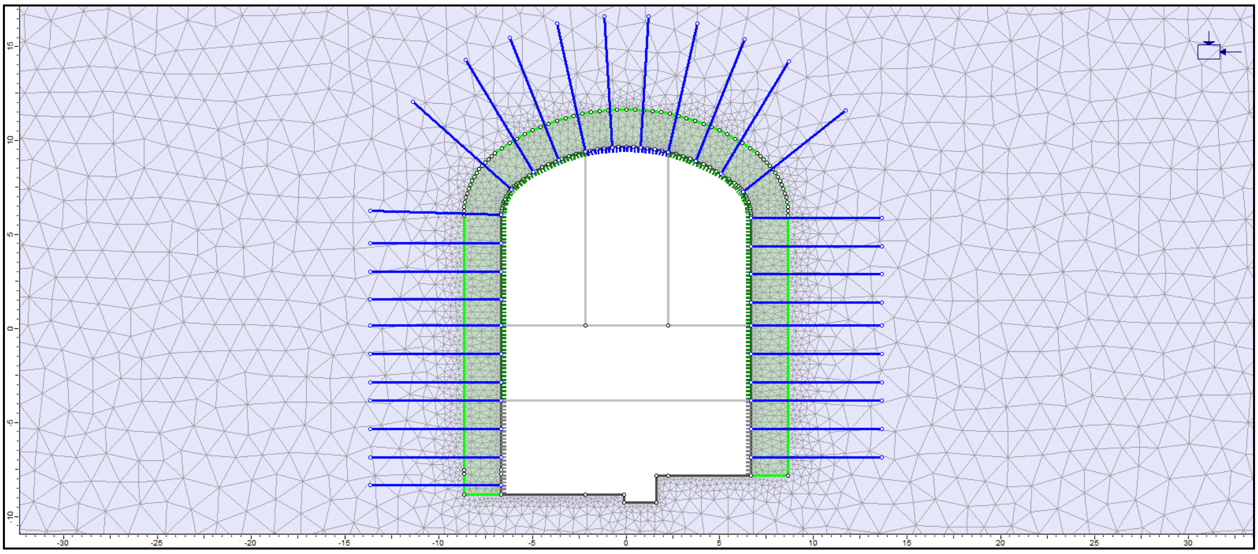

Figure 7-17: Close-up View of 2D Model of Transformer Cavern

The assumptions entering the model included the stages of excavation, the properties of the rock material, the loading details and the support system. These assumptions are presented in detail in Part A3 Chapter 4 of the report and are not reiterated here.

Results and Discussions

The type of analysis performed was plain strain analysis. Plain strain is defined as a state of strain in which the strain normal to the x-y plane (< z) and shear strain (< xz and < yz) are assumed to be zero. The following summary of the various parameters determined after the analysis are given below.

Table 7-39: Summary of Various Parameters Determined

S.N.	Case	Condition	Maximum Displacement (mm)			
			On Left Wall	On Right Wall	At Crown	At Bottom
1	Case-1	Ten Stage Excavation with Support	97.1	96.0	34.2	72.1

Total displacement

The maximum value of displacement was about 97.1 mm at mid-height of left side wall and 96.0 mm at mid height of right side wall of the Transformer Cavern. At the crown, the displacement is about 34.2 mm and at bottom, displacement is about 72.1 mm.

Yielded Zone

The maximum radius of the plastic zone is about 10 m. Thus, the plastic zone is about 3.5 m beyond excavation boundary.

Support Capacity of Rock Bolts

Due to deformation after application of the support system, some rock bolts have yielded. The number of yielded rock bolts are small and hence the designed support is safe.

Conclusion

Based on above results, it is concluded that deformed shape of the model is as expected. As per this 2-D numerical analysis of the Transformer Cavern, with the proposed support system, the maximum deformation is about 97.1 mm, which is about 0.72% of excavated width and, hence, within the maximum allowable limit.

The maximum limit of the yielded elements from the excavated face is about 3.5 m and the proposed rock bolt are up to 7 m, which is safe.

The number of yielded bolts is small and hence safe.

As per support capacity plots, only a limited portion of the shotcrete has a factor of safety less than 1.0, which is acceptable, and the shotcrete design is safe.

The recommended support system based on the above analysis is given in the following subsection results of calculations.

Wedge Analysis of Transformer Cavern

In this study, the stability of the excavation profile of the underground Transformer Cavern was checked against potential wedges that can develop based on identified structural discontinuities in the rock mass. The analysis of the Transformer Cavern was carried out considering the excavated geometry. The wedge stability and the influence of the proposed support was evaluated by the Unwedge software of Rocscience, which is based on a limit equilibrium analysis program. The analysis was carried out before and after the application of the support system. The detail of the analysis is described below.

The assumptions entering the model included the general input parameters, the joint properties and the support system. These assumptions are presented in detail in Part A3 Chapter 4 of the report and are not reiterated here.

Results of Wedge Analysis

Based on combinations of all joints, the summary of results for wedge stability analysis before and after support installation is presented below.

Table 7-40: Summary of Results for Wedge Stability Analysis

S.N.	Joint Combinations	Critical Failure Wedges	Wedge Weight (kN)	Excavated Face Area (m^2)	Factor of Safety (Without support)	Factor of Safety (With Support)
1	J1-J2-J3	Upper Left Wedge (5)	5846	144.31	0.337	4.419
2	J1-J2-J3	Roof Wedge (6)	17.86	5.16	0.693	101.321
3	J1-J2-J3	Upper Right Wedge (8)	20.99	4.64	0.000	67.354
4	J1-J2-J4	Roof Wedge(8)	32.39	4.86	0.000	83.066
5	J2-J3-J4	Roof Wedge (6)	5086	76.12	0.636	3.371
6	J2-J3-J4	Upper Right Wedge (8)	0.07	0.08	0.000	1372.082
7	J1-J3-J4	Upper Left Wedge (6)	768.6	51.84	0.591	14.759
8	J1-J3-J4	Roof Wedge (8)	4.84	2.43	0.000	161.720

Conclusion of Wedge Analysis

The analysis reveals the presence of 8 critical failure wedges. The factor of safety (FOS) at various locations of the cavern assumes greater importance given the height of the Transformer Cavern of about 18 m. Any wedge failure in the crown portion, in particular failure of unconfined wedges around it, could create crown instability of catastrophic proportions. Hence, a very conservative approach to stabilization was adopted. Based on this analysis and based on known joint sets and assumed rock properties, rock bolts of 7 m length achieve the required FOS.

The observed Factor of Safety before and after installation of the support shows that with the application of support, the potential wedges are safe.

7.1.19 Powerhouse and Transformer Caverns combined

Model Generation

A 2-D numerical analysis of a combined Powerhouse and Transformer Cavern was also carried out to judge the adequacy of the rock cover between these caverns. The proposed section of the Powerhouse and Transformer caverns were modelled as per their excavated profiles. The external fixed boundaries were modelled as three times the width of the Powerhouse Cavern in each direction.

For generation of model an excavated boundary was created by importing a “DXF” file from AutoCAD. The external boundaries were also created by importing a “DXF” file from AutoCAD. The model was analysed considering twenty-four stages of excavation and their corresponding support systems. The boundaries for the excavation stages were also created by importing “DXF” files from AutoCAD.

A close- up view of 2-D combined Powerhouse and Transformer Cavern model is shown in **Figure 4-6**.

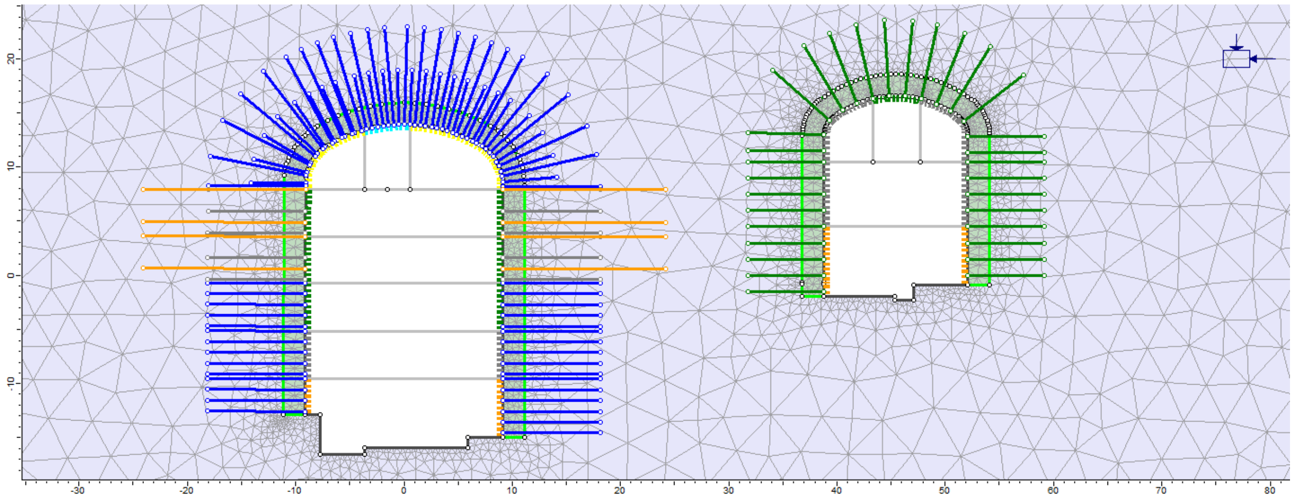

Figure 7-18: Close-up View of 2-D Model of Combined Powerhouse and Transformer Cavern

The assumptions entering the model included the stages of excavation, the properties of the rock material, the loading details and the support system. These assumptions are presented in detail in Part A3 Chapter 4 of the report and are not reiterated here.

Results and Discussions

The type of analysis performed was plain strain analysis. Plain strain is defined as a state of strain in which the strain normal to the x-y plane (< z) and shear strain (< xz and < yz) are assumed to be zero. The following summary of the various parameters determined after analysis are given below.

Table 7-41: Summary of Parameters

S.N.	Case	Condition	Maximum Displacement (mm)			
			On Left Wall	On Right Wall	At Crown	At Bottom
1	Case-1	Twenty Four Stage Excavation with Support				
		Powerhouse Cavern	147.5	157.0	54.6	90.2
		Transformer Cavern	52.4	113.4	51.1	68.6

Total displacement

The maximum value of displacement for the Powerhouse Cavern is about 147.5 mm at mid-height of the left side wall and 157.0 mm at mid height of right side wall of Powerhouse. At the crown of the Powerhouse, the displacement is about 54.6 mm and at the bottom the displacement is about 90.2 mm.

The maximum value of displacement for the Transformer Cavern is about 52.4 mm at mid-height of left side wall and 113.4 mm on mid height of right side wall of Transformer Cavern. At crown of Transformer Cavern, the displacement is about 51.1 mm and at bottom, displacement is about 66.6 mm.

Table 7-42: Summary of Maximum Displacements of Powerhouse Cavern and Transformer Cavern

Powerhouse Cavern	Unit	Displacement
Left Side Wall	mm	147.5
Right Side Wall	mm	157.0
Crown	mm	54.6
Bottom	mm	90.2

Transformer Cavern	Unit	Displacement
Left Side Wall	mm	52.4
Right Side Wall	mm	113.4
Crown	mm	51.1
Bottom	mm	68.6

<u>Yielded Zone</u>

For the Powerhouse Cavern, the maximum radius of the plastic zone is about 15 m. Thus, the plastic zone is about 6 m beyond the excavation boundary.

For the Transformer Cavern, the maximum value of radius of plastic zone is about 11 m. Thus, the plastic zone is about 5 m beyond excavation boundary.

<u>Support Capacity of Rock Bolts</u>

Due to deformation after application of the support system, some of the rock bolts have yielded. The number of yielded rock bolts, however, is small and, hence, the design is safe.

<u>Conclusion</u>

Based on above results, it is concluded that deformed shape of the combined model of Powerhouse and Transformer Caverns is as expected. As per this 2-D numerical analysis of the combined Powerhouse and Transformer Caverns, with the proposed support system, the maximum deformation was about 157.0 mm for Powerhouse Cavern, which is about 0.85% of excavated width. For Transformer caver, the maximum deformation is about 113.4 mm, which is about 0.85% of excavated width. These deformations are within the allowable limits (which is 2%).

The maximum limit of yielded elements from the excavated face is about 6 m for Powerhouse and about 5 m for Transformer Caverns and rock bolts lengths are proposed up to 9 m for Powerhouse Cavern and 7.0 m for Transformer Cavern, which is a safe design.

The number of yielded bolts is small and, hence, the design is safe.

As per support capacity plots, only limited portions of the shotcrete has a factor of safety less than 1.0, which is acceptable and, therefore the shotcrete as designed is safe.

Table 7-43: Proposed Support System

S.N.	Class of Rock	Proposed Support System
1	Class III to V	- 32 mm diameter, 9.0 m length fully grouted rock bolts @ 1.0 m c/c in crown as well as side walls of Power House Cavern above EL 990.81 m and below EL 982.19 m. - 32 mm diameter, 9.0 m length fully grouted rock bolts @ 2.0 m c/c and 36 mm dia DCP monobar anchors @ 3.0 m c/c on Power house side walls from EL 990.81 m to EL 982.19 m - 32 mm diameter, 7.0 m length fully grouted rock bolts @ 1.5 m c/c in crown as well as side walls of transformer cavern - 300 mm thick plain shotcrete on crown and 200 mm thick plain shotcrete on side walls of Power House Cavern with two layers of wire mesh in crown as well as side walls - 200 mm thick plain shotcrete with two layers of wire mesh in the crown and side walls of Transformer cavern

In addition to the above support system, a pattern of drainage holes of 75 mm diameter, 6.0 m length @ 3.0 m c/c will be provided. The flow through drainage holes will be channelled into the surface drains of the Transformer Cavern.

7.1.20 Power Station Area - Bus Duct Tunnels

7.1.20.1 Design of Support System for the Bus Duct Tunnel

The design of the rock support system in the Bus duct Tunnel requires a detailed analysis of the in situ rock stresses, rock strength parameters, application of rock bolts and shotcrete. The deformation modelling of the Bus Duct Tunnel subjected to in situ stresses were studied for the analysis and design of the rock support. The 2-D numerical analysis was carried out with the Phase 2 finite element program for deformation and rock support modelling. The analyses and their results are presented in this report. The initial rock support system was estimated based on empirical relations and the details are presented below. Subsequently, a numerical analysis was performed. Details are presented below.

7.1.20.2 Geology of the Bus Duct Tunnel

The Bus Duct Tunnel is composed of Augen gneiss with partings of chlorite schist. The rock mass has a gneissic texture. It contains quartz, feldspar, muscovite & biotite including phenocrysts of quartz and feld-spar.

Augen gneiss is coarse to very coarse grained, has very closely to moderately spaced foliation joints (spacing >6cm to 60cm) and medium strong to strong in strength. The foliation joints (JS0) & some other joints are filled with sandy, silty and sometimes clayey material up to 5mm of thickness. The rock mass is slightly to moderately weathered.

Four well-developed main joint sets including foliation joints (JS0, JS1, JS2 & JS3) were observed near the Power Station area. Among them, the foliation joints (JS0) are the prominent sets. The dip angles & dip directions of the said joints are as follows: JS0 (Foliation joints): 05-15/330-350, JS1: 75-85 / 010-030, JS2: 65-75/065-085 & JS3: 70-85 / 170-210. The rock mass is blocky and slabby. The said rock bodies are well interlocked.

The rock mass is dry to damp in general. However, damp to slowly flowing groundwater condition can be anticipated during excavation.

7.1.20.3 Specific Design Criteria

Please refer to Section Headworks.

7.1.20.4 Design Method Applied

Please refer to Section Headworks.

7.1.20.5 Design Calculations

The design calculations for the Bus Duct Tunnels follow in principal the same methodology as shown above for the Powerhouse and Transformer Caverns. They are presented in full detail in Part A3 Chapter 4 of this Detailed Design Report.

However, in order to provide the descriptions in the Main Volume of the Detailed Design Report in a concise manner, the details are not reiterated here. Rather, the focus is confined to the results of the analyses which are presented here below.

7.1.20.6 Numerical Analysis of Bus Duct Tunnel

Employed Model

The 2-D numerical analysis was carried out with the Phase2 Finite Element programme. The proposed section of the Bus Duct Tunnel was modelled as per its excavated profile. The external fixed boundary was considered as three times its dimensions in each direction. a full face excavation of the Bus Duct Tunnel is proposed.

For generation of the model, an excavated boundary was created by importing a “DXF” file from AutoCAD. The external boundary was created by considered a box section with an expansion factor of 3 i.e. the external boundaries were created by multiplying the excavated dimensions by three. The model was analysed considering three stages of excavation and the corresponding support systems.

Results and Discussions

The type of analysis performed is plain strain analysis. Plain strain is defined as a state of strain in which the strain normal to the x-y plane (< z) and shear strain (< xz and < yz) are assumed to be zero. The details of various parameters determined after the analysis are given below for each rock class.

Conclusion

Based on above results, it is concluded that deformed shape of the model is as expected. As per this 2-D numerical analysis of the Bus Duct Tunnel, with the proposed support system the maximum deformation was about 16.3 mm, which is within the maximum permissible limit.

The maximum limit of the yielded elements from the excavated face is about 1.0 m and the proposed rock bolt lengths are 2.5 m, which is a safe design.

None of the rocks bolts yielded and hence imply a safe design.

As per support capacity plots, none of the shotcrete elements had a factor of safety less than 1.0, which implies an acceptable and safe shotcrete design.

7.1.20.7 Wedge Analysis of Bus Duct Tunnel

Approach

In this study, the stability of the excavation profile of the Bus Duct Tunnel was checked against potential wedges that can develop based on identified structural discontinuities in the rock mass. The analysis for the Bus Duct Tunnel was carried out considering the excavation geometry. The wedge stability and the influence of the proposed support was evaluated by the Unwedge software from Rocscience, which is based on a limit equilibrium analysis program. The analysis was carried out before and after application of the support system. The detail of the analysis is described below.

Conclusion

The analysis reveals the presence of 7 critical failure wedges. The factor of safety (FOS) at various locations of the tunnel assumes greater importance given the size of Bus Duct Tunnel. Any wedge failure in the crown portion, in particular failure of unconfined wedges around it, could create crown instability of catastrophic proportions. Hence, a very conservative approach to stabilization was adopted. Based on this analysis and based on known joint sets and assumed rock properties, 2.5 m rock bolts achieve the required FOS.

The observed Factor of Safety before and after the installation of the support shows that, with the applied support, the potential wedges are safe.

7.1.20.8 Results of Calculations

Table 7-44: Proposed Support System

S.N.	Class of Rock	Proposed Support System
1	Class V	25 mm diameter fully grouted rock bolts of 2.5 m length @ 1.25 m c/c in crown and sides 100 mm thick plain shotcrete in Crown and sides with one layer of 100x100x4 mm wire mesh

7.1.21 Power Station Area - Terminal & Ventilation Building

Foundation Stress analysis

Based on deformations, the corresponding foundation stresses were computed. The calculated maximum deformation for all levels was 2.26 mm. The corresponding maximum load on foundation per unit area was:

Foundation bearing stress = soil subgrade reaction x deformation

= 80000 kN/m/m^2 x 2.26 mm

= 180.80 kN/m^2

The adopted safe bearing capacity is 80 ton/m^2 (= 800 kN/m^2), hence the structure is safe against bearing failure of the foundation.

7.1.22 Power Station Area - Operation Building

Bearing capacity of soil for ultimate limit state design was taken as 300 kN/m^2.

The bearing pressure on the foundation was checked for each load case. All footings were designed as isolated footings. The maximum bearing pressure was calculated for each load case. The RCC structural design was carried out for permissible maximum bearing pressure at limit state load combinations.

7.1.23 Power Station Area - Work Shop Building

Safe bearing capacity of the foundation material was assumed as 20 t/m^2 and soil subgrade reaction 20,000 kN/m/m^2.

The bearing pressure on foundation was checked for each load case, and a raft foundation is proposed for both blocks. The bearing pressure was calculated based on calculated deflections of area the springs assigned to the model.

7.1.24 Power Station Area - Take-Off Yards & Switching Station

Geology similar to Terminal and Ventilation Building therefore:

Foundation Stress analysis

Based on deformations, the corresponding foundation stresses were computed. The calculated maximum deformation was 2.26 mm. The corresponding maximum load on foundation per unit area was:

Foundation bearing stress = soil subgrade reaction x deformation

= 80000 KN/m/m^2 x 2.26 mm

= 180.80 KN/m^2

The adopted safe bearing capacity is 80 ton/m^2 (= 800KN/m^2), hence the structure is safe against bearing failure of the foundation.

7.1.25 Service Tunnels - Classification

The Service Tunnels are project components which are ancillary facilities which will be used for construction as well as for the operation of the HPP. These tunnels were designed with D-shaped sections and with sufficient dimensions for their intended purpose. These service tunnels are:

- Main Access Tunnel & Access Tunnel to Transformer Cavern
- Cable and Ventilation Tunnel
- Escape Tunnel
- Access Tunnel to Connecting Tunnel (is Adit 1 during construction)
- Aeration Tunnel to Spillway Tunnel (is access to Spillway Tunnel during Construction)
- Access Tunnel to U/S Valve Chamber & Adit 4 Plug (is Adit 4 during construction)
- Surge Tank Ventilation Gallery
- Adit to Tailrace Tunnel
- Access Tunnel to U/S Manifolds

The service tunnels were broadly classified into two different types based on the dimensions of the excavation and the depth of rock cover. The first classification is **Type A Service Tunnels**, for which the dimensions of the excavation were 4.4 m x 5.9 m (or smaller) and a maximum rock cover of about 250 m of metacarbonates with graphitic schist rock. The second classification, **Type B Service Tunnels**, have a maximum excavation size of 6.2 m x 6.1 m and a maximum rock cover of about 300 m of Augen gneiss with chloritic schist partings rock. Below, the service tunnels are arranged according to the above classification.

Given the importance of the Escape Tunnel it was analyzed and designed separately.

Table 7-45: Type A Service Tunnel

S.N.	Name of Service Tunnel
1	Access Tunnel to Connecting Tunnel (Adit 1)
2	Aeration Tunnel to Spillway Tunnel
3	Surge Tank Ventilation Gallery
4	Cable and Ventilation Tunnel

Table 7-46: Type B Service Tunnels

S.N.	Name of Service Tunnel
1	Access Tunnel to Headrace Tunnel (Adit 4)
2	Access Tunnel to U/S Valve Chamber
3	Main Access Tunnel
4	Access Tunnel to Transformer Cavern
5	Access tunnel to U/S Manifolds

In this chapter, the typical support system for these service tunnels are presented. The typical design of the portals of these service tunnels, where required, is as well presented.

The objectives of the study are given below:

- Design of Support System for Service Tunnels.
- Design of Support System for Service Tunnel Portals

7.1.26 Service Tunnels - Geological Description of the Tunnels

The geology of the various sections of service tunnels are discussed below.

7.1.26.1 Access Tunnel to Connection Tunnel (Adit 1 during Construction)

The Adit 1 portal and the corresponding Adit 1 Tunnel extends from approximately Ch. 0 to Ch. 90. The reach between Ch. 0 and Ch. 20 and Ch. 20 to Ch. 90 is composed of bedrock. The Ch. 10 to Ch. 20 is composed of a colluvial deposit

The bedrock is composed of banded gneiss. The rock mass contains quartz, feldspar, biotite, garnet, amphibole, and kyanite. Porphyroblasts of quartz, feldspar and garnet are frequently observed. Likewise, blue or green kyanite blades are also observed in biotite rich bands.

The rock mass is coarse to very coarse grained, strong to very strong in strength, having closely to widely spaced foliation joints. The foliation joints (JS0) & some other joints are filled with sandy silty and sometimes clayey material up to 5mm of thickness. The rock mass is slightly weathered along the joint surfaces.

Four well developed main joint sets, including foliation joints (JS0, JS1, JS2 & JS3), were observed at the Spillway Tunnel area. Among them, the foliation joints (JS0) are the prominent sets. The dip angles & dip directions of the said joints are as follows: JS0 (Foliation joints): 55-65/020-045, JS1: 50-60/080-085, JS2: 30-45/235 & JS3: 80-85/345-350. The rock mass is blocky in general. The said rock bodies are well interlocked.

The rock mass is dry at the outcrop. However, damp to dripping groundwater conditions can be anticipated during excavation.

7.1.26.2 Aeration Tunnel to Spillway Tunnel

The Aeration Tunnel to the Spillway Tunnel is composed of banded gneiss. The rock mass contains quartz, feldspar, biotite, garnet, amphibole, and kyanite. Phenocrysts of quartz, feldspar and garnet are frequently observed. Likewise, blue or green kyanite blades are also observed in biotite rich bands.

The rock mass is coarse to very coarse grained, has closely to widely spaced foliation joints, slightly weathered and strong to very strong in strength.

The foliation joints (JS0) & some other joints are filled with sandy silty and sometimes clayey material up to 5mm of thickness.

Four well developed main joint sets including foliation joints (JS0, JS1, JS2 & JS3) were observed at the Spillway Tunnel area. Among them, the foliation joints (JS0) are the prominent sets. The dip angles & dip directions of the said joints are as follows: JS0 (Foliation joints): 55-65/020-045, JS1: 50-60/080-085, JS2: 30-45/235 & JS3: 80-85/345-350. The rock mass is blocky in general. The said rock bodies are well interlocked.

Rock mass is dry at the outcrop. However, damp to dripping groundwater condition can be anticipated during excavation.

7.1.26.3 Access Tunnel to Headrace Tunnel (Adit 4 during Construction)

The Adit 4 portal and the corresponding tunnel extends from approximately Ch. 0 to Ch. 100. The surrounding rock is composed of Augen gneiss with partings of chlorite schist. The rock mass has a gneissic texture. It contains quartz, feldspar, muscovite & biotite including phenocrysts of quartz and feldspar. Likewise, the reach from approximately Ch. 10 to Ch. 20 is composed of chlorite schist and phyllite.

Augen gneiss is coarse to very coarse grained, has very closely to moderately spaced foliation joints (spacing >6cm to 60cm) and medium strong to strong in strength. The foliation joints (JS0) & some other joints are filled with sandy, silty and sometimes clayey material up to 5mm of thickness. The rock mass is slightly to moderately weathered.

Four well-developed main joint sets, including foliation joints (JS0, JS1, JS2 & JS3), were observed near the Adit 4 area. Among them, the foliation joints (JS0) are the prominent sets. The dip angles & dip directions of the said joints are as follows: JS0 (Foliation joints): 10-20/350-000, JS1: 75-85 / 010-030, JS2: 65-75/065-085 & JS3: 70-85 / 170-210. The rock mass is blocky and slabby. The said rock bodies are well interlocked.

On the other hand, chlorite schist and phyllite are green-gray, have very closely to closely spaced foliation joints (spacing >20cm) and very weak to medium strong in strength. The foliation joints (JS0) & some other joints are filled with silty, clayey material up to 10mm of thickness. The rock mass is slightly to moderately weathered.

Three well-developed main joint sets including foliation joints (JS0, JS1 & JS2) were observed near the Adit 4 area. Among them the foliation joints (JS0) are the prominent sets. The dip angles & dip directions of the said joints are as follows: JS0 (Foliation joints): 10-20/340-350, JS1: 55-70/065-085 & JS2: 70-80 / 155-170. The rock mass is slabby in general. The said rock bodies are well interlocked.

Rock mass at the outcrop of Adit 4 is dry to damp. However, damp to dripping groundwater condition can be anticipated during excavation.

7.1.26.4 Surge Tank Ventilation Gallery

The Surge Tank Ventilation Gallery in its entire reach is composed of chlorite schist and phyllite. They are light gray to green-gray, have very closely to closely spaced foliation joints (spacing >20cm) and very weak to medium strong in strength. The foliation joints (JS0) & some other joints are filled with silty, clayey material up to 10mm of thickness. The rock mass is slightly to moderately weathered.

Three well-developed main joint sets, including foliation joints (JS0, JS1 & JS2), were observed near the tunnel area. Among them the foliation joints (JS0) are the prominent sets. The dip angles & dip directions of the said joints are as follows: JS0 (Foliation joints): 10-20/330-350, JS1: 55-70/065-085 & JS2: 70-80 / 155-170. The rock mass is slabby in general. The said rock bodies are well interlocked.

Rock mass is dry to damp in general. However, damp to dripping groundwater condition can be anticipated during excavation.

7.1.26.5 Access Tunnel to U/S Valve Chamber

The Access Tunnel to the U/S Valve Chamber in its entire reach is composed of chlorite schist and phyllite. They are light gray to green-gray, have very closely to closely spaced foliation joints (spacing >20cm) and very weak to medium strong in strength. The foliation joints (JS0) & some other joints are filled with silty, clayey material up to 10mm of thickness. The rock mass is slightly to moderately weathered.

Three well-developed main joint sets, including foliation joints (JS0, JS1 & JS2), were observed near the tunnel area. Among them, the foliation joints (JS0) are the prominent sets. The dip angles & dip directions of the said joints are as follows: JS0 (Foliation joints): 10-20/330-350, JS1: 55-70/065-085 & JS2: 70-80 / 155-170.

The rock mass is dry to damp in general. However, damp to dripping groundwater condition can be anticipated during excavation.

7.1.26.6 Main Access Tunnel

The Main Access Tunnel to the Powerhouse is composed of Augen gneiss with partings of chlorite Sschist. The rock mass is has a gneissic texture. It contains quartz, feldspar, muscovite & biotite including phenocrysts of quartz and feldspar.

Augen gneiss is coarse to very coarse grained, have very closely to moderately spaced foliation joints (spacing >6cm to 60cm), slightly to moderately weathered and medium strong to strong in strength. The foliation joints (JS0) & some other joints are filled with sandy, silty and sometimes clayey material up to 5mm of thickness.

Four well-developed main joint sets, including foliation joints (JS0, JS1, JS2 & JS3,) were observed near the Power Station area. Among them, the foliation joints (JS0) are the prominent sets. The dip angles & dip directions of the said joints are as follows: JS0 (Foliation joints): 05-15/330-350, JS1: 75-85 / 010-030, JS2: 65-75/065-085 & JS3: 70-85 / 170-210. The rock mass is blocky and slabby. The said rock bodies are well interlocked.

The rock mass is dry to damp in general. However, damp to dripping groundwater condition can be anticipated during excavation.

7.1.26.7 Access Tunnel to Transformer Cavern

The Access Tunnel to the Transformer Cavern is composed of Augen gneiss with partings of chlorite schist. The rock mass has a gneissic texture. It contains quartz, feldspar, muscovite & biotite including phenocrysts of quartz and feldspar.

Augen gneiss is coarse to very coarse grained, has very closely to moderately spaced foliation joints (spacing >6cm to 60cm), slightly to moderately weathered and medium strong to strong in strength. The foliation joints (JS0) & some other joints are filled with sandy, silty and sometimes clayey material up to 5mm of thickness.

Four well-developed main joint sets, including foliation joints (JS0, JS1, JS2 & JS3) were observed near the Power Station area. Among them, the foliation joints (JS0) are the prominent sets. The dip angles & dip directions of the said joints are as follows: JS0 (Foliation joints): 05-15/330-350, JS1: 75-85 / 010-030, JS2: 65-75/065-085 & JS3: 70-85 / 170-210. The rock mass is blocky and slabby. The said rock bodies are well interlocked.

The rock mass is dry to damp in general. However, damp to dripping groundwater condition can be anticipated during excavation.

7.1.26.8 Access Tunnel to U/S Manifolds

The Access Tunnel to the U/S Manifolds is composed of Augen gneiss with partings of chlorite schist. The rock mass has a gneissic texture. It contains of quartz, feldspar, muscovite & biotite including phenocrysts of quartz and feldspar.

Augen gneiss is coarse to very coarse grained, has very closely to moderately spaced foliation joints (spacing >6cm to 60cm), slightly to moderately weathered and medium strong to strong in strength. The foliation joints (JS0) & some other joints are filled with sandy, silty and sometimes clayey material up to 5mm of thickness.

Four well-developed main joint sets, including foliation joints (JS0, JS1, JS2 & JS3), were observed near the Power station area. Among them, the foliation joints (JS0) are the prominent sets. The dip angles & dip directions of the said joints are as follows: JS0 (Foliation joints): 05-15/330-350, JS1: 75-85 / 010-030, JS2: 65-75/065-085 & JS3: 70-85 / 170-210. The rock mass is blocky and slabby. The said rock bodies are well interlocked.

The rock mass is dry to damp in general. However, damp to dripping groundwater condition can be anticipated during excavation.

7.1.26.9 Construction Adit to Tailrace Tunnel

The Construction Adit to the Tailrace Tunnel is composed of Augen gneiss with partings of chlorite schist. The rock mass has a gneissic texture. It contains quartz, feldspar, muscovite & biotite including phenocrysts of quartz and feldspar.

Augen gneiss is coarse to very coarse grained, having very closely to moderately spaced foliation joints (spacing >6cm to 60cm) and medium strong to strong in strength. The foliation joints (JS0) & some other joints are filled with sandy, silty and sometimes clayey material up to 5mm of thickness. The rock mass is slightly to moderately weathered.

Four well-developed main joint sets, including foliation joints (JS0, JS1, JS2 & JS3) were observed near the area. Among them, the foliation joints (JS0) are the prominent sets. The dip angles & dip directions of the said joints are as follows: JS0 (Foliation joints): 05-15/330-350, JS1: 75-85 / 010-030, JS2: 65-75/065-085 & JS3: 70-85 / 170-210. The rock mass is blocky and slabby.

The rock mass is dry to damp in general. However, damp to dripping groundwater condition can be anticipated during excavation.

7.1.26.10 Cable and Ventilation Tunnel

The Power House Cable and Ventilation Tunnel is composed of Augen Gneiss with partings of chlorite schist. The rock mass has a gneissic texture. It contains quartz, feldspar, muscovite & biotite including phenocrysts of quartz and feldspar.

Augen gneiss is coarse to very coarse grained, has very closely to moderately spaced foliation joints (spacing >6cm to 60cm), slightly to moderately weathered and medium strong to strong in strength. The foliation joints (JS0) & some other joints are filled with sandy, silty and sometimes clayey material up to 5mm of thickness.

Four well-developed main joint sets, including foliation joints (JS0, JS1, JS2 & JS3), were observed near the Power Station area. Among them the foliation joints (JS0) are the prominent sets. The dip angles & dip directions of the said joints are as follows: JS0 (Foliation joints): 05-15/330-350, JS1: 75-85 / 010-030, JS2: 65-75/065-085 & JS3: 70-85 / 170-210. The rock mass is blocky and slabby. The said rock bodies are well interlocked.

The rock mass is dry to damp in general. However, damp to dripping groundwater condition can be anticipated during excavation.

7.1.26.11 Escape Tunnel

The Escape Tunnel is composed of Augen gneiss with partings of chlorite schist. The rock mass has a gneissic texture. It contains quartz, feldspar, muscovite & biotite including phenocrysts of quartz and feldspar.

Augen gneiss is coarse to very coarse grained, has very closely to moderately spaced foliation joints (spacing >6cm to 60cm) and medium strong to strong in strength. The foliation joints (JS0) & some other joints are filled with sandy, silty and sometimes clayey material up to 5mm of thickness. The rock mass is slightly to moderately weathered.

Four well-developed main joint sets including foliation joints (JS0, JS1, JS2 & JS3) are observed near the Power Station area. Among them, the foliation joints (JS0) are the prominent sets. The dip angles & dip directions of the said joints are as follows: JS0 (Foliation joints): 05-15/330-350, JS1: 75-85 / 010-030, JS2: 65-75/065-085 & JS3: 70-85 / 170-210. The rock mass is blocky and slabby. The said rock bodies are well interlocked.

The rock mass is dry to damp in general. However, damp to slowly flowing groundwater condition can be anticipated during excavation.

7.1.27 Service Tunnels - Type A Service Tunnels

The design of the rock support system in the Service Tunnels requires a detailed analysis of the; in situ rock stresses, rock strength parameters, application of rock bolts and shotcrete for rock support. The deformation modelling of the tunnels subjected to in situ stresses were studied for the analysis and design of the rock support. The 2-D) numerical analysis was carried out with the Phase 2 finite element program for deformation and rock support modelling. Initially, the rock support systems were estimated based on empirical relations. Subsequently, a numerical analysis was performed. The analysis and design are presented below.

7.1.27.1 Specific Design Criteria

Please refer to Part A3 Chapter 5 of this report.

7.1.27.2 Design Methods Applied

Please refer to the Section Connecting Tunnel above.

7.1.27.3 Design Calculations

The design calculations included the numerical analysis of the tunnels and the wedge analysis.

Numerical Analysis of Service Tunnels Type A

Model Generation

A 2-D numerical analysis was carried out with the Phase 2 Finite Element program. The proposed section of Type A Service Tunnels and Type B Service Tunnels are modelled as per their excavated profile. The external fixed boundary was considered as three times its dimension in each direction. Full face excavation of tunnels is proposed.

For generation of the model, an excavated boundary was created by importing a "DXF" file from AutoCAD. The external boundary was created by considering a box section with an expansion factor of 3 i.e. the external boundary was created by three times of its excavated width on both sides and top and bottom. The model was analysed considering three stages of excavation and support system.

A close-up view of 2-D Service Tunnel Type A is shown in the following figure.

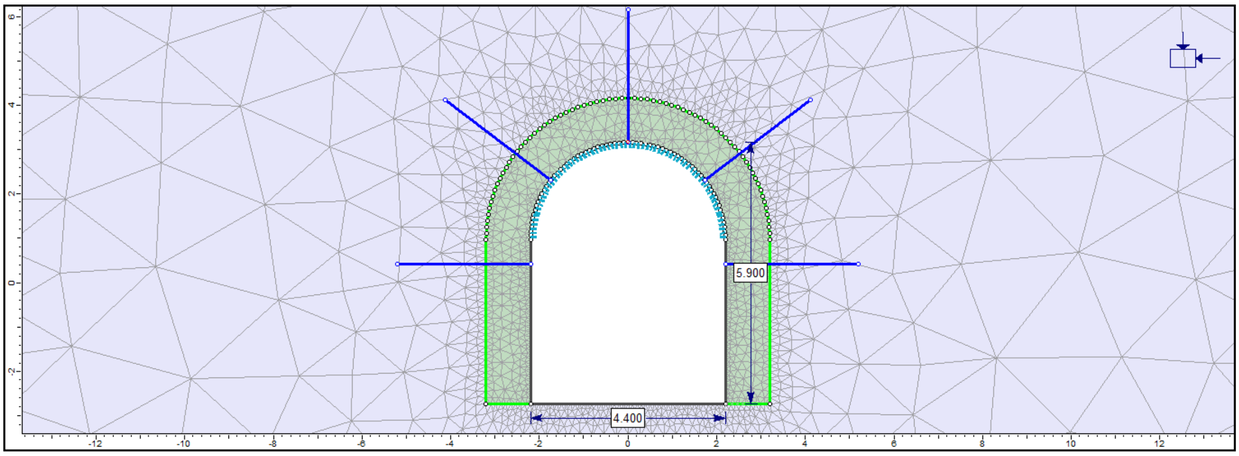

Figure 7-19: Close-up View of 2-D Model of Type A Service Tunnel

The assumptions entering the model included the stages of excavation, the properties of the rock material, the loading details and the support system. These assumptions are presented in detail in Part A3 Chapter 5 of the report and are not reiterated here.

Results and Discussions

The type of analysis performed is plain strain analysis. Plain strain is defined as a state of strain in which the strain normal to the x-y plane (< z) and shear strain (< xz and < yz) are assumed to be zero. The details of various parameters determined after analysis are given below for each rock class.

Conclusion on Numerical Analysis

Based on above results, it is concluded that deformed shape of the model was as expected. As per this 2-D numerical analysis of connecting tunnel, with the proposed support system, the maximum deformation for Type A service tunnels was about 6.4 mm for Class II rock, 14.5 mm for Class III rock, 33.2 mm for class IV rock and 66.7 mm for Class V rock, which are within the allowable limit. The details are shown in the annexes to Part A3 Chapter 5 of this report.

The maximum value of yielded elements from the excavated face is about 1.5 m. The rock bolts are proposed up to a length of 3.0 m, which is a safe design. The details are shown in the annexes to Part A3 Chapter 5 of this report.

The number of yielded bolts is very small and hence the design is safe. The details are shown in the annexes to Part A3 Chapter 5 of this report.

As per the support capacity plots, only limited portions of the shotcrete have a factor of safety less than 1.0 and only limited portions of steel ribs have a factor of safety less than 1.5, which is acceptable. The shotcrete and steel ribs as designed are safe. The details are shown in the annexes to Part A3 Chapter 5 of this report.

The recommended support system based on the above analysis is given in below Subsection Results of Calculations.

Wedge Analysis of Type A Service Tunnels

In this study, the stability of the excavation profile of the underground Service Tunnels (Type A) was checked against potential wedges that can develop based on identified structural discontinuities in the rock mass. The analysis for the Service Tunnels was carried out considering the excavation geometry. The wedge stability and the influence of the proposed support was evaluated by the Unwedge software from Rocscience, which is based on a limit equilibrium analysis program. The analysis was carried out before and after the application of a support system.

The assumptions entering the model included the general input parameters, the joint properties and the support system. These assumptions are presented in detail in Part A3 Chapter 5 of the report and are not reiterated here.

Results of Wedge Analysis

Based on combinations of all joints, the summary of results for wedge stability analysis, before and after support installation, is presented below.

Table 7-47: Summary of Results for Wedge Stability Analysis

S.N.	Joint Combinations	Critical Failure Wedges	Wedge Weight (kN)	Excavated Face Area (m^2)	Factor of Safety (Without support)	Factor of Safety (With Support)
1	J2-J3-J4	Upper Right Wedge (4)	166.82	10.12	0.955	6.512
2	J2-J3-J4	Roof Wedge (8)	1.13	0.51	0	158.029
3	J1-J3-J4	Roof Wedge (8)	0.01	0.03	0	744.124
4	J1-J2-J3	Upper Right Wedge (6)	270.12	11.29	1.057	5.798
5	J1-J2-J3	Roof Wedge (8)	0.09	0.11	0	1710

<u>Conclusion of Wedge Analysis</u>

The analysis reveals the presence of 5 critical failure wedges for the Service Tunnels Type A. The factor of safety (FOS) at various locations of the tunnel assumes greater importance given size of the Service Tunnels. Any wedge failure in the crown portion, in particular failure of unconfined wedges around it, could create crown instability of catastrophic proportions. Hence, a very conservative approach was adopted. Based on this analysis and based on known joint sets and assumed rock properties, 3.0 m rock bolts for the Service Tunnel Type A contributes to the required FOS.

The observed factor of safety before and after installation of the support shows that, with the applied support, the potential wedges are safe. The critical wedges formed due to various joint combinations for Service Tunnel Type A are shown in the annexes to Part A3 Chapter 5 of this report.

7.1.27.4 Results of Calculations

Table 7-48: Proposed Support System

S.N.	Class of Rock	Proposed Support System
1	Class II	25 mm diameter fully grouted rock bolts @ 2 m c/c in crown only 50 mm thick plain shotcrete in crown only
2	Class III	25 mm diameter fully grouted rock bolts @ 1.75 m c/c in crown only 50 mm thick plain shotcrete in crown and sides
3	Class IV	25 mm diameter fully grouted rock bolts @ 1.5 m c/c in crown and sides 100 mm thick plain shotcrete in crown and sides with one layer of welded wire mesh of 100x100x4 mm
4	Class V	25 mm diameter fully grouted rock bolts @ 1.25 m c/c in crown and sides 100 mm thick plain shotcrete in crown and sides with one layer of welded wire mesh of 100x100x4 mm Steel Ribs of ISHB150 @ 0.5 m c/c

In addition to the above support system, drainage holes of 50 mm diameter, 4.0 m length will be provided as per requirement. Also, to avoid corrosion of the steel ribs, an additional layer of 50 mm thick plain shotcrete is proposed to be applied on the exposed area of the steel ribs i.e. for Class V rock.

7.1.28 Service Tunnels - Type B Service Tunnels

7.1.28.1 Specific Design Criteria

Please refer to Part A3 Chapter 5 of this report.

7.1.28.2 Design Methods Applied

Please refer to the Section Connecting Tunnel above.

7.1.28.3 Design Calculations

The design calculations included the numerical analysis of the tunnels and the wedge analysis.

Numerical Analysis of Service Tunnels Type B

Model Generation

A 2-D numerical analysis was carried out with the Phase 2 Finite Element programme. The proposed section of Type B Service was modelled as per their excavated profile. The external fixed boundary was considered as three times its dimension in each direction. Full face excavation of tunnels is proposed.

For generation of the model, an excavated boundary was created by importing a “DXF” file from AutoCAD. The external boundary was created by considering a box section with an expansion factor of 3 i.e. the external boundary was created as three times of its excavated width on both sides and top and bottom. The model was analysed considering three stages of excavation and support system.

A close up view of 2-D Service Tunnel Type B Model is shown in below figure.

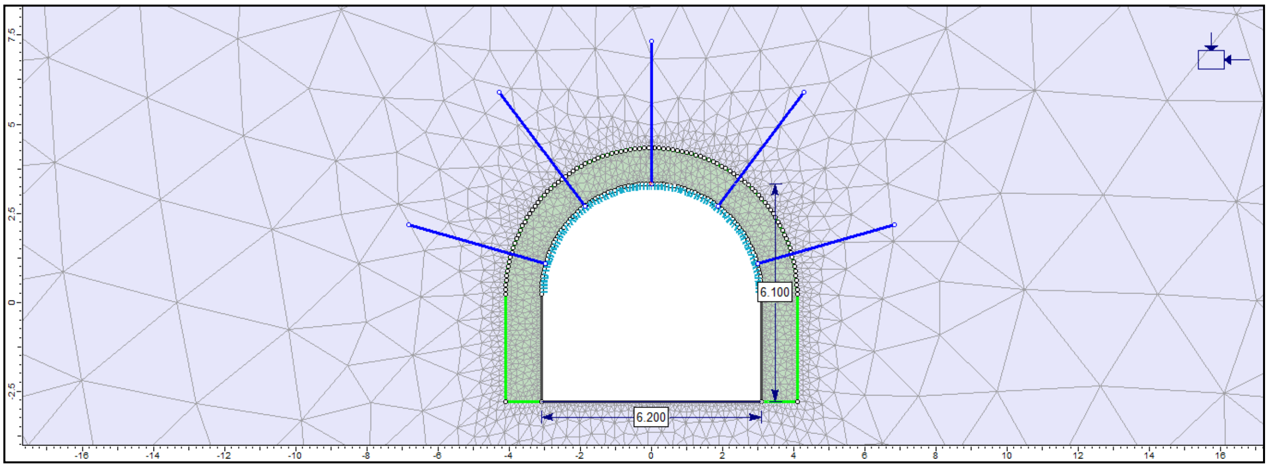

Figure 7-20: Close-up View of 2D Model of Type B Service Tunnel

The assumptions entering the model included the stages of excavation, the properties of the rock material, the loading details and the support system. These assumptions are presented in detail in Part A3 Chapter 5 of the report and are not reiterated here.

Results and Discussions

The type of analysis performed is plain strain analysis. Plain strain is defined as a state of strain in which the strain normal to the x-y plane (< z) and shear strain (< xz and < yz) are assumed to be zero. The details of various parameters determined after analysis are given below for each rock class.

Conclusion of Numerical Analysis

Based on the above results, it was concluded that the deformed shape of the model was as expected. As per this 2-D numerical analysis of the Service Tunnel, with the proposed support system, the maximum deformation for Type B Service Tunnels was about 4.3 mm for Class II rock, 8.3 mm for Class III rock, 18.3 mm for class IV rock and 38.6 mm for Class V rock, which is within the allowable limit. The details are shown in the annexes to Part A3 Chapter 5 of this report.

The maximum value of the yielded elements from the excavated face was about 1.0 m and the proposed rock bolt lengths up to a length of 4.0 m, which is a safe design. The details are shown in the annexes to Part A3 Chapter 5 of this report.

The number of yielded bolts are very small and hence the design is safe. The details are shown in the annexes to Part A3 Chapter 5 of this report.

As per the support capacity plots, only a limited portion of the shotcrete has a factor of safety less than 1.0 and only limited a portion of steel ribs has a factor of safety less than 1.5, which is acceptable. The shotcrete and the steel ribs contribute to a safe design. The details are shown in the annexes to Part A3 Chapter 5 of this report.

The recommended support system based on the above analysis is given below in the Subsection Results of Calculations.

Wedge Analysis of Type B Service Tunnels

In this study, the stability of the excavation profile of the underground Service Type B was checked against potential wedges that can develop based on identified structural discontinuities in the rock mass. The analysis for the Service Tunnels Type B was carried out considering the excavation geometry. The wedge stability and the influence of the proposed support was evaluated by the Unwedge software from Rocscience, which is based on a limit equilibrium analysis program. The analysis was carried out before and after the application of a support system.

The assumptions entering the model included the general input parameters, the joint properties and the support system. These assumptions are presented in detail in Part A3 Chapter 5 of the report and are not reiterated here.

Conclusion of Wedge Analysis

The analysis reveals 9 critical wedges for Service Tunnel Type B. The factor of safety (FOS) at various locations of the tunnels assumes greater importance given size of the Service Tunnels. Any wedge failure in the crown portion, in particular failure of unconfined wedges around it, could create crown instability of catastrophic proportions. Hence, a very conservative approach was adopted. Based on this analysis and based on known joint sets and assumed rock properties, 4.0 m rock bolts for Service Tunnel Type B contributes to the required FOS.

The observed Factor of Safety before and after installation of the support shows that, with the applied support, the potential wedges are safe. The critical wedges formed due to various joint combinations for and for Service Tunnel Type B are shown in the annexes to Part A3 Chapter 5 of this report.

7.1.28.4 Results of Calculations

Table 7-49: Proposed Support System

S.N.	Class of Rock	Proposed Support System
1	Class II	25 mm diameter fully grouted rock bolts @ 2 m c/c in crown only 50 mm thick plain shotcrete in crown only
2	Class III	25 mm diameter fully grouted rock bolts @ 1.75 m c/c in crown only 50 mm thick plain shotcrete in crown and sides
3	Class IV	25 mm diameter fully grouted rock bolts @ 1.5 m c/c in crown and sides 100 mm thick plain shotcrete in crown and sides with one layer of welded wire mesh of 100x100x4 mm
4	Class V	25 mm diameter fully grouted rock bolts @ 1.25 m c/c in crown and sides 100 mm thick plain shotcrete in crown and sides with one layer of welded wire mesh of 100x100x4 mm Steel Ribs of ISHB150 @ 0.75 m c/c

In addition to the above support system, drainage holes of 50 mm diameter, 4.0 m length will be provided as per requirement. Also, to avoid corrosion of the steel ribs, an additional layer of 50 mm thick plain shotcrete will be applied to the exposed steel ribs i.e. for Class V Rock.

7.1.29 Service Tunnels - Escape Tunnel

The design of the rock support system in the Escape Tunnel requires a detailed analysis of the in situ rock stresses, rock strength parameters application of rock bolts and shotcrete for rock support. The deformation modelling of the Powerhouse Escape Tunnel subjected to in situ stresses was studied for the analysis and design of the rock support. The 2-D numerical analysis has been carried out with the Phase^2 Finite Element Program for deformation and rock support modelling. The analyses and their results have been discussed in this report. Initially, the rock support system was estimated based on empirical relations. The details are presented below in this report.

7.1.29.1 Specific Design Criteria

Please refer to Part A3 Chapter 5 of this report.

7.1.29.2 Design Methods Applied

Please refer to the Section Connecting Tunnel above.

7.1.29.3 Design Calculations

The design calculations included the numerical analysis of the tunnel and the wedge analysis.

Numerical Analysis of Powerhouse Escape Tunnel

Model Generation

A 2-D numerical analysis was carried out with the Phase 2 Finite Element program. The proposed section of Escape Tunnel was modelled as per their excavated profile. The external fixed boundary was considered as three times its dimension in each direction. Full face excavation of tunnels is proposed.

For generation of the model, an excavated boundary was created by importing a “DXF” file from AutoCAD. The external boundary was created by considering a box section with an expansion factor of 3 i.e. the external boundary was created by three times of its excavated width on both sides and top and bottom. The model was analysed considering three stages of excavation and support system.

A close- up view of 2-D Powerhouse Escape Tunnel model is shown in below figure.

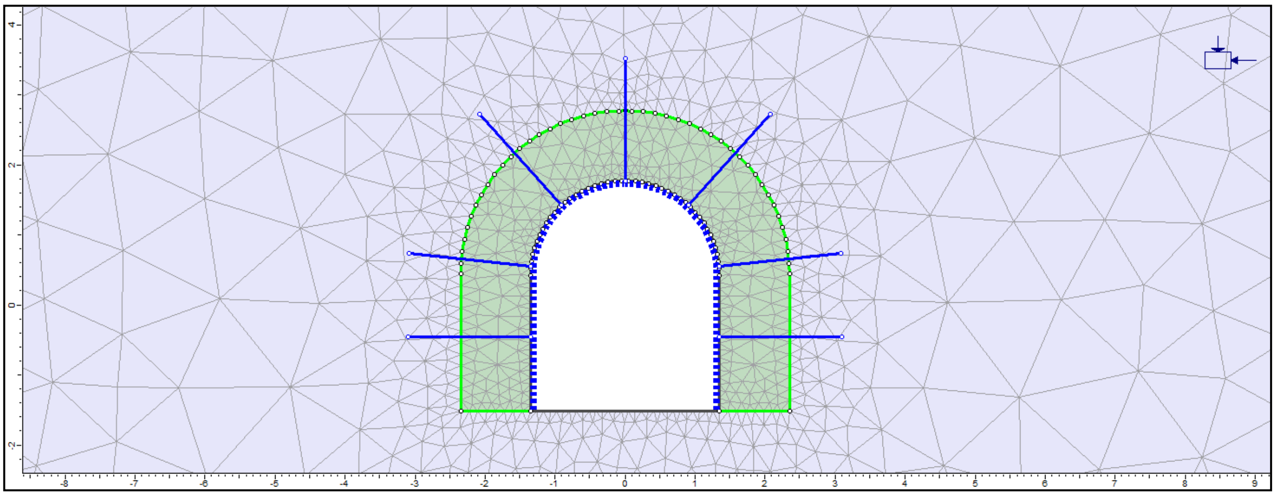

Figure 7-21: Close-up View of 2-D Model of Powerhouse Escape Tunnel

The assumptions entering the model included the stages of excavation, the properties of the rock material, the loading details and the support system. These assumptions are presented in detail in Part A3 Chapter 5 of the report and are not reiterated here.

Results and Discussions

The type of analysis performed is plain strain analysis. Plain strain is defined as a state of strain in which the strain normal to the x-y plane (< z) and shear strain (< xz and < yz) are assumed to be zero. The detail of various parameters determined after the analysis are given below.

Conclusion on Numerical Analysis

Based on above results, it was concluded that the deformed shape of the model was as expected. As per this 2-D numerical analysis of the Escape Tunnel, with the proposed support system, the maximum deformation was about 13.1 mm, which is within the allowable limit. The details are shown in the annexes to Part A3 Chapter 5 of this report.

The maximum limit of yielded elements from the excavated face was about 0.5 m and rock bolts are proposed up to a length of 1.75 m, which is safe. The details are shown in the annexes to Part A3 Chapter 5 of this report.

None of the rocks bolts yielded and hence the design is a safe design. The details are shown in the annexes to Part A3 Chapter 5 of this report.

As per support capacity plots, only a limited number of the shotcrete elements have a factor of safety less than 1.0 and none of the steel rib elements has a factor of safety less than 1.5, which is acceptable and the shotcrete and the steel ribs as designed are safe. The details are shown in the annexes to Part A3 Chapter 5 of this report.

The recommended support system based on above analysis is given below in the Subsection Results of Calculations.

Wedge Analysis of Escape Tunnel

In this study, the stability of the excavation profile of the Escape Tunnel was checked against potential wedges that can develop based on identified structural discontinuities in the rock mass. The analysis for the Escape Tunnel was carried out considering the excavation geometry. The wedge stability and the influence of the proposed support was evaluated with the Unwedge software from Rocscience, which is based on a limit equilibrium analysis program. The analysis was carried out before and after application of the support system.

The assumptions entering the model included the general input parameters, the joint properties and the support system. These assumptions are presented in detail in Part A3 Chapter 5 of the report and are not reiterated here.

Results of Unwedge Analysis

Based on combinations of all joints, the summary of results for wedge stability analysis before and after support installation is presented below.

Table 7-50: Summary of Results for Wedge Stability Analysis

S.N.	Joint Combinations	Critical Failure Wedges	Wedge Weight (kN)	Excavated Face Area (m^2)	Factor of Safety (Without support)	Factor of Safety (With Support)
1	J2-J3-J4	Upper Right Wedge (8)	0.02	0.03	0.000	1230
2	J1-J3-J4	Upper Left Wedge (6)	9.15	2.16	1.485	107
3	J1-J3-J4	Roof Wedge (8)	0.01	0.05	0.000	1348
4	J1-J2-J3	Upper Left Wedge (5)	36.99	4.88	0.834	53.478
5	J1-J2-J3	Upper Right Wedge (8)	0.54	0.38	0.000	293
6	J1-J2-J4	Roof Wedge (8)	0.09	0.10	0.000	1106

Conclusion

The analysis reveals the presence of 6 critical failure wedges. The factor of safety (FOS) at various locations of the Escape Tunnel assumes greater importance given the size of the Escape Tunnel. Any wedge failure in the crown portion, in particular failure of unconfined wedges around it, could create crown instability of serious significance. Hence, a very conservative approach was adopted. Based on this analysis and based on known joint sets and assumed rock properties, 1.75 m rock bolts achieve the required FOS.

The calculated factor of safety before and after the installation of the support shows that with the applied support, the potential wedges are safe. The critical wedges formed due to various joint combinations are shown in the annexes to Part A3 Chapter 5 of this report.

7.1.29.4 Results of Calculations

Table 7-51: Proposed Support System

S.N.	Class of Rock	Proposed Support System
1	Class V	25 mm diameter fully grouted rock bolts of 1.75 m length @ 1.0 m c/c in crown and sides 100 mm thick plain shotcrete in Crown and sides with one layer of 100x100x4 mm wire mesh Steel Ribs of ISMB150 @ 1.0 m c/c spacing

7.1.30 Service Tunnels - Tunnel Portals

The objective of this section is the analysis, the design and determination of the stability of the Service Tunnel Portals and the determination of the required excavated profile and support system, if any.

7.1.30.1 Specific Design Criteria

Please refer to Part A3 Chapter 5 of this report.

7.1.30.2 Design Methods Applied

For simplicity of analysis, the large number of portals were grouped together according to the tunnel support system, i.e. Type A and Type B. and according to their size. The most critical case among these types were selected for analysis and a generalization was made for the purpose of determining the required cut slope angle and the required rock support. The most critical case was considered the one with the weakest rock and highest height of cut.

The type of analysis performed is plain strain analysis. plain strain is defined as a state of strain in which the strain normal to the x-y plane (< z) and shear strain (< xz and < yz) are assumed to be zero.

7.1.30.3 Design Calculations

The assumptions entering the model included the stages of excavation, the properties of the rock material, the loading details and the support system. These assumptions are presented in detail in Part A3 Chapter 5 of the report and are not reiterated here.

2-D slope stability analysis was carried out with the Phase2 Software based on a shear strength reduction approach. The proposed excavated section for the critical cases for the Service Tunnel Portals were modelled at the portal location. The stability calculations were carried out with and without support systems and include / exclude seismic loads. The excavated slopes modelled with the Phase2 software are given in the figures below**.**

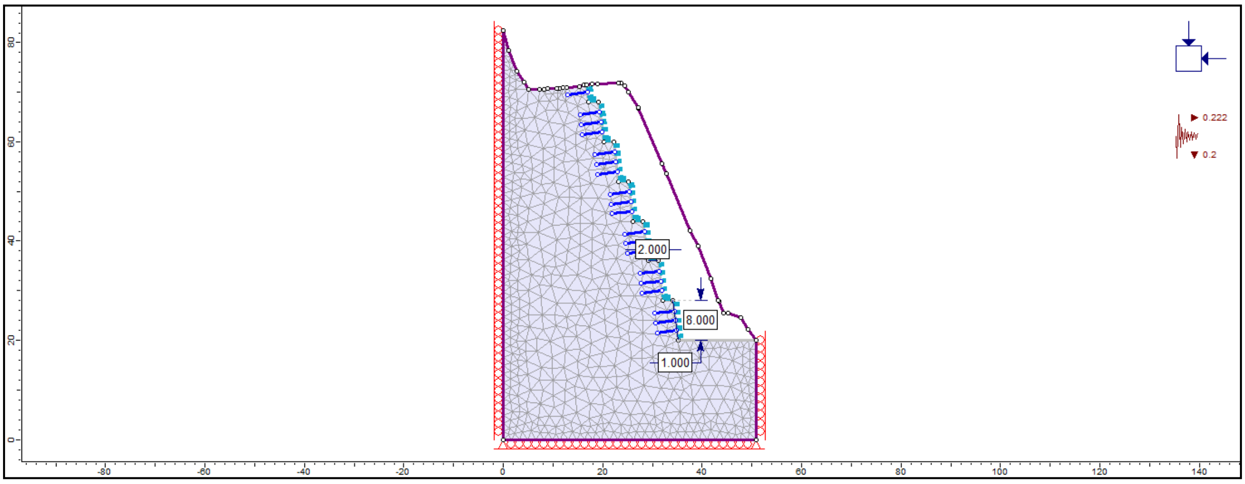

Figure 7-22: Excavated Slope of Power House Cable and Ventilation Tunnel Portal Modelled in Phase2 Software Generalized for Type A Tunnel Portals

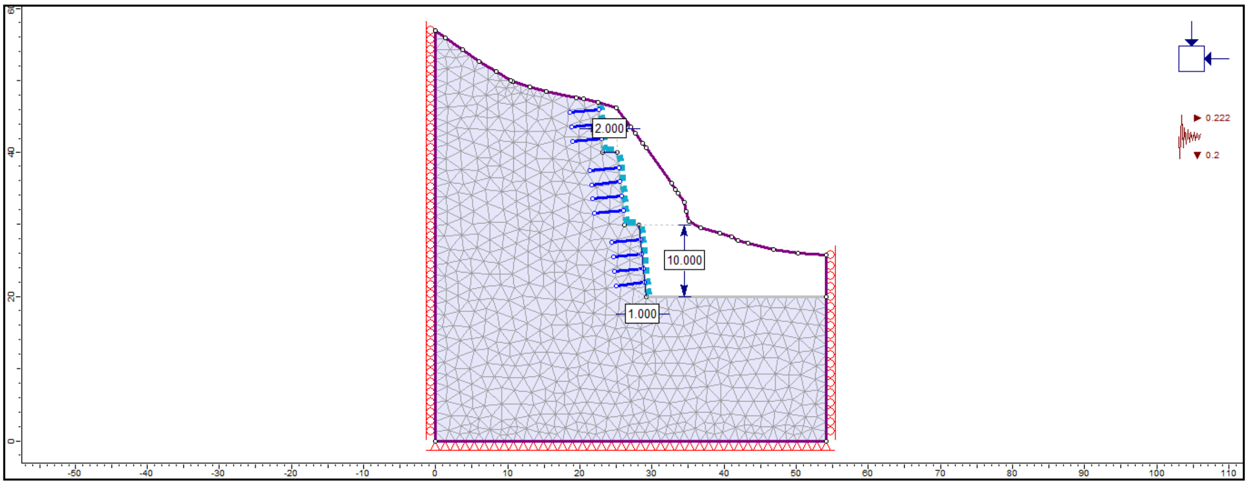

Figure 7-23: Excavated Slope of Main Access Tunnel Portal Modelled in Phase2 Software Generalized for Type B Service Tunnels

7.1.30.4 Results of Calculations

<u>Type A Service Tunnel Portals</u>

The proposed excavated slopes and support for was designed with a slope of 8V:1H with a 2m wide bench at a height of 8 m. The excavated slope is protected with 100 mm thick shotcrete with one layer of wire mesh of 100x100x4 mm and 4m long rock bolts at @ 2 m c/c staggered.

<u>Type B Service Tunnel Portals</u>

The proposed slopes and support for were designed with a slope of 10V:1H with a 2m bench at a height of 10 m. The excavated slope is protected with 50 mm thick shotcrete and 4m long rock bolts at @ 2 m c/c staggered. The excavated section for the slope stability analysis was considered accordingly. Based on results of the slope stability analysis, the summary of the factors of safety calculated under various scenarios is given in the annexes to Part A3 Chapter 5 of this report.

Table 7-52: Summary of Factors of Safety

S.N.	Case	Condition	Factor of Safety (Without Support System)	Factor of Safety (With Support System)
1	Case-1 (Type A)	Normal Case	1.77	1.81
2	Case-2 (Type A)	Seismic Case	1.21	1.23
3	Case-1 (Type B)	Normal Case	2.15	2.32
4	Case-2 (Type B)	Seismic Case	1.40	1.49

Based on the above results, it is concluded that the proposed design for the slopes and the corresponding support systems are safe. The minimum factor of safety with the designed support system is 1.5 under normal condition and 1.2 under seismic conditions, which reflects a safe design.

7.2 Structural Design

7.2.1 Headworks - General

This chapter covers the analysis and design of:

5) the Spillway Weir,

6) the Spillway Tunnel Portal Structure and

7) the Spillway Terminal Structure d/s of Tunnel Portal.

7.2.2 Headworks - Connecting Tunnel

No structural analysis or design was required. For rock support see Rock Mechanics Subchapter.

7.2.3 Headworks - Headpond

No structural analysis or design was required. For rock support, including concrete liner, see Rock Mechanics Subchapter.

7.2.4 Headworks - Spillway Weir

Stability Analysis & Design of Spillway Weir

In the event of a load rejection at one of the Tamakoshi V HEP units, there is a possibility of waves travelling back to the Headpond and from there into the TRT of the Upper Tamakoshi HEP. To prevent this, a Spillway Weir has been foreseen and designed. The Spillway Weir has a length of 55.50 m, a height of 5.10 m and was positioned just upstream of the HRT of Tamakoshi V HEP so that the water can spill over the spillway weir into the 5.0m wide and 55.5m long collecting channel and, subsequently, flow into the Spillway Tunnel, through the Spillway Terminal Structure and finally returned to the Tamakoshi River. This section covers the stability and design of the Spillway Weir.

7.2.4.1 Specific Design Criteria

Input Data for Stability & Design

The Input data available for the stability & design are given in the table below.

Table 7-53: Available Input Data for Design of Spillway Weir

Parameter	Value	Unit
Length	55.50	m
Alignment w.r.t flow of water into HRT (Horizontal)	6	Degree
Crest level	1158.20	El. m
Foundation level	1152.60	El. m
Headpond at NOWL	1158.00	El. m
Headpond at HOWL (load rejection)	1159.50	El. m
Grade of Concrete (Class A, XC2)	C20/25	
Horizontal seismic coefficient (OBE)	0.22	g
Vertical seismic coefficient (OBE)	0.20	g
Horizontal seismic coefficient (MCE)	0.34	g
Vertical seismic coefficient (MCE)	0.31	g
Unit weight of concrete	23.00	KN/m^3
Unit weight of rock-soil debris	22.00	KN/m^3
Angle of internal friction (ϕ)	31.80	Degree
Cohesion between rock & concrete	260	KN/m^2
Bearing capacity of rock	1000	KN/m^2
Allowable compressive stress of concrete	6000	KN/m^2

Design Assumptions

- It was assumed that silt free water enters the Headpond of Tamakoshi V HEP;
- Transverse contraction joints are neither keyed nor grouted. Individual weir blocks will transfer its entire load to the foundation acting as independent elements, without transferring any part of the load to adjacent blocks;
- The rock, that constitutes the foundation and abutments, is strong enough to carry the forces imposed by the weir with stresses well within the elastic limit at all places along the contact planes;
- The base of the weir is thoroughly keyed into the rock formations along the foundation and abutments;
- The concrete in the weir is a homogeneous, isotropic and uniform elastic material;
- Earthquake inertial forces in horizontal and vertical direction were considered in accordance with international practices;
- Only the slip path between the contact surface of the weir base with the rock foundation was considered; and
- Construction operations will be conducted so as to secure a satisfactory bond between the concrete and rock materials at all areas of contact along the foundation and abutments.

7.2.4.2 Results of Calculations

From the results of the calculations it is seen that the factor of safety against sliding is greater than the required 1.5 (1.0 for seismic load combinations D and E) in all the load combinations and, hence, the weir design is safe against sliding. To consider the construction stages of the project and to be on the conservative side, rock anchors of 4 m length (2.5 m into the foundation rock and 1.5m into the concrete weir) and 25 mm dia @ 2m c/c and arranged staggered, are proposed in the bottom slab of the weir. During the detailed design stage, the above requirement shall be reviewed based on the foundation conditions at the weir location. Also, all the compressive and tensile stresses are within permissible limits and, therefore, the weir's design is safe against sliding.

Reinforcement Details for Controlling Shrinkage and Early Age Thermal Cracks

The spillway weir structure is a mass concrete structure and it is not required to design the structure as a structural member. However, to resist shrinkage and early age thermal cracks, an adequate amount of steel reinforcement was provided on the downstream and upstream surfaces of the weir body. Based on the calculations, a reinforcement of 16 mm dia @ 200 mm c/c, vertically & horizontally on both surfaces of the Spillway Weir was calculated.

7.2.5 Headworks - Spillway Tunnel Portal

It was proposed to construct the Spillway Tunnel Portal with reinforced concrete around the periphery of the proposed tunnel portal. The main function of the Spillway Tunnel Portal is to provide a well-defined and safe access to the Spillway Tunnel.

7.2.5.1 Specific Design Criteria

The following assumptions were made for the design of the Spillway Tunnel Portal.

- The rock load was calculated assuming a 45° dispersion of the rock load above the portal beam;
- A superimposed load of 2.0 m height of soil-rock debris over the portal beam was assumed to account for an additional load due to rock slides, if any;
- The unit weight of the rock was considered as 27 kN/m^3;
- The unit weight of loose soil-rock debris was considered as 22 kN/m^3;
- For the earthquake load case was a rock load, 50% of the superimposed load (rock fall) was considered together with the self-weight of the portal; and
- Safe bearing capacity of the foundation material was assumed as 100 t/ m^2 (1000KN/m2).

Other assumptions and input data made for the analysis are presented in Part A3 Chapter 1 of this report and are not reiterated here.

7.2.5.2 Results of Calculations

<u>Design of the Spillway Portal Structure Members</u>

Frame the above results, members of the Spillway Portal Structure (beam, columns and footings) were designed. The details can be taken from Part A3 Chapter 1 of this report.

7.2.6 Headworks - Spillway Terminal Structure

Excess water in the Headpond is spilled over the Spillway Weir and the spilled water is collected in a collecting channel which is positioned just downstream of the Spillway Weir. From the collecting channel, water is carried to the River Tamakoshi through the Spillway Tunnel. At the outlet of the tunnel, the Spillway Terminal Structure, of about 35 m length, was designed to accommodate the maintenance & service gates. On top of the structure a public access road and a gate operation building was planned. The Spillway Terminal Structure was proposed to be executed in two phases. In Phase 1, the excavation and concreting of the D/S part of the Spillway Terminal Structure (which accommodates the public access road and the service gate arrangement) was planned and in Phase 2, the excavation of tunnel portal and concreting of U/S part of the Spillway Terminal Structure (which accommodates the service gate and at top Operation Building) completes the structure.

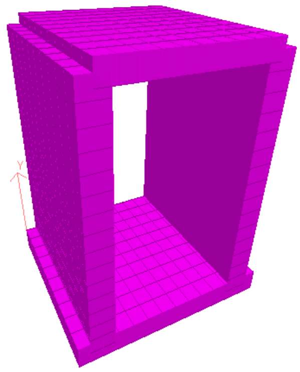

Figure 7-24: 3-D View of the Spillway Terminal Structure

This section covers the structural design of the heavily loaded part of the Spillway Terminal Structure, specifically, the D/S of part of the Spillway Terminal Structure.

7.2.6.1 Specific Design Criteria

The criteria, assumptions and input data made for the analysis are presented in Part A3 Chapter 1 of this report and are not reiterated here.

Analysis and Results

After the analysis, the results were interpreted component wise and the designs have been done using spread sheets prepared in Excel.

Top Slab:

The top slab was modelled with centerline dimensions using finite element modelling. The thickness of the slab required from the shear point of view was 800 mm. The analysis results are given below. Based on the results, the slab was designed for the maximum moments and shear. The detailed design calculations and the required reinforcement for the top slab are given in the annexes to Part A3 Chapter 1 of this report.

Table 7-54: Summary of Plate Results for Top Slab

	Plate	L/C	Shear		Membrane			Moments		
			SQX (local) N/mm^2	SQY (local) N/mm^2	SX (local) N/mm^2	SY (local) N/mm^2	SXY (local) N/mm^2	Mx kNm/m	My kNm/m	Mxy kNm/m
Max Qx	509	6 DL+LL+EP	0.31	0.06	-0.46	-0.05	0.02	478.91	60.82	-12.76
Min Qx	500	6 DL+LL+EP	-0.31	0.06	-0.46	-0.05	-0.02	478.91	60.82	12.76
Max Qy	511	6 DL+LL+EP	-0.22	0.09	-0.44	-0.03	0.00	478.13	26.24	11.05
Min Qy	412	6 DL+LL+EP	-0.22	-0.09	-0.44	-0.03	0.00	478.13	26.24	-11.05
Max Sx	428	8 DL+LL+EP(NO SURCHARGE)+WP	0.22	-0.01	-0.30	0.00	0.01	249.31	9.33	1.06
Min Sx	439	6 DL+LL+EP	0.31	-0.06	-0.46	-0.05	-0.02	478.91	60.82	12.76
Max Sy	502	8 DL+LL+EP(NO SURCHARGE)+WP	-0.15	0.01	-0.32	0.00	0.00	148.27	10.90	2.14
Min Sy	500	6 DL+LL+EP	-0.31	0.06	-0.46	-0.05	-0.02	478.91	60.82	12.76

	Plate	L/C	Shear		Membrane			Moments		
			SQX (local) N/mm^2	SQY (local) N/mm^2	SX (local) N/mm^2	SY (local) N/mm^2	SXY (local) N/mm^2	Mx kNm/m	My kNm/m	Mxy kNm/m
Max Sxy	509	6 DL+LL+EP	0.31	0.06	-0.46	-0.05	0.02	478.91	60.82	-12.76
Min Sxy	500	6 DL+LL+EP	-0.31	0.06	-0.46	-0.05	-0.02	478.91	60.82	12.76
Max Mx	500	6 DL+LL+EP	-0.31	0.06	-0.46	-0.05	-0.02	478.91	60.82	12.76
Min Mx	474	8 DL+LL+EP(NO SURCHARGE)+WP	-0.03	0.00	-0.31	0.00	0.00	47.56	1.84	0.14
Max My	470	6 DL+LL+EP	-0.29	0.00	-0.41	-0.02	0.00	470.50	76.69	1.29
Min My	504	8 DL+LL+EP(NO SURCHARGE)+WP	-0.03	0.00	-0.31	0.00	0.00	50.07	-0.93	-0.02
Max Mxy	501	6 DL+LL+EP	-0.23	0.02	-0.46	0.00	-0.01	347.57	34.25	12.93
Min Mxy	508	6 DL+LL+EP	0.23	0.02	-0.46	0.00	0.01	347.57	34.25	-12.93

Side Walls:

Side walls were modeled as finite elements as per center line dimensions. The thickness of the plate elements for the abutments is 1.0 m. Since the two side walls are similar to each other, one of the side wall i.e. right wall was designed from the STADD.Pro V8i results. Since the design axial load on the wall is less than 0.04fckAg, the wall is designed as a slab (refer clause 32.3.2 of IS 456: 2000). Summary of the analysis results for right side wall is given in the table below. The design of wall is given in the annexes to Part A3 Chapter 1 of this report.

Table 7-55: Summary of Plate Results for Right Side Wall

Summary of Plate Results for Right Side Wall										
			Shear		Membrane			Bending Moment		
	Plate	L/C	SQX (local) N/mm^2	SQY (local) N/mm^2	SX (local) N/mm^2	SY (local) N/mm^2	SXY (local) N/mm2	Mx kNm/m	My kNm/m	Mxy kNm/m
Max Qx	23	6 DL+LL+EP	0.09	-0.71	0.01	-0.60	0.01	56.45	896.97	-10.89
Min Qx	41	6 DL+LL+EP	-0.09	-0.71	0.01	-0.60	-0.01	56.45	896.97	10.89
Max Qy	197	6 DL+LL+EP	0.02	0.36	0.02	-0.22	-0.03	16.19	342.73	19.04
Min Qy	39	6 DL+LL+EP	-0.03	-0.72	-0.02	-0.66	0.01	124.64	892.55	13.02
Max Sx	197	6 DL+LL+EP	0.02	0.36	0.02	-0.22	-0.03	16.19	342.73	19.04

Summary of Plate Results for Right Side Wall										
			Shear		Membrane			Bending Moment		
	Plate	L/C	SQX (local) N/mm^2	SQY (local) N/mm^2	SX (local) N/mm^2	SY (local) N/mm^2	SXY (local) N/mm2	Mx kNm/m	My kNm/m	Mxy kNm/m
Min Sx	37	8 DL+LL+EP(NO SURCHARGE)+WP	0.00	-0.53	-0.03	-0.64	0.02	118.40	722.57	5.91
Max Sy	197	6 DL+LL+EP	0.02	0.36	0.02	-0.22	-0.03	16.19	342.73	19.04
Min Sy	39	6 DL+LL+EP	-0.03	-0.72	-0.02	-0.66	0.01	124.64	892.55	13.02
Max Sxy	214	6 DL+LL+EP	-0.02	0.36	0.02	-0.22	0.03	16.19	342.73	-19.04
Min Sxy	197	6 DL+LL+EP	0.02	0.36	0.02	-0.22	-0.03	16.19	342.73	19.04
Max Mx	33	6 DL+LL+EP	0.00	-0.69	-0.03	-0.63	0.00	153.30	873.28	1.76
Min Mx	118	6 DL+LL+EP	0.00	0.01	0.00	-0.45	0.00	-68.13	-642.30	0.39
Max My	41	6 DL+LL+EP	-0.09	-0.71	0.01	-0.60	-0.01	56.45	896.97	10.89

Summary of Plate Results for Right Side Wall										
			Shear		Membrane			Bending Moment		
	Plate	L/C	SQX (local) N/mm^2	SQY (local) N/mm^2	SX (local) N/mm^2	SY (local) N/mm^2	SXY (local) N/mm2	Mx kNm/m	My kNm/m	Mxy kNm/m
Min My	123	6 DL+LL+EP	0.02	0.01	0.00	-0.45	0.00	-3.80	-659.83	0.98
Max Mxy	65	6 DL+LL+EP	0.00	-0.53	0.00	-0.56	-0.01	10.78	79.27	39.19
Min Mxy	55	6 DL+LL+EP	0.00	-0.53	0.00	-0.56	0.01	10.78	79.27	-39.19

<u>Bottom Slab:</u>

Bottom Slab was modeled as finite elements as per centre line dimensions. The thickness of the plate elements for the bottom slab was 1.00 m. Summary of the analysis results for bottom slab is given in below table. The design of bottom slab is given in the annexes to Part A3 Chapter 1 of this report.

Table 7-56: Summary of Plate Results for Bottom Slab

Summary of Plate Results for Bottom Slab of Spillway Terminal Structure										
			Shear		Membrane			Moments		
	Plate	L/C	SQX (local) N/mm^2	SQY (local) N/mm^2	SX (local) N/mm^2	SY (local) N/mm^2	SXY (local) N/mm^2	Mx kNm/m	My kNm/m	Mxy kNm/m
Max Qx	618	6 DL+LL+EP	0.78	-0.02	-0.79	-0.03	-0.02	-831.27	-123.63	-0.49
Min Qx	627	6 DL+LL+EP	-0.78	-0.02	-0.79	-0.03	0.02	-831.27	-123.63	0.49
Max Qy	530	6 DL+LL+EP	0.75	0.07	-0.73	0.02	-0.01	-820.05	-73.13	-0.36
Min Qy	629	6 DL+LL+EP	0.75	-0.07	-0.73	0.02	0.01	-820.05	-73.13	0.36
Max Sx	707	6 DL+LL+EP	-0.11	-0.04	0.01	-0.03	-0.01	46.05	42.04	15.32
Min Sx	557	6 DL+LL+EP	-0.78	0.02	-0.79	-0.03	-0.02	-831.27	-123.63	-0.49
Max Sy	691	6 DL+LL+EP	-0.09	0.00	-0.01	0.08	-0.01	36.18	4.10	-1.15

Summary of Plate Results for Bottom Slab of Spillway Terminal Structure										
			Shear		Membrane			Moments		
	Plate	L/C	SQX (local) N/mm²	SQY (local) N/mm²	SX (local) N/mm²	SY (local) N/mm²	SXY (local) N/mm²	Mx kNm/m	My kNm/m	Mxy kNm/m
Min Sy	598	6 DL+LL+EP	0.76	0.00	-0.77	-0.04	-0.01	-817.90	-139.10	-1.40
Max Sxy	703	6 DL+LL+EP	-0.10	-0.02	0.01	0.04	0.03	45.14	23.49	4.30
Min Sxy	682	6 DL+LL+EP	-0.10	0.02	0.01	0.04	-0.03	45.14	23.49	-4.30
Max Mx	639	8 DL+LL+EP(NO SURCHARGE)+WP	-0.08	0.02	-0.56	0.00	0.00	295.73	-1.93	-4.85
Min Mx	618	6 DL+LL+EP	0.78	-0.02	-0.79	-0.03	-0.02	-831.27	-123.63	-0.49
Max My	593	8 DL+LL+EP(NO SURCHARGE)+WP	-0.07	0.00	-0.59	0.01	0.00	289.81	45.77	-0.07
Min My	588	6 DL+LL+EP	0.76	0.00	-0.77	-0.04	-0.01	-815.06	-139.62	-0.60
Max Mxy	546	8 DL+LL+EP(NO SURCHARGE)+WP	-0.55	0.02	-0.54	0.00	0.01	-277.94	-18.86	20.01

Summary of Plate Results for Bottom Slab of Spillway Terminal Structure										
			Shear		Membrane			Moments		
	Plate	L/C	SQX (local) N/mm^2	SQY (local) N/mm^2	SX (local) N/mm^2	SY (local) N/mm^2	SXY (local) N/mm^2	Mx kNm/m	My kNm/m	Mxy kNm/m
Min Mxy	532	8 DL+LL+EP(NO SURCHARGE)+WP	0.55	0.02	-0.54	0.00	-0.01	-277.94	-18.86	-20.01

7.2.6.2 Results of Calculations

Summary of Reinforcement

Based on the analysis and design, experience on the similar projects and constructability, the required reinforcement for the Spillway Terminal Structure is given in the table below.

Table 7-57: Summary of Reinforcement for Spillway Terminal Structure

Component	Top		Bottom		Vertical		Horizontal	
	Flow direction	Traffic direction	Flow direction	Traffic direction	Outer	Inner	Outer	Inner
Top Slab (El.1160.50 m)	20 @200 c/c	20 @ 150 c/c	20 @ 200 c/c	20 @ 200 c/c				
Bottom Slab (El.1150.00)	20 @ 125 c/c	20 @ 175 c/c	20 @ 200 c/c	20 @ 200 c/c				
Side Walls					25 @125 c/c	25@175 c/c	20 @200 c/c	20@200c/c

Check for Limit State of Serviceability

Vertical Deflection

As per clause 23.2 IS 456-2000, the deflection of a slab shall generally be limited to the span/250. Maximum deflection of 2.27 mm was calculated for service load conditions under load case 10 which is quite less than (L/250 = 5400/250 =21.6 mm). Hence, limit state of serviceability is satisfied.

Check for Lateral in Side Walls

As per clause 23.3 of IS 456, the lateral sway should not exceed span/250. Maximum deflection of 1.826 mm was calculated for service load case 9 which is much less than (9500/250) 38 mm. Hence, limit state of serviceability is satisfied.

7.2.7 Power Waterways - Valve Chamber Crane Support System

The Valve chamber is located just downstream of Surge tank. The Valve Chamber is designed to accommodate the U/S valve of high pressure water conveyance system.

This section covers the structural design of the crane supporting structures in the U/S Valve Chamber.

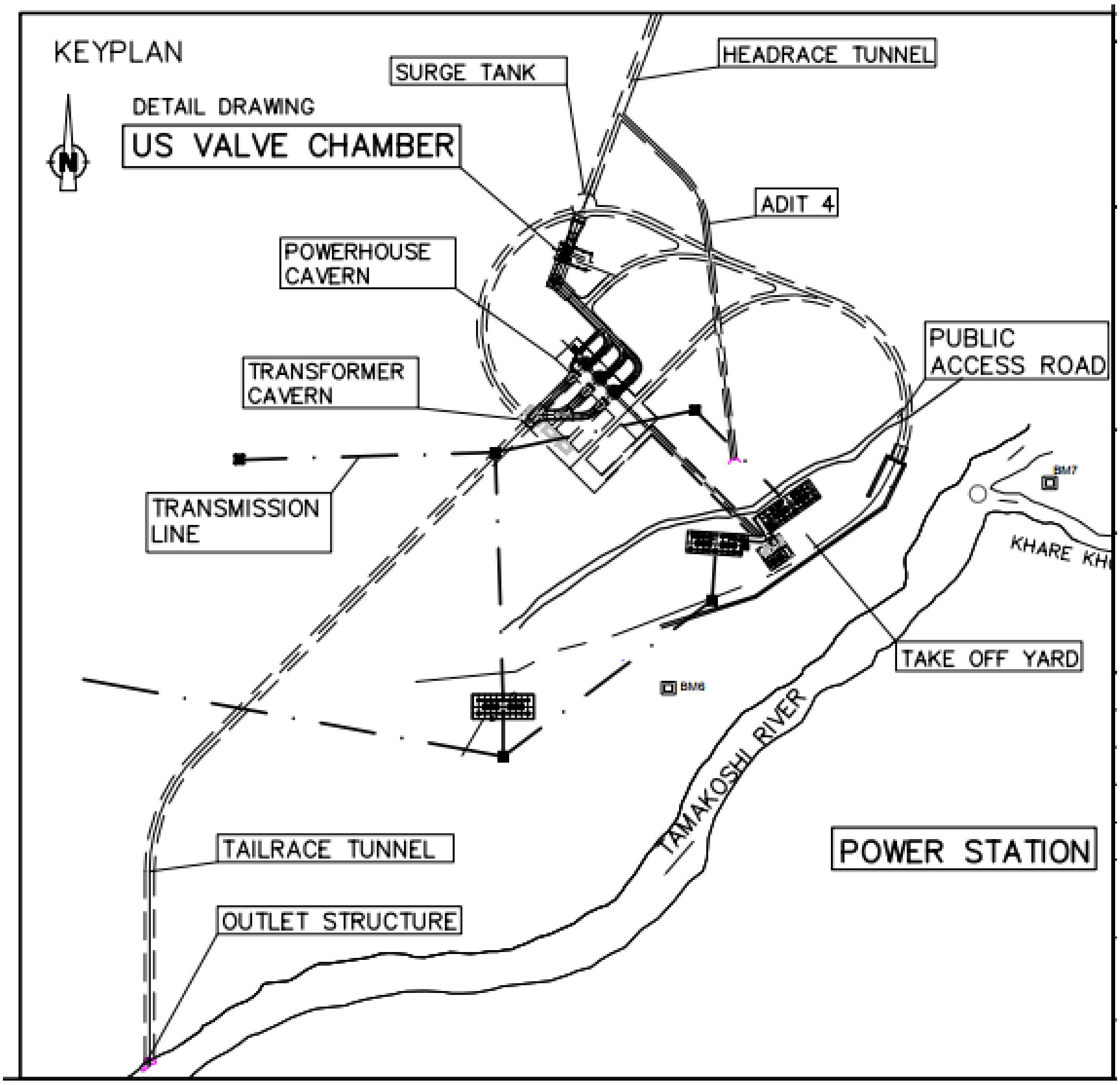

Figure 7-25: Sketch of the plan of US valve chamber and surrounding structures

7.2.7.1 Specific Design Criteria

An electric overhead travelling (EOT) crane was planned in the valve chamber for installation and maintenance of the U/S Valve. The EOT crane will be operated on steel rails which are supported by reinforced concrete support beams. The beams are supported by columns which ultimately transfers the loads from the EOT crane to the foundation.

The crane supporting beams are supported on three pairs of columns of equal spans. Identical frames of two sides are supported on a 1m thick concrete raft which is founded on a rock foundation. Two crane supporting frames are connected with framing beam at the ends at crane beam level to effectively limit lateral deflection.

The function of the crane support system is to provide proper support to operate the EOT crane during installation and maintenance of the U/S Valve. It is designed to carry the crane load with maximum weight that the crane can handle. The following assumptions are made for the crane support system.

- Crane support systems on two ends of the crane will not be connected and act independently.
- The crane will rarely be operated, and the effect of earthquake and handling of maximum weights were not superimposed.
- The support beam will not deform axially, hence longitudinal load due to crane is applied at a point
- The earthquake load has been calculated considering weight of crane girder with crab; and
- Safe bearing capacity of the foundation material is assumed as 800kN/ m^2 and soil subgrade reaction 80,000 KN/m/m^2.

A 3-D frame model was prepared which also included a raft to simulate the foundation. Compression springs of 80,000 KN/m/m^2 were applied to the foundation. The model was loaded with the moving crane loads. A seismic load was applied based on the response spectrum method using the function based on IS 1893-2016 and scaled to the desired seismic load based on a concentrated mass on the structure. Fifty percent of the maximum credible earthquake, as per seismological investigation report, was considered for this underground structure. The dead weight of the columns and beams are concentrated at the beam-column junctions. To investigate the maximum response to the central column, the inertial weight of the crane at its maximum eccentric position (laterally) was considered (minus the weight to lift).

The model considered the eccentricity of the column and crane beam centers automatically as the crane beam was modeled with 200 mm offsets to column centerline.

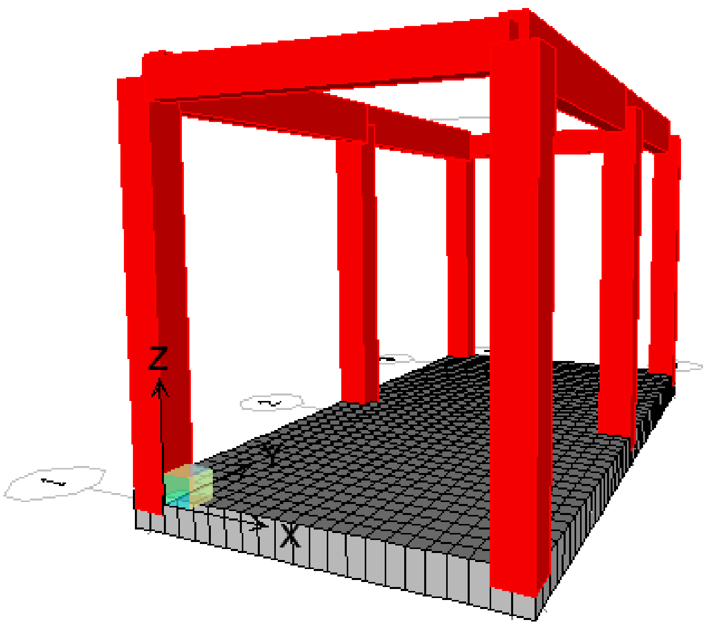

Figure 7-26: SAP model of Frame

Other assumptions and input data made for the analysis are presented in Part A3 Chapter 2 of this report and are not reiterated here.

7.2.7.2 Results of Calculations

Design of Beam

The design of beam was performed directly with SAP 2000 based on IS 456: 2000 standard. The results of the design are as follows:

- Size of Crane support beam (B x D) = 0.6 m x 1.4 m
- Required maximum longitudinal reinforcement area = 3146 mm^2
- Provided longitudinal reinforcement Top 10 nos, 20 mm dia.

  Bottom 10 nos, 20 mm dia.
- Required Shear reinforcement area 2526 mm^2/m
- Provided Shear Reinforcement 4- legged 12 mm dia. 200 mm C/C
- Side face Reinforcement 4 nos, 20 mm dia. At each face
- Size of End beam (B x D) 0.6 m x 1.4 m
- Required Maximum Longitudinal Reinforcement area = 1776 mm^2
- Provided longitudinal reinforcement Top 6 nos, 20 mm dia.

  Bottom 6 nos, 20 mm dia.
- Required Shear reinforcement area 665 mm^2/m
- Provided Shear Reinforcement 4- legged 10 mm dia. 250 mm C/C
- Side face Reinforcement 3 nos, 20 mm dia. At each face

Design of Columns

- Size: 1 m x 1 m
- Reinforcement:
- Longitudinal:16 nos, 28 mm dia.
- Stirrups plus intermediate ties: 12 mm dia. @ 200 mm C/C

<u>Design of Slab</u>

- Slab thickness: 1 m
- Reinforcement Generally: 16 mm dia. @ 200 mm C/C both ways
- Under column area: 3.0 m x 3 m
- Provide additional 16 mm dia. bars at 200 mm C/C both ways.
- For edge column effective area = 2.5 m x 2.5 m
- Provide additional 16 mm dia. Bars at 200 mm C/C on the effective area.
- No shear reinforcement required

7.2.8 Outlet Structure - Outlet Tunnel with Portal and Tailbay

This section presents the analysis and design of the Outlet Tunnel with Portal & Tailbay. The deepest section has a length of 6 m. As per the topography and site investigations the right bank of the Outlet Tunnel with Portal & Tailbay consists of rock and the left bank consists of overburden material. After construction of the Outlet Tunnel with Portal & Tailbay, earth backfill will be placed behind the left wall, whereas, behind the right wall, concrete backfill was proposed and, hence, no earth pressure acts on right wall of the Outlet Tunnel with Portal & Tailbay. To release the uplift pressure, weep holes were designed in the slab of the Outlet Tunnel with Portal & Tailbay.

This section covers the structural design of deepest U-Shaped Outlet Tunnel with Portal & Tailbay section. The section considered is 7 m from the portal axis. A section width of 2 m (in flow direction) was considered for the analysis and design.

7.2.8.1 Specific Design Criteria

The geometric design parameters, material properties and other input data can be taken from Part A3 Chapter 3 and Part E Album of Drawings of this report. They are not repeated in this place.

7.2.8.2 Design Method Applied

Deepest cross section of Terminal Structure & Tailbay was modeled with 3-D plate elements in the STADD. Pro V8i software and the critical load cases were considered for the analysis. The design of the components was done using an Excel spreadsheet. The Terminal Structure & Tailbay was analyzed as a 3-D structure modelled along the centerline line. Compression spring supports were provided to the bottom slab to represent the rock concrete interface interactions. A 3-D view of a section of the Terminal Structure & Tailbay is given in below figure.

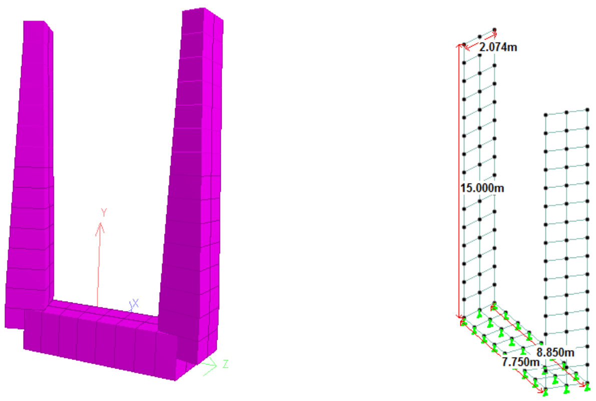

Figure 7-27: 3D View of Outlet Structure Tailbay

The assumptions entering the model included the stages of excavation, the properties of the rock material, the loading details and the support system. These assumptions are presented in detail in Part A3 Chapter 3 of the report and are not reiterated here.

Results of the Analysis - Summary of Reinforcement

Based on the analysis and design, experience on the similar projects and the construction point of view, the required reinforcement for deepest section of Terminal Structure & Tailbay is given in the below Subsection Results of Calculations. The detailed calculations for the reinforcement is given in the annexes to Part A3 Chapter 3 of this report.

Table 7-58: Summary of Reinforcement Details for Deepest section of Terminal Structure & Tailbay of Outlet Structure

Component	Top		Bottom		Vertical		Horizontal	
	Flow direction	Perpendicular to flow direction	Flow direction	Perpendicular to flow direction	Outer	Inner	Outer	Inner
Bottom Slab (Centre line of slab to till end of slab on left side)	25 @ 125 c/c	25 @ 150 c/c (2 rows)	25@ 125 c/c	32 @ 150 c/c (2 rows)				
Bottom Slab (Centre line of slab to till end of slab on right side)	25 @ 125 c/c	25 @ 150 c/c	25 @ 125 c/c	32 @ 150 c/c				
Left Side Wall (Bottom 50% height of wall)					32 @125 c/c (2 rows)	25 @125 c/c	25 @125 c/c	25 @125 c/c

Component	Top	Bottom	Vertical		Horizontal	
Left Side Wall (Top 50% height of wall)			32 @125 c/c	25 @125 c/c	25 @125 c/c	25 @125 c/c
Right Side Wall (Bottom 50% height of wall)			25 @150 c/c	25 @ 150 c/c	25 @200 c/c	25 @200 c/c
Right Side Wall (Top 50% height of wall)			25 @ 300 c/c	25 @ 300 c/c	25 @ 200 c/c	25 @ 200 c/c

Check for Limit State of Serviceability

Vertical Deflection of Bottom slab

As per clause IS 1904, the maximum permissible settlement in raft foundation is 100 mm. However, the maximum settlement observed (Load case 19) from the calculation for the raft of 3 m thick is 28.25 mm. Hence, limit state of serviceability is satisfied.

Check for Lateral deflection in Side Walls

As per clause 7.11.1.1 of IS 1893, the lateral sway/drift shall not exceed 0.004 times the height of the structure. A maximum deflection of 77.06 mm (at top of left wall) was calculated for the service load case 19 which is slightly greater than the allowable deflection (0.004*15.5) = 60mm. The slightly greater calculated deflection is not critical. Deflections could be further reduced with the provision of beams connected to both the tops of the left and right wall. Anchors could also be placed in the tops of left wall and anchored in the backfill.

7.2.8.3 Results of Calculations

Table 7-59: Summary of Reinforcement Details for Deepest section of Terminal Structure & Tail bay of Outlet Structure

Component	Top		Bottom		Vertical		Horizontal	
	Flow direction	Perpendicular to flow direction	Flow direction	Perpendicular to flow direction	Outer	Inner	Outer	Inner
Bottom Slab (Centreline of slab to till end of slab on left side)	25 @ 125 c/c	25 @ 150 c/c (2 rows)	25@ 125 c/c	32 @ 150 c/c (2 rows)				
Bottom Slab (Centreline of slab to till end of slab on right side)	25 @ 125 c/c	25 @ 150 c/c	25 @ 125 c/c	32 @ 150 c/c				
Left Side Wall (Bottom 50% height of wall)					32 @125 c/c (2 rows)	25 @125 c/c	25 @125 c/c	25 @125 c/c
Left Side Wall (Top 50% height of wall)					32 @125 c/c	25 @125 c/c	25 @125 c/c	25 @125 c/c
Right Side Wall (Bottom 50% height of wall)					25 @150 c/c	25 @ 150 c/c	25 @200 c/c	25 @200 c/c
Right Side Wall (Top 50% height of wall)					25 @ 300 c/c	25 @ 300 c/c	25 @ 200 c/c	25 @ 200 c/c

7.2.9 Power Station Area - Powerhouse Cavern

The Tamakoshi V HEP Powerhouse Cavern accommodates four turbine/generator units. The dimension of the Powerhouse Cavern is 69.00 m (L) X 30.35 m (H) X 18.00 m (W). The EOT Crane 85t/15t will be mounted at El. 989.19 m, on a concrete beam which will run through the entire Powerhouse length of 69 m. A 25 mm wide expansion joint was provided between the Erection Bay and the Unit Bays (between Axis 4 and 5). The plan of the Powerhouse at Machine Hall Floor and the cross section of the Powerhouse through the center line of the Unit 1 are given in the two figures flowing below.

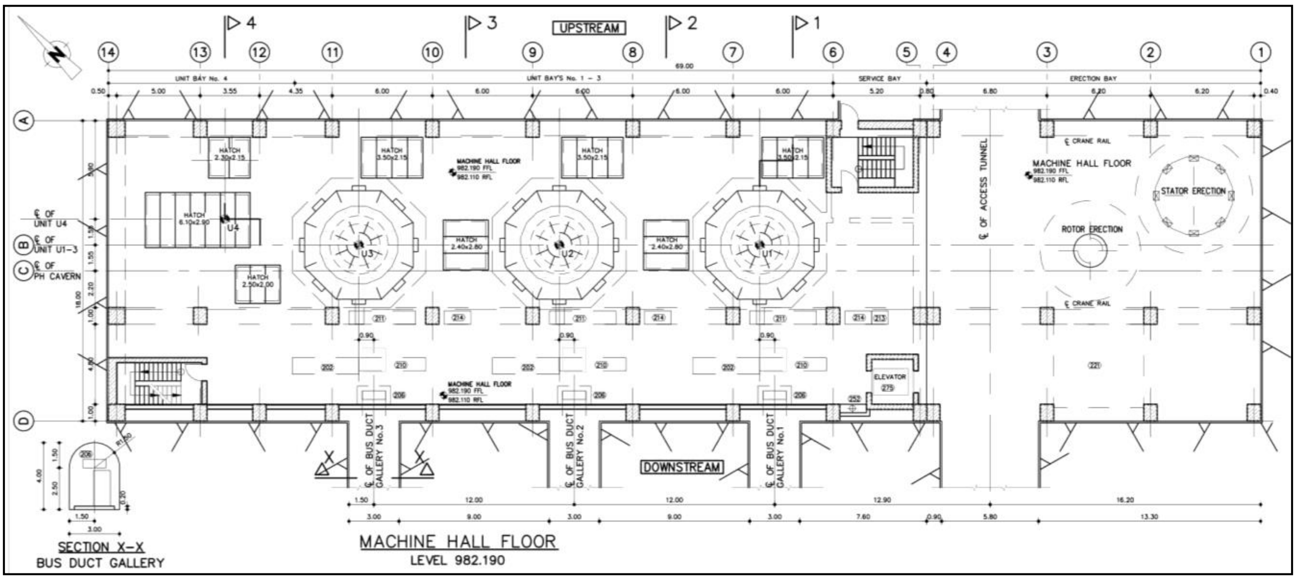

Figure 7-28: Plan View of the Powerhouse Floor (El. 982.19 m)

The important structural elements of the Powerhouse Cavern were analyzed by detailed structural calculations. These calculations are presented in full detail in Part A3 Chapter 4 of this Detailed Design Report.

However, in order to provide the descriptions in the Main Volume of the Detailed Design Report in a concise manner, the details are not reiterated here. Rather, the focus is confined to brief descriptions of the methods and results of the analyses which are presented here below.

7.2.9.1 Crane Beam

An overhead EOT crane of capacity 85T/15T for erection and maintenance purpose of E&M equipment's was planned. This EOT crane will be mounted at El 989.19 m, over a concrete beam which runs all through the Powerhouse length of 69 m both in A-line and D-line frame. However, an expansion joint of 25mm thick has been introduced at one location of the beam, between service bay and unit bays, for the movement of the crane, the rails with its embedded parts shall be fixed at top of the crane beam. Two end stoppers are provided at both the ends of the crane beam to avoid the movement of crane beyond the Powerhouse frame. The center to center distance between the crane rail in A-line frame and D-line frame is 10.4m.

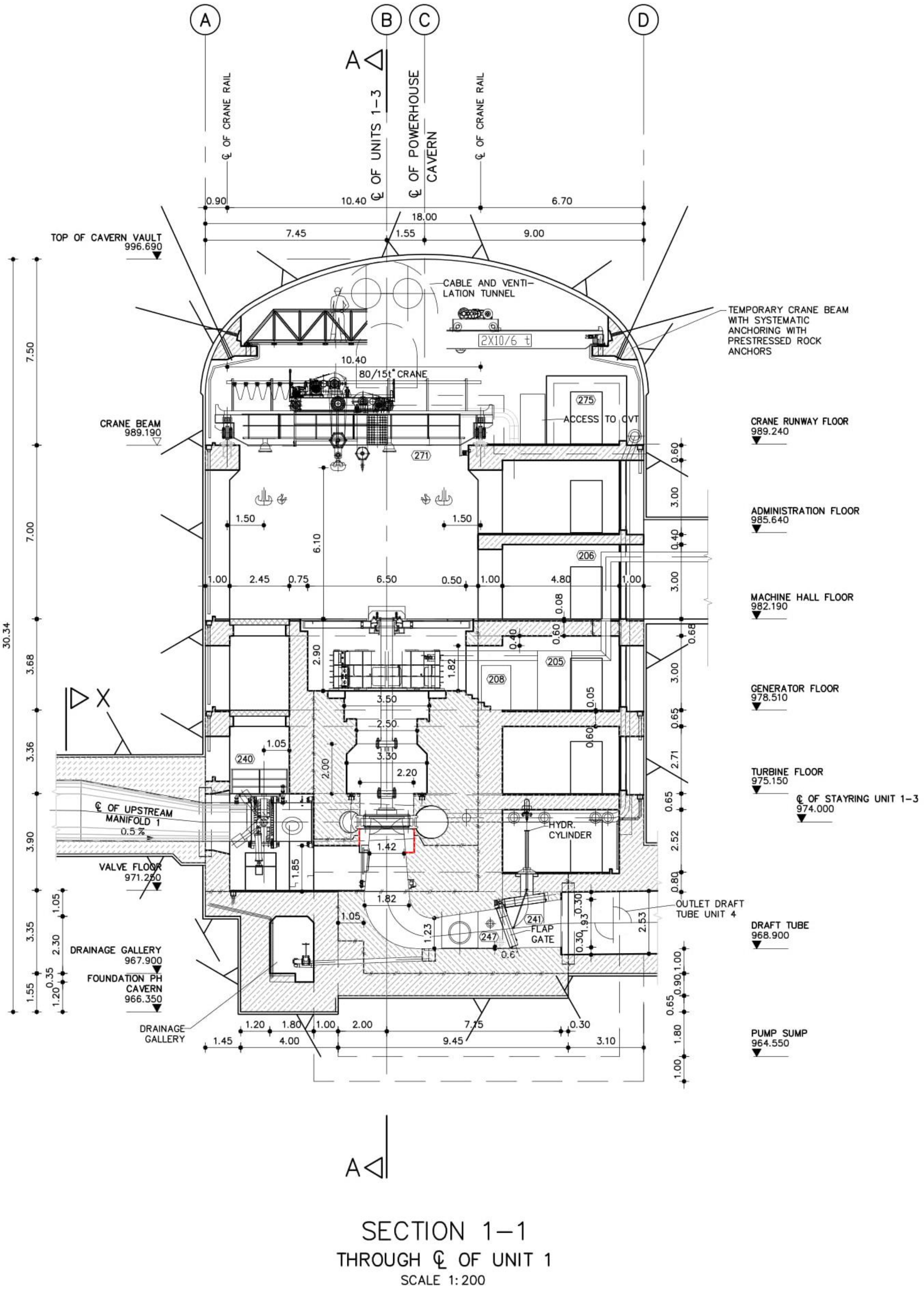

Figure 7-29: Section of Powerhouse through Centerline of Unit 1

Specific Design Criteria

The Crane beam was analyzed and designed for the wheel loads of the crane and, at same time, the maximum nodal reactions at the supports were calculated. These reactions were applied on the appropriate locations of the Service Bay and Machine Hall Axis A and Axis D columns for the frame analysis and design. The analysis of the beam was carried out using STAAD Pro. Software. The maximum moments and shear forces, were calculated from the STAAD Pro. Software used to design the beams was an Excel spread sheet.

Results of Calculations

Summary of Reinforcement

Based on the design, experience on the similar projects and the construction point of view, the required reinforcement for Crane beam of 1400 m (W) x 1000 m (D) is given in table below:

Table 7-60: Summary of Reinforcement Details for Crane Beam and Dowel Bars (connecting the beam with the column

Component	Top Hogging reinforcement	Bottom Sagging Reinforcement
Crane beam	7 Numbers of 25 mm Dia	7 Numbers of 25 mm Dia
Dowel Bars for connecting the Crane Beam to the Column	2 Numbers 25 mm Dia, U-Shaped bars at each column location. Depth of embedment of bar in the column is 350 mm	
Side Face Reinforcement on each side of beam	4 Numbers of 16 mm Dia.	

7.2.9.2 Design of Service Bay/Erection Bay Frame b/w the Grid 1-4

This section covers the analysis and design of the Service Bay/Erection Bay frame structure. The center to center dimension of the frame is 19.2 m (L) X 17.0 m (W) X 6.88 m (H). All the columns of the frame are embedded in the raft (thickness 600 mm) at El. 982.19 m. A slab of 600 mm thick and 400 mm thick was provided at El. 989.19 m and El. 986.64 m respectively in between the d/s crane rail and the Axis D. Erection Bay frame, modelled in Staad.pro and 3D rendered view, is shown in the figure below.

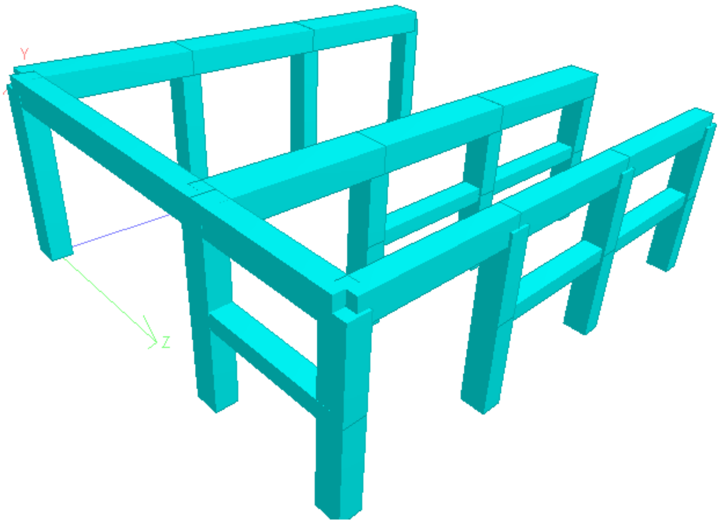

Figure 7-30: Erection Bay/Service Bay Frame Model in STAAD.Pro Software

Specific Design Criteria

The Analysis of the frame is carried out using STAAD Pro. Software using line elements. The columns are designed for a maximum vertical load and its corresponding moments, maximum moment in X-direction and corresponding load and maximum moment in Z-direction and its corresponding vertical load. In the above three conditions, the critical condition is checked. The beam along the D-line is also designed for the maximum moments and the shear. Slabs at Crane runway floor and Administration floor are designed for the Live load of 10 KN/m^2 using the spread sheet prepared in excel.

Results of Calculations

Summary of Reinforcement for Service Bay Frame

Based on the design, experience on the similar projects and the construction point of view, the required reinforcement for Service Bay frame in given the table below:

Table 7-61: Summary of Reinforcement for Slabs, Beams and Columns of Service Bay Frame

Component	Top Hogging reinforcement	Bottom Sagging Reinforcement
Beams (0.6 m (W) X 0.8 m (H) in Administration Floor	5 -25ø & 2- 20 ø	5- 25ø
	Provide 4 legged 10ø @ 100 mm (within 1150 mm from support on both sides of beam) at support & 250 mm c/c in the middle	
Slab at Crane Runway Floor (El. 989.24 m)		Main reinforcement : 12 ø @ 125 C/C Distribution reinforcement: 12 ø @ 125 C/c

Component	Top	Bottom
Slab at Administration Floor (El. 985.64 m)		Main reinforcement : 12 ø @ 150 C/c Distribution reinforcement: 12 ø @ 150 C/C
All Crane Columns & Columns in D-Line	Longitudinal Bars: 18 Nos of 25ø Transverse reinforcement: • 4 legged 10ø @ 200 C/C and 1 no. 10ø link at 200C/C near the support for a length of 1.0 m • 4 Legged 10ø @ 200 C/C in the mid length of the Column.	

7.2.9.3 Design of Service Bay Raft

<u>Specific Design Criteria</u>

The height of Service Bay from the top of raft to crane beam top is 7.0 m (i.e. from EL.982.19 m to EL.989.19m). The Length of Service Bay is 20.00 m. The width of Service Bay from A-line to D-line is 18.0m. The overall thickness considered for the design of raft foundation is 1075.00mm (i.e.1000mm as structural part of RCC grade C25/30 + 75mm floor finish). The bottom level of the excavated profile is El.981.11 m. Total 12 nos. of columns (of size 0.8 m (W) X 1.0 m (D)) are placed above the service raft. The center to center distance of the columns and the size of the Service bay raft is shown in below figure.

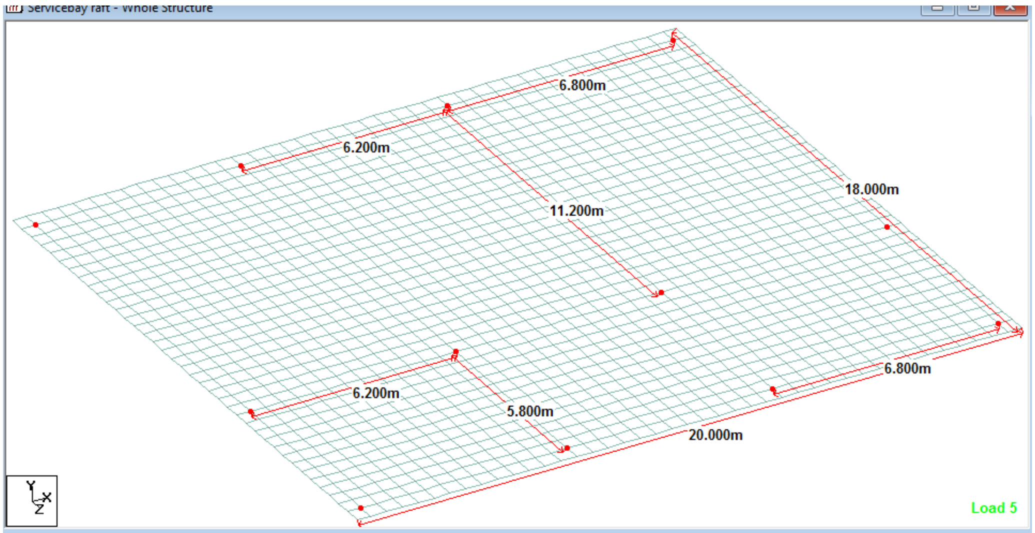

Figure 7-31: Centre to Centre Distance of Column & Service Bay Raft Size

A 3-Dimensional analysis was carried out using STAAD-Pro software, by considering the loads mentioned subsequently. The structure was analyzed as a space model. The whole raft was modeled using plate elements. The thickness of the raft was assumed as 1000mm.

Results of Calculations

Summary of Reinforcement for Service Bay Raft

Based on the design, experience on the similar projects and the construction point of view, the required reinforcement for Service bay raft is given in below table:

Table 7-62: Summary of Reinforcement for Raft below Columns and in between the Columns

Component	Top	Bottom
Below Columns in Service Bay Raft	Longitudinal direction: 25ø @ 175 c/c Transverse direction: 25ø @ 125 c/c	
In b/w columns in Service Bay Raft	Longitudinal direction: 20ø @ 200c/c Transverse direction: 20ø @ 200 c/c	

7.2.9.4 Powerhouse Unit Block

The FEM model of the Powerhouse, excluding the Erection Bay, was modelled with SAP2000 software.

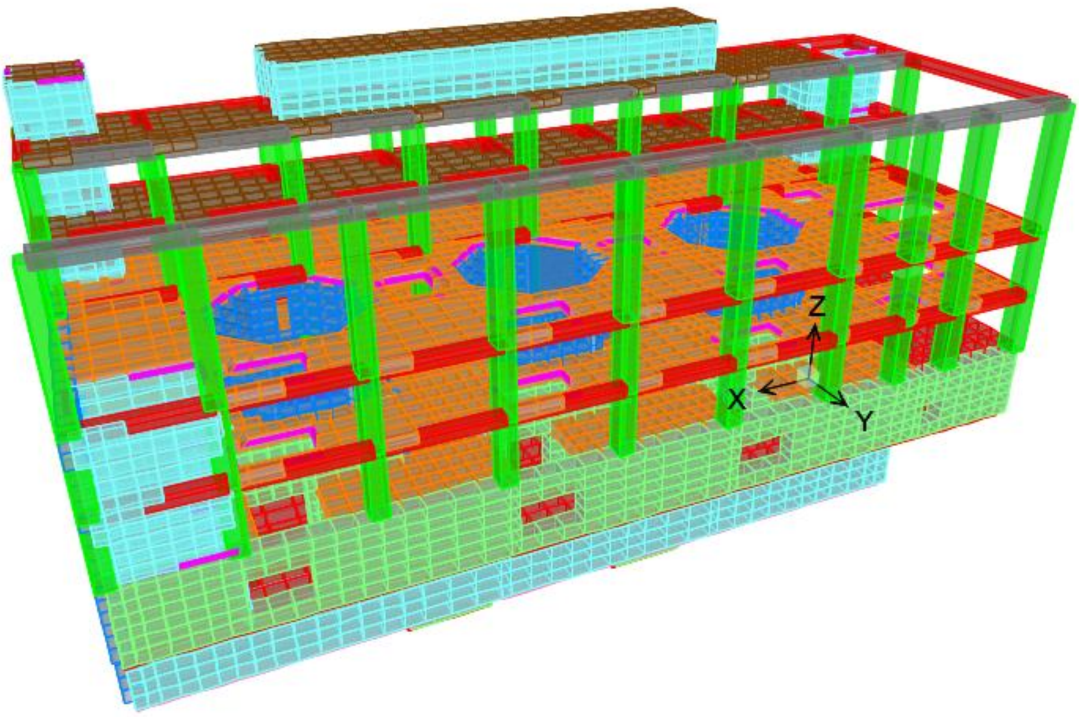

Figure 7-32: 3-D model of the Powerhouse

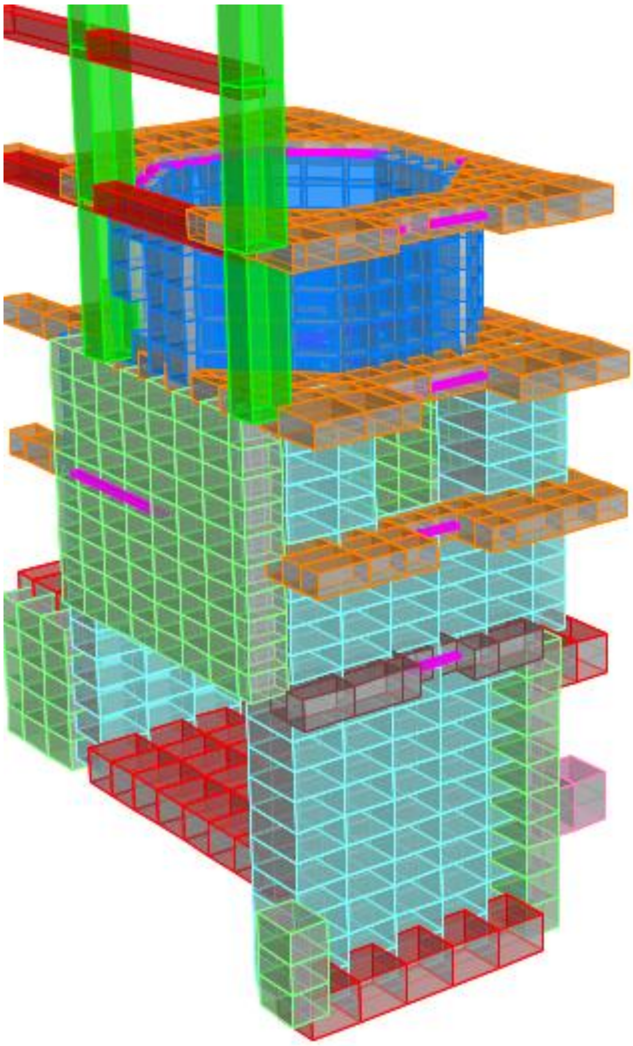

Figure 7-33: 3-D model of the Powerhouse Transverse Section (modelled integral with structural system)

Specific Design Criteria

Assumptions

The following assumptions were made for the structural analysis and design:

- The Powerhouse was analyzed as an RCC frame
- The structures after completion of first stage concrete was modeled with FEM software and the effect of secondary concrete was not considered. The addition of secondary concrete will enhance the structural performance.
- The live load at each floor was taken as per planned occupancy

- The earthquake load was calculated considering dead weight of structure, occupancy load and weight of the crane girder with crab
- The crane will be operated rarely, and the effect of earthquake and handling of maximum weight were not superimposed.
- The support beam will not deform axially, hence, longitudinal loads due to crane was applied at a point
- The total weight of the generator including turbine runner was applied to the Generator Floor Level distributed equally on eight points (200 kN at each point for a total of 1600 kN). Similarly, thrust was applied at the front wall location of the turbine floor as a uniformly distributed load (68.75 kN/m equivalent to a total thrust of 550 kN).
- Safe bearing capacity of the foundation material was assumed as 80 t/ m^2 and soil subgrade reaction was assumed as 80,000 KN/m/m^2.

Result of Calculations

Table 7-63: Summary of Reinforcement for slab/Raft

Location	Effective Depth provided (mm)	Reinforcement bars provided			
Machine Hall Floor	560	16	mm bar at	200	mm C/c
	560	16	mm bar at	200	mm C/c
Generator Floor	560	16	mm bar at	200	mm C/c
	560	16	mm bar at	200	mm C/c
Turbine floor	560	20	mm bar at	200	mm C/c
	560	20	mm bar at	200	mm C/c
Turbine floor unit 4 area	960	20	mm bar at	200	mm C/c
	960	20	mm bar at	200	mm C/c
Auxiliary floor	560	20	mm bar at	200	mm C/c
	560	20	mm bar at	200	mm C/c
Slab above drainage gallery	960	20	mm bar at	200	mm C/c
Valve floor raft	960	20	mm bar at	200	mm C/c
	960	20	mm bar at	200	mm C/c
Administration floor	260	16	mm bar at	200	mm C/c
Crane Runaway floor	560	16	mm bar at	200	mm C/c
Cover slab of HVAC fan room	260	16	mm bar at	200	mm C/c

For Machine Foundation walls provide 0.25 % reinforcement on each wall face in vertical and horizontal direction. And for other walls provide 0.12 % reinforcement on each direction on each face of wall.

Wall thickness	**Machine foundation**	**Other**
1200 mm	25 mm dia. @ 150 mm C/C both ways in two layers (eg. first stage concrete wall for turbine and generator foundation)	20 mm dia. @ 200 mm C/C both ways in two layers
1000 mm	25 mm dia. @ 200 C/C both ways in two layers (eg. first stage concrete wall for turbine and generator foundation)	16 mm dia. @ 200 mm C/C both ways in two layers
800 mm	20 mm dia. @ 150 C/C both ways in two layers (eg. first stage concrete wall for barrel above generator floor)	16 mm dia. @ 200 mm C/C both ways in two layers.
300 mm	Not applicable	12 mm dia. @ 200 mm C/C both ways in two layers

For other beams provide minimum 0.3 percent of longitudinal and shear reinforcement in accordance to section size.

7.2.9.5 Design of Dog-Legged Staircase between the Grid 5- 6

The dog-legged staircase (in between the grid 5-6 & in between 13-14) was proposed to access the different floors of the Powerhouse. To determine the loads due to the staircase arrangement on the surrounding columns, a typical design staircase, between the Generator Floor (El. 978.15 m) and the Turbine Floor (El. 975.15 m) with the same load was applied on all the staircase connected to the columns in the Powerhouse. In between the above two floors, one mid-landing was foreseen at the El. 976.83 m. Staircases were designed by the Limit State Method based on IS 456. A live load of 5 KN/m^2 was considered for the design based on the IS 875 (Part 2). Detailing was carried out as per the SP: 34. Partial safety factor of 1.5 was considered. The support system for the staircase was considered as simply supported (allowing for precast construction). The details of the reinforcement is given in the figure below.

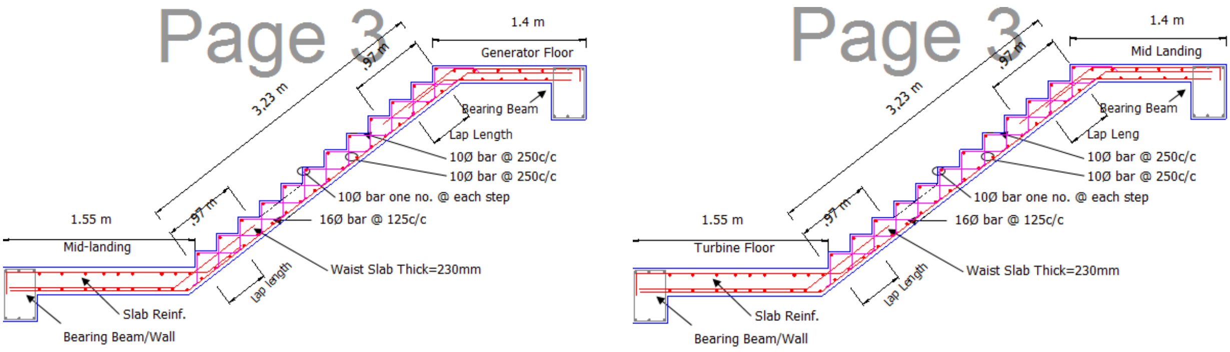

Figure 7-34: Details of Reinforcement for a Typical Staircase between El.978.15 m and El.975.15 m

7.2.9.6 Design of Bottom Slab of Draft Tube

Since the Powerhouse super structure is mounted above a 15 m high mass concrete of M30 grade, the uplift pressures acting below the first stage concrete is negligible compared with the self-weight of the Machine Hall structure and, hence, there is no need for checking the global stability due to uplift. However, the uplift pressures shall be checked locally, and the members shall be design accordingly. In this section, the thickness of the slab below the draft tube elbow was designed. Sectional elevation and plan of draft tube slab portion is given in the figure below.

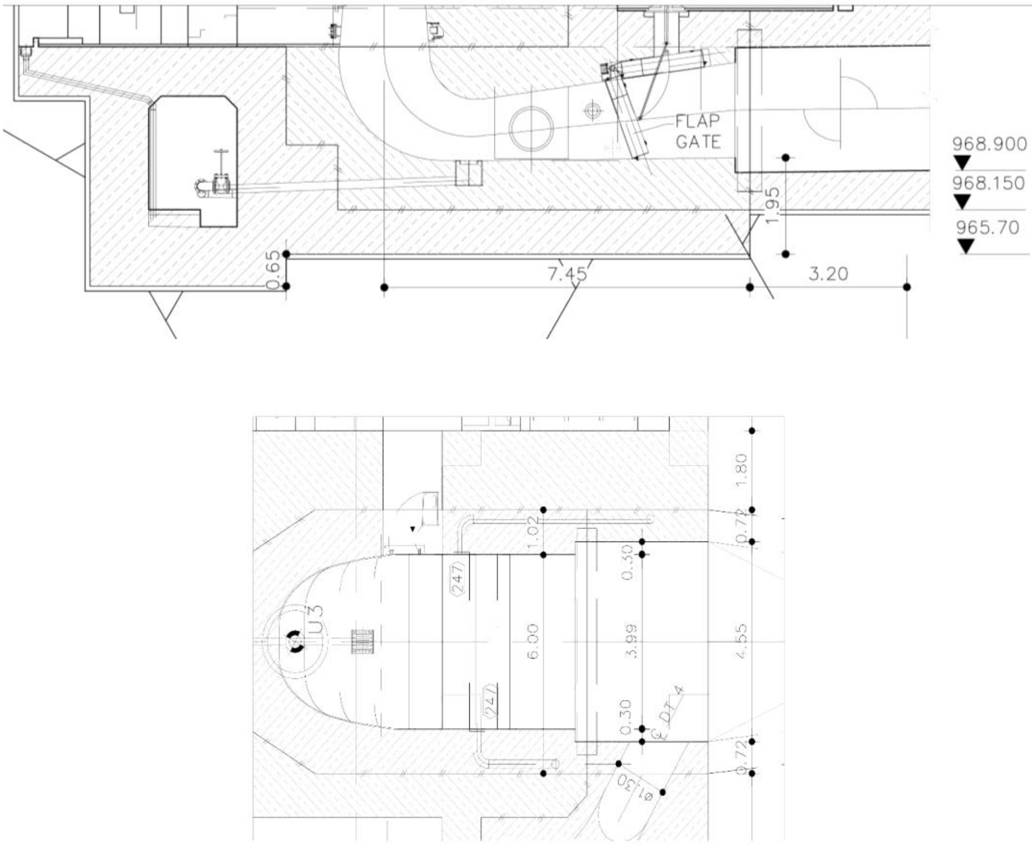

Figure 7-35: Sectional & Plan Details of Draft Tube Slab

Specific Design Criteria

The bottom slab of the Draft Tube was designed for the uplift pressures. Uplift pressures below the Powerhouse foundation (between A- Line & D- Line) were calculated based on the possible water levels (at the Outlet Structure & normal water levels in the Surge Tank) and the foundation level of the Powerhouse. At this stage, based on the rock condition, it is assumed that only 30 % of the head (El.1158 - 965.70 m = 192.30 m) will act at the upstream of the Powerhouse (i.e. at Axis A). Downstream of the Draft Tube, 100% uplift head (El.991.30 - 965.70 = 25.6 m) was considered to act on the bottom of the Powerhouse. Therefore, the above uplift pressures were considered for the analysis and design of Draft Tube Slab. The thickness of the slab below Draft Tube elbow is around 1.9 m with a width of 5 m and a length of 7.45 m (length assumed for calculations). The slab was designed as a one-way slab for the assumed uplift pressures.

Results of Calculations

The bottom slab of the Draft Tube was designed for the uplift pressures as a one way slab. On top of the slab, reinforcement was calculated for the uplift pressures and for the bottom of the slab a minimum reinforcement of 0.12% of cross-sectional area (BD, B- Width & D-Overall depth) was provided.

Based on the design, experience on similar projects and the construction point of view, the required reinforcement for bottom slab of the Draft Tube is given in table below.

Table 7-64: Summary of Reinforcement for Bottom Slab of Draft Tube

Component	Top	Bottom
Bottom Slab for a length of 7.45 m (b = 1 m, D = 1.95 m). However, same reinforcement is provided bottom of the draft Tube from Junction of 1st & 2nd stage concrete below draft tube	32 ø @ 100 C/c (Parallel to length of PH direction) 25 ø @ 200 C/c (Water flow direction)	25ø @ 200 C/C both Directions
Bottom Slab (First stage concrete) for a length of 3.2 m (b = 1m D = 800 mm)	32 ø @ 100 C/c (Parallel to length of PH direction). Shear reinforcement or increase of the slab thickness may be considered in the next stage of the project. 20 ø @ 250 C/c (Water flow direction)	20ø @ 250 C/C both Directions

7.2.10 Power Station Area - Transformer Cavern

7.2.10.1 Description of Structure

The Transformer Cavern will accommodate three 3-Phase transformers on the Transformer Floor (El. 982.10 m) and GIS equipment will be placed on the GIS Floor (El. 989.99 m). Access to GIS Floor from Transformer floor is planned through two stair cases at each end of the Transformer Cavern. A 10 t capacity EOT crane, was planned to run on beams along the entire length (47.6 m) of the Transformer Cavern. The Transformer floor is founded directly on rock. The plan view of the Transformer Floor and the GIS Floor is given in the figure below.

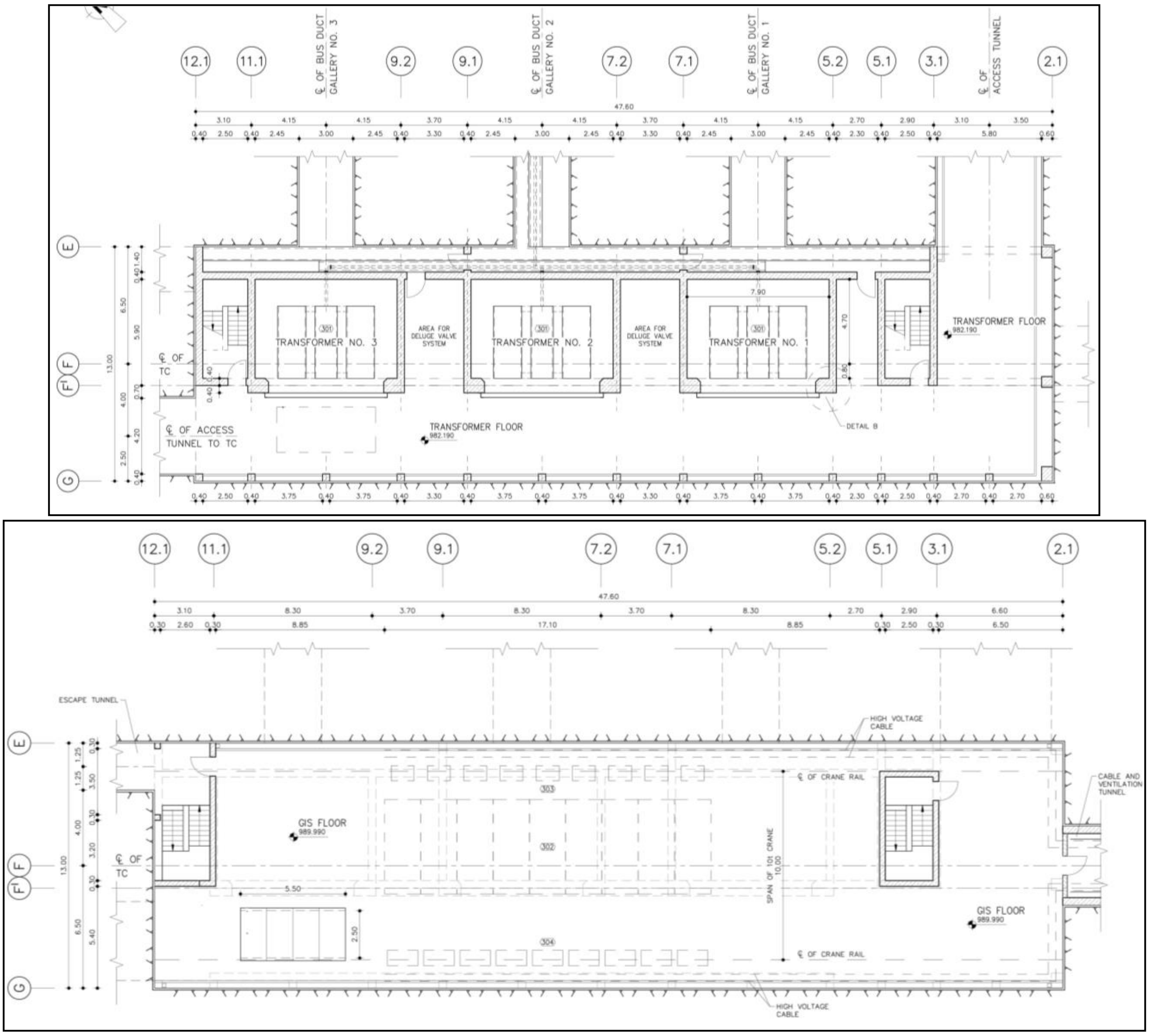

Figure 7-36: Plan of Transformer Floor (El. 982.19 m) and GIS Floor (El.989.99 m)

This section covers:

- The transformer foundations & top & side walls,
- The reinforced concrete enclosure of the transformers,
- The GIS floor slab.
- The two staircases between the Transformer and GIS floor

7.2.10.2 Results of Calculations

Transformer Foundation

Table 7-65: Summary of Reinforcement for Transformer Foundation

Component	Cavern length direction (Main)	Cavern width direction (Distribution)	Bottom: Cavern length direction (Main)	Bottom: Cavern width direction (Distribution)	Vertical: Outer (Main Rein.)	Vertical: Inner (Main Rein.)	Horizontal: Outer (Distribu-tion Rein.)	Horizontal: Inner (Distribu-tion Rein.)
Foundation Slab	12 @ 100 c/c	10 @ 100 c/c	12 @ 100 c/c	10 @ 100 c/c				
Pedestal Wall					12 @ 100 c/c	12 @ 100 c/c	10 @ 100 c/c	10 @ 100 c/c
Jacking Pad					16 Nos. of 16 mm dia verti-cal bars (main reinforcement) around the Pad		8 mm dia @ 150 C/c transverse reinforce-ment	

Reinforced Concrete Enclosure of Transformer

Table 7-66: Summary of Reinforcement Details for Top Slab and Vertical Walls of Transformer RCC Box Structure

Component	Top: Cavern width dir.	Top: Cavern length dir.	Bottom: Cavern width dir.	Bottom: Cavern length dir.	Vertical: Outer	Vertical: Inner	Horizontal: Outer	Horizontal: Inner
Top Slab	12 @175 c/c	12 @175 c/c	12 @150 c/c	12 @175 c/c				
Vertical Walls					20 @175 c/c	20 @175 c/c	16 @ 200 c/c	16 @ 200 c/c

GIS Floor Slab

Summary of Reinforcement for GIS Floor Slab b/w the Grid G-F’-12.1-2.1

Based on the design and experience on the similar projects and the construction point of view, the required reinforcement for GIS floor slab located in the grid G-F’-12.1-2.1 is given in below table.

Table 7-67: Summary of Reinforcement Details for GIS Floor Slab

Component	Top		Bottom	
	Cavern length direction	Cavern width direction	Cavern length direction	Cavern width direction
GIS Floor Slab	12 @ 200 c/c	12 @ 200 c/c	12 @ 200 c/c	12 @ 200 c/c

Staircases

The staircases were proposed to access the GIS floor El. 989.99 m from Transformer floor El. 982.10m. Between these two floors, four mid landings were proposed. This section covers the typical design of staircase between two mid landings i.e. El. 985.31 m & El. 986.87 m. For all other flights, the design is like the above. staircase was designed by the Limit State Method based on IS 456. A live load of 5 KN/m^2 was considered for the design based on the IS 875 (Part 2). Detailing was carried out as per the SP: 34. A partial safety factor of 1.5 was considered. The support system for the staircase was considered as simply supported. The details of the reinforcement are given in the figure below.

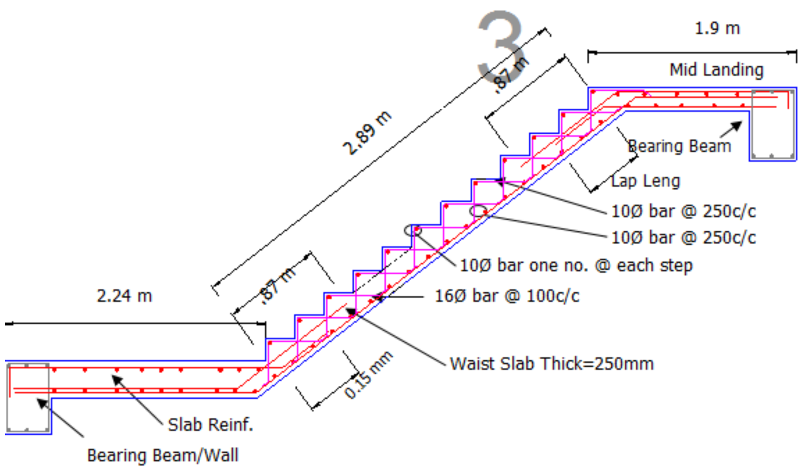

Figure 7-37: Details of Reinforcement for a Typical Flight Between El.986.87 m and El.985.31 m

7.2.11 Power Station Area - Terminal and Ventilation Building

7.2.11.1 Description of Structure

The Terminal & Ventilation Building is located at the outlet of the Cable and Ventilation Tunnel. The Terminal and Ventilation Building was designed to accommodate systems for ventilation of the Powerhouse Cavern. The Cable and Ventilation Tunnel contains a cable section which accommodates cables from the Transformer Cavern. A sketch showing the plan view of the Terminal and Ventilation Building is presented below:

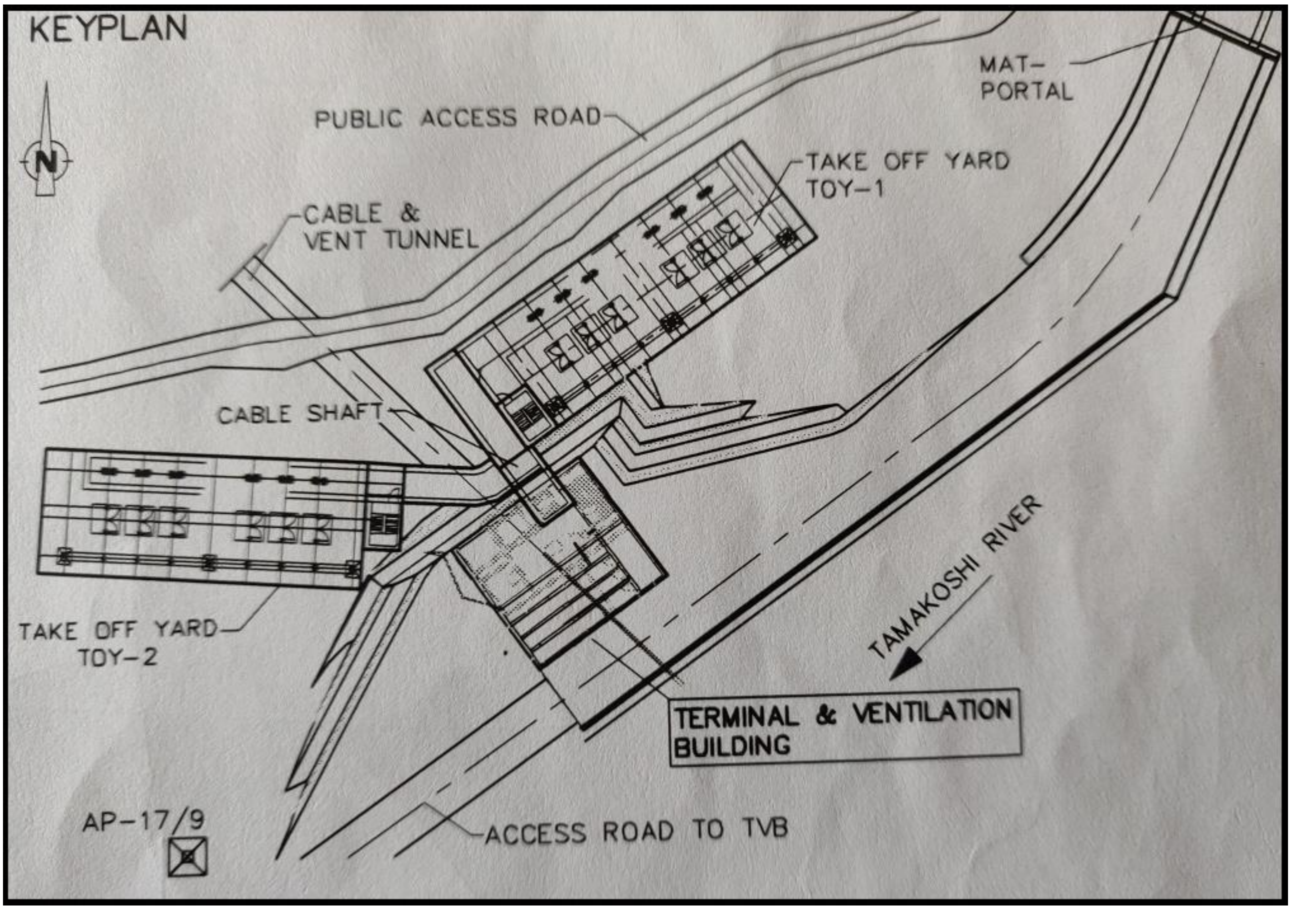

Figure 7-38: Sketch of the Plan of Terminal Ventilation Building and Surrounding Structures

7.2.11.2 Results of Calculations

Roof Slab

Overall thickness of Slab = 300 mm

Required maximum longitudinal reinforcement area = 1042.22 mm^2 /m

Provided longitudinal reinforcement Top 16 mm bar at 190 mm C/C both ways.

Temperature reinforcement 16 mm bar at 200 mm C/C.

Ventilation Floor Slab

Overall thickness of Slab = 300 mm

Required maximum longitudinal reinforcement area = 1037.83 mm^2/m

Provided longitudinal reinforcement Top 16 mm bar at 190 mm C/C both ways.

Temperature reinforcement 16 mm bar at 200 mm C/C.

<u>Electrical Floor Slab</u>

Overall thickness of Slab = 600 mm

Required maximum longitudinal Reinforcement area = 1063.20 mm^2/m

Provided longitudinal reinforcement   Top 20 mm bar at 150 mm C/C both ways.

Temperature reinforcement 16 mm bar at 200 mm C/C.

7.2.12 Power Station Area - Operation Building

7.2.12.1 Description of Structure

The Operation Building is a r.c. frame structure. It was designed based on the most critical limit state and checked for other limit states of serviceability (ref. table 18 IS456-2000 and Cl. 8.3.2 IS 1893 (Part4) 2015).

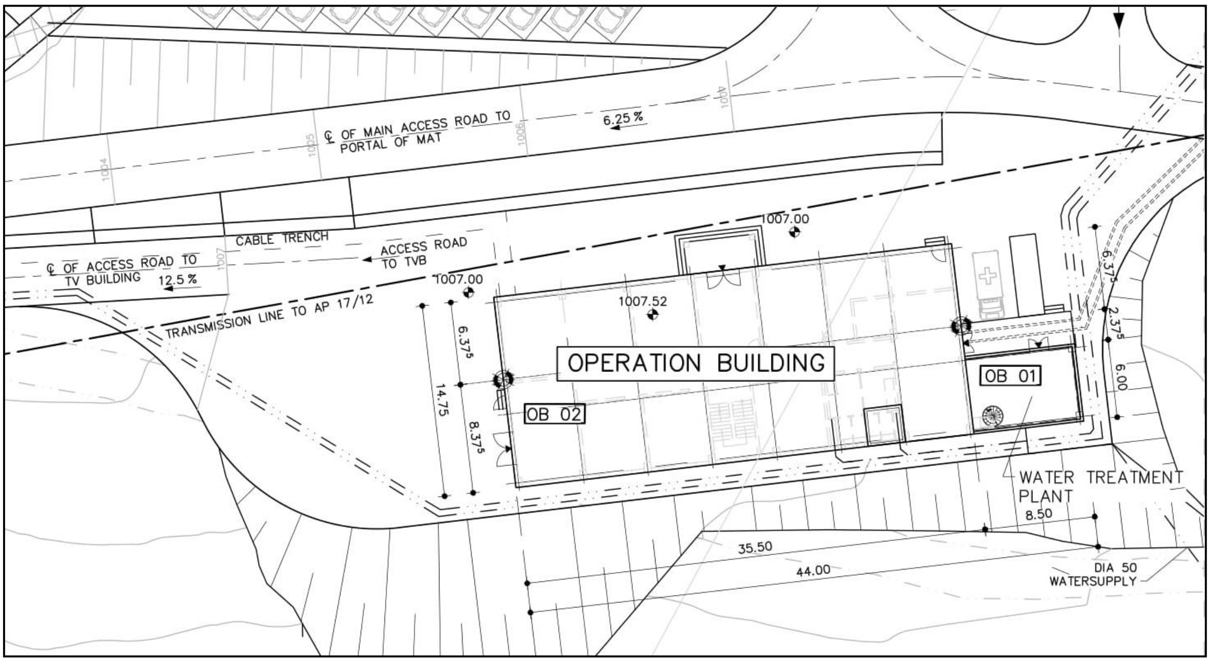

Figure 7-39: Plan Showing the Operation Building and Surrounding Structures

7.2.12.2 Results of Calculations

All slab panels were designed with 200 mm thickness and 12 mm dia. bars at top and bottom.

Table 7-68: Beam Reinforcement Detail

Floor	Beam Dimension B mm X D mm	Reinforcement		
		Longitudinal		Shear
		Top	Bottom	
Plinth	400 x 650	4- 20 mm dia. Through 4- 16 mm dia Extra	4- 16 mm dia. Through 2- 16 mm dia Extra	4 legged 8 mm dia. at 100 c/c throughout the length.
Ground	350 X 500	4- 20 mm dia. Through 4- 16 mm dia Extra	4- 16 mm dia. Through 2- 16 mm dia Extra	4 legged 8 mm dia. at 100 c/c throughout the length.
	400 X 650	4- 25 mm dia. Through 4- 20 mm dia Extra	4- 20 mm dia. + 2- 20 mm dia Through	4 legged 10 mm dia. at 100 c/c throughout the length.
First	350 X 500	4- 16 mm dia. Through 4- 16 mm dia Extra	4- 16 mm dia. Through	4 legged 8 mm dia. at 100mm c/c throughout the length.
	400 X 650	4- 20 mm dia. Through 4- 20 mm dia Extra	4- 16 mm dia. + 2- 16 mm dia Through	4 legged 8 mm dia. at 100mm c/c throughout the length.

Table 7-69: Detail of Column Reinforcement

Floor	Column Dimension B mm X D mm	Reinforcement Longitudinal	Confinement
Below Plinth level	400 x 400	12-16 mm dia.	8 mm dia, at 100mm c/c at ends (500mm from face of beam) and 8 mm dia, at 150mm c/c remaining height
	600 x 700	14-25 mm dia.	
Ground Floor	400 x 400	12-16 mm dia.	8 mm dia, at 100mm c/c at ends (700mm from face of beam) and 8 mm dia, at 150mm c/c remaining height
	600 x 700	14-25 mm dia.	
First Floor	600 x 700	14-25 mm dia.	8 mm dia, at 100mm c/c at ends (700mm from face of beam) and 8 mm dia, at 150 c/c remaining height

The size of foundation for each joints is tabulated below.

Table 7-70: Details of Each Foundation

Foundation Type	Joint numbers	Footing Size (L x B) m	Depth of Foundation, m	Overall Depth of Footing Slab, mm	Reinforcement
F1	177, 180	1.3x 1.3	1.5	400	12 mm dia at 150 mm c/c bothways
F2	9, 13	2.3 x 2.3	1.5	500	12 mm dia at 150 mm c/c bothways
F3	17	2.5 x 2.5	1.5	550	12 mm dia at 150 mm c/c bothways
F4	21, 25, 29, 33, 45, 49, 61, 65,77,81	2.8 x 2.8	1.5	550	16 mm dia at 150 mm c/c bothways
F5	37,53,69,85,93,97,109,113 ,117,125,129,133,141	3.0 x 3.0	1.5	600	16 mm dia at 150 mm c/c bothways
F6	27, 39, 29, 41, 31, 43, 45, 33	3.1 x 3.1	1.5	650	16 mm dia at 150 mm c/c bothways

A dog legged type stair case was designed for this building. The structural analysis required a slab thickness of 200 mm. The chosen reinforcement is 16 mm dia. main bars spaced 200 mm c/c in span direction and 10 mm dia. bars space 200 mm c/c perpendicular to the span direction.

7.2.13 Power Station Area - Workshop Building

7.2.13.1 Description of Structure

The Workshop Building is in the Power Station area. The Workshop Building was designed to accommodate space for repair and maintenance of various electrical and mechanical equipment. The Workshop Building was provided with a crane with a capacity of 10 tons to handle equipment at the Workshop Building during maintenance.

The Workshop Building consists of two separate blocks separated by a joint. Block 1 (northern block) accommodates office rooms, the electrical workshop and storage areas. Block 2 (southern block) is provided with a 10 t capacity crane.

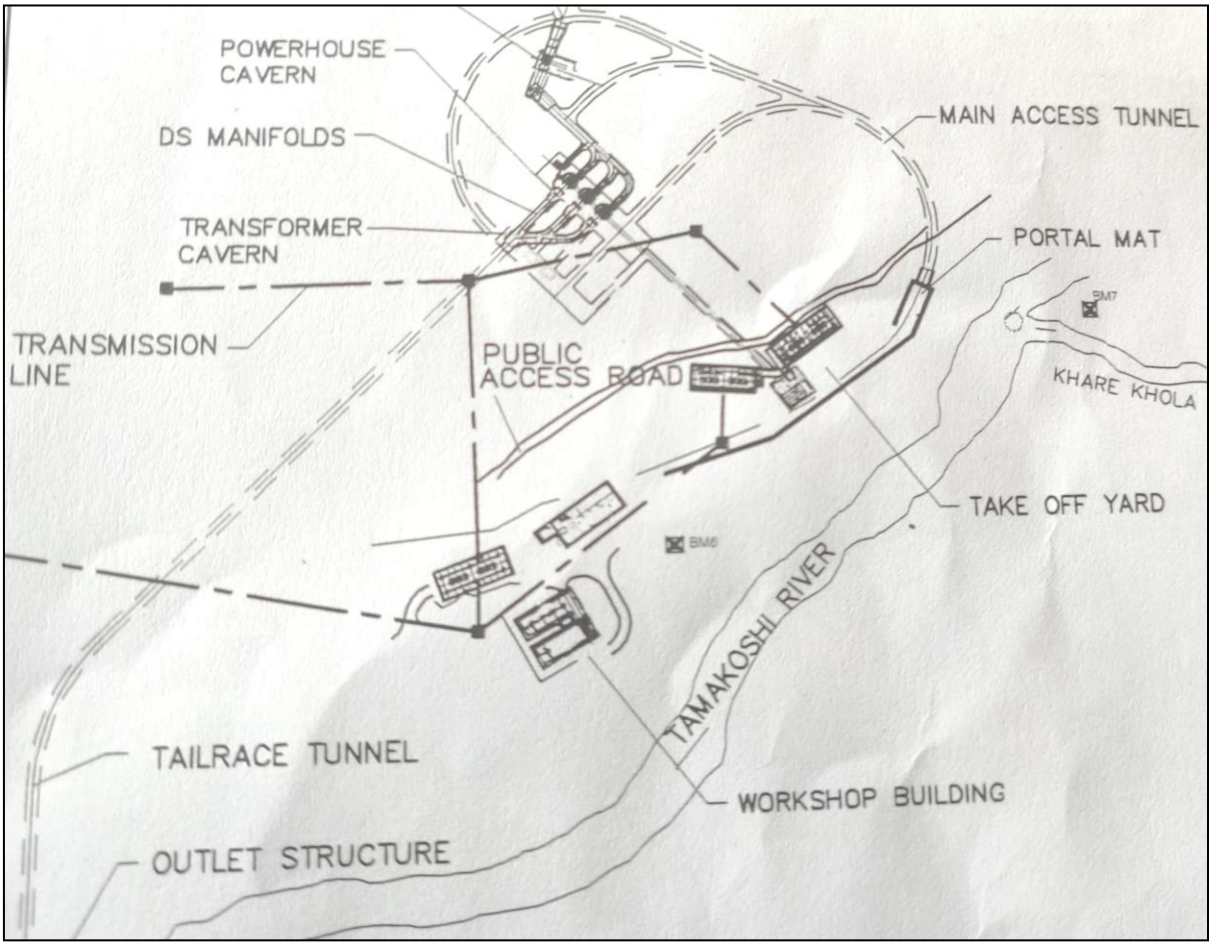

Figure 7-40: Sketch of the Plan of Workshop Building and Surrounding Structures

7.2.13.2 Results of Calculations

Beam and Column Reinforcement

Block 1

Table 7-71: Detail of Beam Reinforcement of Workshop Building Block 1

Floor/ level	Beam Dimension B mm x D mm	Reinforcement		
		Longitudinal		Shear
		Top	Bottom	
EL. 1011.10	300 x 500	3-20 mm dia. Through 2 – 20 mm dia. Extra	3- 16 mm dia. Through	4 legged 10 mm dia. At 75 c/c at the ends (500 mm from face of column) and 4 legged 10 mm dia. at 150 c/c remaining span
Roof	300 x 500	3- 20 mm dia. Through 2- 20 mm dia Extra	3- 16 mm dia. Through	4 legged 10 mm dia. At 75 c/c at the ends (500 mm from face of column) and 4 legged 10 mm dia. at 150 c/c remaining span
Staircase cover	300 x 500	3- 20 mm dia. Through	3- 16 mm dia. Through	2 legged 10 mm dia. At 75 c/c at the ends (500 mm from face of column) and 2 legged 10 mm dia. at 150 c/c remaining span

Table 7-72: Detail of Column Reinforcement of Workshop Building Block 1

Type	Column Dimension B mm X D mm	Reinforcement	
		Longitudinal	Confinement
All column	600 x 600	16-16 mm dia.	8 mm dia, at 125 c/c at ends (600 mm from face of beam) and 8 mm dia, at 150 c/c remaining height

Block 2

Table 7-73: Detail of Beam Reinforcement of Workshop Building Block 2

Floor/ level	Beam Dimension B mm x D mm	Reinforcement		
		Longitudinal		Shear
		Top	Bottom	
EL. 1011.10	300 x 500	3-20 mm dia. Through 2 – 20 mm dia. Extra	3- 16 mm dia. Through	4 legged 10 mm dia. At 75 c/c at the ends (500 mm from face of column) and 4 legged 10 mm dia. at 150 c/c remaining span
EL. 1011.10 Crane beam	1200 x 600	12-16 mm dia. through	12-16 mm dia through	4 legged 10 mm dia. At 150 C/C
Roof	300 x 500	3- 25 mm dia. Through 2- 25 mm dia Extra	3- 25 mm dia. Through	4 legged 10 mm dia. At 75 c/c at the ends (500 mm from face of column) and 4 legged 10 mm dia. at 150 c/c remaining span

Table 7-74: Detail of Column Reinforcement of Workshop Building Block 2

Type	Column Dimension B mm X D mm	Reinforcement	
		Longitudinal	Confinement
Crane supporting column	700 x 1000	16-25 mm dia.	10 mm dia, at 125 c/c at ends (1000 mm from face of beam) and 10 mm dia, at 150 c/c remaining height
Other column	600 x 600	14-20 mm dia	10 mm dia, at 150 c/c at ends (600 mm from face of beam) and 10 mm dia, at 200 mm C/C remaining height
Columns above crane beam level	700 x 700	16 – 20 mm dia	10 mm dia, at 125 c/c ends (600 mm from face of beam) and 10 mm dia, at 200 mm C/C remaining height

Reinforcement for 200 m slabs

Block 1 building 10 mm dia. bars at top and bottom is provided at 150 mm C/C

Block 2 building 12 mm dia. Bars at top at bottom is provided at 130 mm C/C.

Reinforcement for raft 300 mm/500 mm foundation

16 mm dia.@ 200 mm C/C was chosen for both direction top and bottom.

The dog legged stair case is proposed on the block 1 of workshop building, Staircase was designed with waist slab thickness of 220 mm with 16 mm dia. main bar at 150 mm c/c and 10 mm dia. bar at 200 mm c/c is provided as distribution bar.

8 HYDROMECHANICAL DESIGN

8.1 General

8.1.1 Introduction

The hydromechanical equipment described in this chapter consists of the following main components:

- One Flap Gate in the spillway terminal structure
- One Pressure Door for the headrace tunnel
- One Butterfly Valve in the valve chamber
- One Overhead Bridge Crane in the valve chamber
- Steel Lining
- One Fixed-Wheel Gate in the outlet structure
- One tailrace outlet rack in the outlet structure
- One mobile crane

The locations of the individual components are shown in the Album of Drawings, Part E of the Detailed Engineering Design Report. Details about the construction features and special tools are provided in Part A3 Chapter 6 of this report.

8.1.2 Standards and Basic Design Criteria

The hydromechanical equipment shall be designed according to international standards. For example, DIN 19704, ISO, EN, ASTM, ASME, IEC, etc.

8.1.2.1 Material and Workmanship

All materials incorporated in the works shall be according to the above-mentioned international standards and the most suitable for the duty concerned. The materials shall be new and of first class commercial quality, free from imperfection and selected for a long-life span and minimum maintenance.

The composition and properties of the materials shall be verified by chemical analysis and non-destructive testing. The testing shall be specified, in detail, in the tender documents.

For highly stressed fabricated constructions, especially those subjected to headwater pressure, only materials fully normalized or of quenched and tempered quality and of high durability (including low temperature ductility and age resistant), shall be used. After the fabrication, the construction shall be subject to heat treatment for stress relieving.

Important steel castings, in particular for constructions subjected to headwater pressure, shall be ductile, even at low temperatures. After completion of the heat treatment and the pre-machining, the casting shall be subject to non-destructive testing.

Important forgings e.g. shafts shall be of vacuum-degassed steel and forged thoroughly and through going. Forging to shape, heat treatment and pre-machining to raw dimensions shall be completed before material testing.

8.1.2.2 Design Stresses

All components shall be designed for a minimum service life of 50 years under normal operating conditions. Additionally, components subject to fatigue shall also be designed with a minimum safety factor of 1.5 against failure unless otherwise stated.

For the design, the loading shall be divided into 3 different classes as defined in Table 8-1. The stress limits shall be in accordance with DIN 19704 or Table 8-2.

Stress analysis and fatigue analysis shall be performed considering dynamic loads and earthquake loads.

The following seismic coefficients shall be used. For underground structures at a depth greater than 50 m the seismic coefficients can be reduced by 50%. Between a depth of 0 to 50 m the reduction factor shall be interpolated.

	Horizontal	vertical
Operation Basis Earthquake	0.222	0.2
Maximal Credible Earthquake	0.34	0.31

Table 8-1: Definition of loading types

Loading type	Definition	
Normal Loading	Permanents loads	Loading due to dead loads Hydrostatic loads
	Transient loads	Hydrodynamic loads Loading due to water hammer Environmental loads (50 years return period) Pressure rise at design floods etc. Dynamic loads dueto normal operation
Unusual Loading	Permanents loads + Operation Basis Earthquake loading Permanents loads + Sediment load Loading due to transportation and erection	
Extreme Loading	Permanents loads + Maximal Credible Earthquake (MCE) Permanents loads + Operation Basis Earthquake (OBE) + Sediment Permanents loads + Extreme floods (PMF, Glacier Outburst Flood, etc.) Pressure tests or load tests	

Table 8-2: Stress limit for several design conditions

Design Condition	Material	Stress Condition	Stress Limit
Normal	Structural steel	Maximum general stress level	67 % of the yield strength
	Structural steel	Average stress at points with local stress concentrations, calculated according to methods approved by the Engineer	75 % of the yield strength, provided that the quality of material can be confirmed by approved methods at the point with stress concentration.
	Cast steel	Maximum general stress level	50 % of the yield strength
	Cast steel	Average stress at points with local stress concentrations, calculated according to methods approved by the Engineer	67 % of the yield strength, provided that the quality of material can be confirmed by approved methods at the points with stress concentration
	Cast iron	Maximum tensile stress level	20 N/mm^2 (20 < t < 30 mm)
Unusual	Structural steel	Maximum general stress level	75% of the yield strength
	Structural steel	Average local stress concentrations, provided that the material is not subject to fatigue	90% of the yield strength
Extreme	Structural steel	Maximum general stress level	90% of the yield strength
	Structural steel	Average local stress concentrations, provided that the material is not subject to fatigue	100% of the yield strength

8.1.2.3 Welding

Prior to its commencing the welding method selected shall be surveyed, especially regarding the applicability. Welding procedure specification (WPS) shall be issued for all welding .

All welding materials used shall be of the highest quality.

Prior to performing any welding, welders engaged shall pass the welder's qualification tests according to international standards and codes.

8.1.2.4 Maintenance Work

All components of the equipment shall be designed for easy maintenance work. Components subject to wear and tear shall be accessible and replaceable. Necessary special tools for this maintenance and replacement work shall be provided.

8.1.2.5 Spare Parts

Priority shall be given to the use of standard componentsand equipment in order to facilitate keeping stocks, maintenance, replacement, inter-changeability and quick delivery, etc. The Contractor shall provide general spare parts, which is necessary for 5 years of operation..

The scope of supply of the spare parts shall be specified in detail in the tender documents.

8.1.2.6 Pipe

Unless otherwise specified, pipe shall be designed for PN 10. Standard pipe shall be used as far as possible.

Water pipe with a nominal diameter less than DN 200 shall be of stainless steel, since corrosion protection inside of pipe cannot be carried out sufficiently, especially for repair work. For connection of water pipe with dissimilar metals, insulating unions shall be provided to prevent the passage of more than 1% of the galvanic current, which would exist with metal-to-metal contact.

Pressured oil pipe systems shall generally be of stainless steel.

Water pipe connecting the operation water supply system shall be executed via weld-in standard flange. For safety reasons, connections which have nominal diameters more than DN 200 upstream of the main inlet valves shall be equipped with two shut-off valves in series arranged as close as possible to the connection.

8.1.2.7 Corrosion Protection

After manufacturing the components/equipment it shall be carefully cleaned and protected by the application of an adequate corrosion protection system. For surfaces in contact with water, the painting material shall conform to the latest international and national environmental requirements.

Material subject to corrosion and/or abrasion shall be designed to have 2 mm additional thickness.

In the tender documents methods for surface treatment and conservation, as well as the minimum required coating thickness of the painting material and its examination, shall be specified in detail.

8.1.2.8 Quality Assurance, Control and Testing

During the manufacturing, assembly, erection and commissioning, control and testing shall be carried out on all materials, components and assemblies to confirm that the material and the manufacturing methods used as well as the equipment furnished are adequate and fully comply with the requirements and guarantees.

As a minimum, the following control and testing shall be carried out in the workshop and at Site:

- Material test (chemical analysis, mechanical properties, crack detection on steel plates and castings)
- Weld inspection (liquid penetrant testing, magnetic-flux testing, ultrasonic testing, time of flight diffraction, radiographic testing if necessary)
- Pressure test at 150% of the design pressure (Pressure vessels, piping, etc.)

- Run-out control (shaft, etc., individual and together)
- Balancing control (turbine runner, individual and together with the generator rotor)
- Dimension and surface roughness measurement
- Function tests (equipment, individual and together)
- Turbine model tests (optional)

The tender documents shall contain the detailed requirements about the scope of the inspection and testing to be performed by the Contractor in the workshop and at Site.

8.1.2.9 Control and Safety Devices

The hydromechanical equipment and its auxiliaries shall be equipped with control, measuring and safety devices as well as indicating instrumentation, which are necessary for both manual and fully automatic operation. All conditions of importance for safe and proper operation shall be monitored.

Pressure taps shall be carried out in accordance with IEC 60041, Clause 11.4.3. A shut-off valve of stainless steel shall be arranged as close as possible to the tap. Next to the pressure transmitter and indicator, a quick release coupling for connecting mobile manometer shall be provided.

8.2 Flap Gate at Spillway Terminal Structure

8.2.1 General

In order to prevent scree and sediment from entering the spillway tunnel and for inspection and maintenance of the spillway tunnel, a flap gate shall be installed in the spillway terminal structure.

8.2.2 Design Criteria and Operation Conditions

The main parameters for designing the flap gate are defined as following:

• Number of opening	1	
• Type of gate to be provided	flap gate	
• Number of gates to be provided	1	
• Number of embedded parts to be provided	1	
• Clear width of gate opening	4.40	m
• Clear height of gate opening	4.35	m
• Sill elevation of gate	EL.1150.15	m asl.
• Extreme water level	EL.1160.25	m asl.
• Design head	0.1	MPa

The flap gate shall be designed and constructed in accordance with DIN 19704 (all 3 parts).

The flap gate will isolate the spillway tunnel from the Tamakoshi river during extreme flood eventsto prevent incoming of scree and sediment. The flap gate will also provide access for inspection and maintenance. Generally, the gate will remainopen. It will be only closed, if the water level in Tamakoshi river reaches EL.1157.5 m asl., or for inspection and maintenance work in the spillway tunnel.

The flap gate shall be closed by its own weight under approximately balanced condition. It shallbe opened by an oil servomotor, also under the condition that on the downstream side there is 1 m water with/without sediments above the gate sill elevation. Both the closing and the opening time shall be adjustable.

If the spillway tunnel inadvertently becomes pressurised and the flap gate is in the fully closed position, the flap gate shall be designed to automatically lift up for pressure release without any damage on the gate up to the maximum flow of 68 m^3/s.

The flap gate shall be designed and constructed that no vibration occurs under the any operating conditions and that the working stresses shall not exceed the maximum allowable design values given in Table 8-2.

The flap gate shall be designed to <u>close</u> locally or remotely. <u>Opening</u> shall be designed to only be locally performed due to safety reasons.

8.2.3 Main Components

The flap gate shall consist of following main components:

- Gate frame and steel lining
- Gate leaf
- Gate seals
- Gate operation mechanism
- Oil pressure unit
- Water level switches
- Local control system

8.2.4 Instrumentation and Safety Devices

The flap gate shall be furnished with all instruments, control, etc., necessary for manual, automatic and supervision of the operation. All conditions of importance for operation shall be monitored.

At a minimum, the following measuring devices as well as monitoring and indicating instruments shall be provided. The Contractor shall add instrumentation and/or safety device, where it is required.

- Gate leaf position
- End positions of the oil-hydraulic operated mechanical locking device, if applicable
- Oil pressure unit as specified
- Supervision of “lifting rod breaking”

In the event of a non-intentional closing of the flap gate an alarm shall be designed to acitivate

8.3 Door Type Gate at Adit 4

8.3.1 General

The headrace adit gate shall isolate the Adit 4 from the headrace tunnel. After emptying the headrace tunnel, the adit gate can be opened for inspection and maintenance of the headrace tunnel.

8.3.2 Design Criteria and Operation Conditions

The main parameters for designing the headrace adit gate are defined as following:

• Number of opening	1	
• Type of gate to be provided	door type	
• Number of gates to be provided	1	
• Number of embedded parts to be provided	1	
• Clear width of gate frame opening	2.1	m
• Clear height of gate frame opening	2.8	m
• Sill elevation of gate	approx. EL. 1113.4	m asl.
• Design head (preliminary)	0.7	MPa
• Design sediment level	1 m above sill elevation	

The gate shall be designed and constructed in accordance with DIN 19704 (all parts). The final design head shall be determined by the transient calculations, which shall be performed by the turbine supplier.

In normal operation of Tamakoshi V HEP, the headrace adit gate shall be in the closed position. It shall be ensured that there is no leakage through the gate seals and the steel structure.

The gate shall be rotatable more than 90° around hinged bearing towards headrace tunnel. It shall be so designed and constructed that no vibration occurs under the operating conditions and that the working stresses shall not exceed the maximum allowable values (see Table 8-2). The bearing shall be of self-lubricating type. The axels shall be of stainless steel.

The gate shall be manually operated.

8.3.3 Main Components

The headrace adit gate shall consist of following main components:

- Gate frame and steel lining
- Gate body
- Gate seals
- Drainage system

8.3.4 Instrumentation and Safety Devices

A manometer for measuring the water pressure upstream of the gate shall be arranged at an adequate location for safe opening of the gate body.

8.4 Butterfly Valve at U/S Valve Chamber

8.4.1 General

Between the surge tank and the pressure shaft, a butterfly valve of through-flow type shall be installed. It shall have state of the art design according to international standards. The valve shall have a service seal.

8.4.2 Design Criteria and Operation Conditions

The main parameters for designing the butterfly valve are defined as following:

- Number of valves to be provided 1
- Type of valve to be provided butterfly, bi-plane
- Installation elevation of valve axis EL.1114.958 m asl.
- Nominal diameter 4200 mm
- Design head 0.7 MPa
- Design flow 66 m^3/s
- Maximum flow 68 m^3/s

The butterfly valve will isolate the pressure shaft and the upstream manifolds from the headwater way for inspection and maintenance. In addition, it will protect the powerhouse cavern against flooding, if there should be pipe burst upstream of the main inlet valves. Therefore, the butterfly valve and its operation mechanism shall be designed for closing under the design flow specified below.

Generally, the butterfly valve will remain in the fully opened position. It will be closed for inspection and maintenance of the pressure shaft and the upstream manifolds as well as for cavern flooding protection. Normally, opening and closing will be under approximately balanced condition with the turbine guide vanes in the closed position.In the event of cavern flooding, the butterfly valve will close under flow up to the design flow.

The butterfly valve shall be designed to close by closing weights and opened by oil servomotors. It shall be designed to operate via the control systems or manually at the oil pressure unit included in the scope of supply.

The butterfly valve shall be designed and constructed that no vibrations occur under the various operating conditions and that the working stresses shall not exceed the maximum allowable values (see Table 8-2).

The opening and closing time of the butterfly valve shall be designed to be independently adjustable between 30 s and 120 s. The closing time shall be adjusted that the momentary pressure caused at the valve will not be higher than the design pressure. To avoid any vacuum condition in the pressure shaft and the upstream manifolds, the closing of the valve shall only be started, if the guide vanes or related main inlet valve of each turbine-generating unit are in the closed position.

The main inlet valve will be installed/dismantled by using the crane installed in the valve chamber.

8.4.3 Main Components

The butterfly valve will consist of following main components:

- Valve body and support foundation
- Valve rotor and trunnions
- Trunnion sealing and bearing
- Valve seal
- Valve operation mechanism
- Oil pressure unit
- Extension pipe
- Dismantling pipe
- Bypass
- Local control system

8.4.4 Instrumentation and Safety Devices

The butterfly valve shall be furnished with all instruments, control, etc., necessary for manual, automatic for supervision of the operation. All conditions of importance for the operation shall be monitored.

The following measuring devices as well as monitoring and indicating instruments shall be provided as minimum. The Contractor shall add instrumentation and/or safety device, where it is required.

- Pressure upstream and downstream of the butterfly valve
- Valve rotor position
- Oil pressure unit as specified
- End positions of the bypass operation valves

Non-intentional closing of the butterfly valve will cause a quick stop of the all turbine generating units in the powerhouse cavern.

8.5 Overhead Bridge Crane in U/S Valve Chamber

8.5.1 General

To facilitate assembly, erection and dismantling work of the equipment installed in the valve chamber, an overhead bridge crane shall be provided. All mechanical and electrical equipment required shall be provided, including cables, cable conduits and terminals.

8.5.2 Design Criteria and Operation Conditions

The overhead bridge crane shall be of double girder type and designed according to international standards for example, EN, FEM, etc.

The crane shall have a main and an auxiliary hook. The heaviest component to be lifted will be the butterfly valve without the valve operation mechanism. Thus, the rated capacity of the hook shall be optimised by the Contractor and approved by the Employer/Engineer.

Preliminarily, the overhead bridge crane has the following main parameters:

•	Number of cranes		1	
•	Rated lifting capacity	main hook	100	t
		auxiliary hook	15	t
•	Span		9.6	m
•	Lifting height	main hook	9	m
		auxiliary hook	11	m
•	Rail base elevation		approx. 1022	m asl.
•	Rail track length		approx. 19	m
•	Bridge travelling speed	normal	20	m/min
		creeping	0.6	m/min
•	Trolley travelling speed	normal	15	m/min
		creeping	0.6	m/min
•	Main hook lifting speed	normal	1.2	m/min
		creeping	0.1	m/min
•	Auxiliary hook lifting speed	normal	4	m/min
		creeping	0.5	m/min

The crane shall be remotely controlled by using radio-operated portable control panel. Operator’s cabin is not necessary and shall not be provided to minimise project costs.

The crane shall be tested at 125% of the rated lifting capacity at Site. The testing load shall be provided and removed after the testing by the Contractor. The deflection of the bridge structure under the rated lifting capacity shall not exceed 1/1000 of the span.

8.5.3 Main Components

The overhead bridge crane will have following main components:

- Runway rails
- Crane bridge
- Trolley
- Lifting equipment
- Brakes
- Power supply

- Control equipment

Along the crane runways as well as on the girders and trolley, handrails with provisions for safety shall be supplied. Vertical height of the handrails above the footing plate shall be not less than 1000 mm. There shall be middle rails, approximately 450 mm above the footing, and breast boards of at least 70 mm height at the bottom of the handrails.

8.5.4 Instrumentation and Safety Devices

The powerhouse crane shall be furnished with all instruments, control, etc., necessary for manual, automatic and supervision of the operation. All conditions of importance for operation shall be monitored.

At a minimum the following measuring devices and monitoring and indicating instruments shall be provided. The Contractor shall add instrumentation and/or safety device, where it is required.

- Overload protection for the hoist
- Indicator of the lifted weight
- Protection for the motors

8.6 Steel Lining

8.6.1 General

The headwater way from the headrace tunnel transition starting at the elevation EL.1115.713 m asl. downstream of the surge tank to the end of the upstream manifolds at the elevation EL.974.0 m asl. shall be completely steel lined. The nominal diameter and length of the individual steel lining sections are shown in the Album of Drawings, Part E of the Detailed Engineering Design Report.

8.6.2 Design Criteria and Operation Conditions

The steel lining shall be fully encased in concrete except for the section in the valve chamber. It shall be designed both for the maximum inner pressure including the water hammer and for the outer pressure due to the groundwater.

The contribution of the rock surrounding the steel liner will not be considered in dimensioning the steel liner.The design calculation for the steel lining shall be based on international methods like:

- Design procedures suggested by S. Jacobsen and E. Amstutz published in the “Water Power and Dam Construction” in 1970, Jan. & Dec. 1974, Dec. 1977, June 1983, April 1990, etc.
- ASCE: Steel Penstocks – Steel Penstock Manuals and Reports on Engineering Practice No.79
- Comité Européen de la Chaudronnerie et de la Tolerie (CECT)

Other calculation standards and codes proposed by the Contractor shall be approved by the Employer/ Engineer.

The internal design pressure at the end of the upstream manifolds shall be assumed to be identical to that for the turbine spiral case, which shall be according to the transient calculations to be performed by the turbine supplier. A value of about 2.1 MPa is expected.

According to the topographical investigation, the external pressure will be affected by the water level in the surge tank.

The design flow of the steel lining will be 66 m³/s for the part of common waterway.

Non-ageing fine grain steel according to EN 10113 or equivalent shall be used for the steel lining. The final selection of the material shall be made by the Contractor based on his experience and approved by the Employer/Engineer.

The following safety factors shall be applied for steel lining against internal pressure:

- From start of lower bend to end of upstream manifolds: 2.0
- From start of transition to start of lower bend: 1.5

The critical external buckling pressure shall be a minimum 1.5 times higher than the design pressure. The ovality factors shall be according to CECT. The initial gap between the steel lining and the surrounding concrete shall be finalised by the Contractor. Preliminary, 1.5 mm is assumed.

8.6.3 Main Components

The steel lining will have following main components:

- Steel lined headrace tunnel
- Pressure shaft
- High pressure tunnel
- Upstream Manifolds

The steel lining shall be sufficiently stiffened. Each section shall have adequate supports and anchors for transportation, levelling and fixing during the installation and concrete back-filling. The sections shall be prepared as far as practicable in a workshop including:

- Cutting the plates to exact size,
- Preparing edges for welding,
- Rolling to required shape,
- Holes for concrete grouting and related closing threaded plugs, if required.

All internal welds shall be hydraulic-smoothly ground. The variation of plate thickness at section joints between adjacent plates shall be limited to a maximum 2 mm.

NDT shall be performed for all welds, especially for those carried out at Site.

8.7 Fixed Wheel Gate at Outlet Structure

8.7.1 General

To keep the tailrace tunnel free from scree and sediments as well as for providing inspection and maintenance of the tailrace tunnel, a fixed-wheel gate shall be installed in the outlet structure.

8.7.2 Design Criteria and Operation Conditions

The main parameters for designing the fixed-wheel gate are defined as following:

• Number of opening	1	
• Type of gate to be provided	vertical, fixed-wheel	
• Number of gates to be provided	1	
• Number of embedded parts to be provided	1	
• Clear width of gate opening	4.4	m
• Clear height of gate opening	5.6	m
• Sill elevation of gate	974.0	m asl.
• Water level at rated turbine discharge	984.0	m asl.
• Extreme water level	997.9	m asl.
• Design sediment level	976.0	m asl.

The fixed-wheel gate shall be designed and constructed in accordance with DIN 19704 (all 3 parts).

The fixed-wheel gate will isolate the tailrace tunnel and downstream manifolds from the Tamakoshi river during extreme floods to prevent incoming of scree and sediment as well as for inspection and maintenance. Generally, the gate will remain in the open position. It will be only closed, if the water level in Tamakoshi river reaches EL.986.50 m asl., or for inspection and maintenance work in the tailrace tunnel and downstream manifolds.

The fixed-wheel gate shall be closed by its own weight under balanced condition. It shall be opened by an oil servomotor, also under the condition that on the downstream side there is 1 m water with/without sediments above the gate sill elevation. Both the closing and the opening time is adjustable.

The fixed-wheel gate shall be so designed and constructed that no vibration occurs under any operating conditions and that the working stresses shall not exceed the maximum allowable values (see Table 8-2).

The fixed-wheel gate shall be designed to close locally or remotely. Opening should only be possible locally due to safety reason.

8.7.3 Main Components

The fixed-wheel gate will consist of following main components:

- Gate body
- Gate wheels
- Gate seals
- Embedded parts
- Cover plate
- Gate operation mechanism
- Oil pressure unit
- Local control system

8.7.4 Instrumentation and Safety Devices

The fixed-wheel gate shall be furnished with all instruments, control, etc., necessary for manual, automatic and supervision of the operation. All conditions of importance for operation shall be monitored.

The following measuring devices as well as monitoring and indicating instruments shall be provided as minimum. The Contractor shall add instrumentation and/or safety device, where it is required.

- Gate body position
- End positions of the oil-hydraulic operated mechanical locking device, if applicable
- Oil pressure unit as specified
- Redundant monitoring of “lifting rod breaking”

A master float switch shall be provided for emergency closing of the fixed-wheel gate via a hydraulic valve.

Non-intentional closing of the fixed-wheel gate shall initialize quick stop of all turbine-generating units.

8.8 Trash Rack at Outlet Structure

8.8.1 General

The outlet of the tailrace tunnel shall be protected by coarse trash rack to prevent people or larger animals from entering the tunnel when the turbines are not operating. Additionally, it can prevent debris from entering the tailrace culvert during extreme floods.

8.8.2 Design Criteria and Operation Conditions

The main parameters for designing the coarse rack are defined as following:

- Number of openings 1
- Inclination of coarse rack from vertical 0°

• Set of coarse racks to be provided	1	
• Set of embedded parts to be provided	1	
• Clear width of outlet opening	4.4	m
• Clear height of outlet opening	5.6	m
• Sill elevation of coarse rack	974.0	m asl.
• Clear rack bar spacing	150	mm
• Design differential head	3	m
• Maximum flow through coarse rack	68	m^3/s

The coarse trash rack of normal steel shall be designed and constructed in accordance with DIN 19704 (all 3 parts). All relevant factors having a bearing on the calculations and design, e.g. equally distributed water pressure, water velocity, erection and concreting procedures, forces during operation and maintenance. Sediment load and earthquake loads need not be considered.

The coarse trash rack shall be so designed and constructed that no vibration occurs under any operating conditions. The nature! frequency of the rack bars shall be at least 2.5 times the vortex shedding frequency. This also applies even if the structural calculation permits larger distances between the horizontal stiffeners.

The design shall ensure smooth water flow, minimum head losses and high abrasion resistance.

With the exception of completely embedded parts, the coarse trash rack shall be hot-dip galvanised. Fixing material shall also be galvanised. Minor damage due to installation etc. shall be adequately repaired.

8.8.3 Main Components

The coarse rack will consist of following main components:

- Six rack panels assembled of rack bars and stiffeners, including connecting material
- Embedded parts, including a horizontal beam

8.9 Mobile Crane

8.9.1 General

For installation and remove of the equipment in the spillway terminal structure and in the outlet structure, a mobile crane shall be provided. It shall be parked in a garage and driven via public road and access road to the equipment destination. Details are shown in the Album of Drawings, Part E of the Detailed Engineering Design Report.

8.9.2 Design Criteria and Operation Conditions

One mobile crane shall be provided. It shall be an approved manufacturer's standard commercial product meeting all applicable requirements for highway travel and capable of travelling on rough terrain with high cross slope. The mobile crane shall be fitted with service disc brakes, which shall stop and hold the crane on an 80% slope, and parking brake, which shall restrain the crane on at least a 45% longitudinal slope.

The mobile crane shall be equipped with multipurpose, all traction tread pneumatic tyres and able to reach a travel speed of minimum 50 km/h over a paved surface.

The mobile crane shall be able to operate under any weather conditions, which can be expected throughout the Project area.

The technical data of the mobile crane shall be proposed by the Contractor and approved by Employer/ Engineer after calculation of the heaviest weight to be lifted. Preliminary, a mobile crane with 35 t rated lifting capacity is selected. A section of the fixed-wheel gate installed in the outlet structure shall be the heaviest weight to be handled by the mobile crane.

8.9.3 Main Components

The mobile crane shall consist of a continuously revolving superstructure with boom support, hydraulically operated telescoping boom, main and auxiliary winch with wire rope, hook block, cab, controls and necessary accessories. The crane shall allow different configurations to permit connection and operation with different hardware parts as required for various operations.

The mobile crane shall be driven by a diesel engine, provided with all systems and accessories, which shall include a cold-weather starting aid and accessible oil-sampling valve, a commercial power shift, automatic transmission, or equivalent and shall be all-wheel drive.

The mobile crane shall be equipped with selected modes for front-wheel, all-wheel, and crab steering, and an emergency/secondary steering system. An in-the-cab indicator shall provide at-a-glance presentation of the steering position of the wheels.

The mobile crane shall be equipped with adequate operator-adjustable lighting for night operations.

The mobile crane shall be equipped with a Load Moment Indicating (LMI) system in accordance with standard commercial industry practices, using existing crane electrical power and able to operate under all specified climatic conditions.

The mobile crane shall incorporate the commercial standard capability for fault identification and diagnostics or Diagnostic Connector Assembly (DCA), with an easily accessible fault identification and diagnostics or DCA in the operator's cab.

9 POWER STATION MECHANICAL EQUIPMENT

9.1 General

9.1.1 Introduction

The mechanical equipment described in this chapter consists of the following main components:

- Three Francis Turbines
- Three Turbine Governors
- Three Main Inlet Valves
- Three Draft Tube Flap Gates
- Cooling Water System
- Drainage and Dewatering System
- Compressed Air System
- Oil Handling System
- One Powerhouse Crane
- One Dual Purpose Cargo and Passenger Elevator
- Mechanical Workshop Equipment
- One Small Hydro Unit
- HVAC + Sanitary + Sewerage

All above-mentioned equipment will be installed in the powerhouse cavern of Tamakoshi V Hydroelectric Project (HEP). The locations of the individual components are shown in the Album of Drawings, Part E of the Detailed Engineering Design Report. Details about the construction features and special tools are provided in Part A3 Chapter 7 of this report.

9.1.2 Operation Conditions

Tamakoshi V HEP is to be operated “in tandem” with Upper Tamakoshi HEP (UTK), i.e. the operation of Tamakoshi V HEP is solely determined by the discharge released from UTK.

UTK is designed as a power plant for peak loads. UTK is equipped with six Pelton units, each designed for a rated discharge of 11 m³/s. Beside the daily peaking, UTK will operate one unit with a discharge up to the minimum of 1.5 m³/s as Spinning Reserve. In addition, it is intended to perform inspection on the turbine-generating units in UTK one by one every year during the dry season.

The mechanical equipment of Tamakoshi V HEP is designed for operation in the generating mode. It is expected to start/stop each unit 4 times per day. Synchronous condenser operation is not required. The “speed no-load” mode is possible and will enable the dewatering of the headrace tunnel and the surge tank.

The power station will be equipped with batteries and an emergency diesel engine.

Usually, the power station will be manned, and the turbine-generating units will be remotely operated and controlled from the central control room in the operation building. In addition, operation and control at the unit local control boards (UCB) are possible. Manual operation of individual equipment, e.g. main inlet valves, pumps, etc., can be locally carried out

9.1.3 Standards and Basic Design Criteria

The mechanical equipment will be designed according to international standards and codes. For example, IEC, ISO, EN, ASTM, ASME, etc.

9.1.3.1 Material and Workmanship

All materials incorporated in the mechanical equipment will be according to the above-mentioned international standards and the most suitable for the duty concerned. The materials will be new and of first class commercial quality, free from imperfection and selected for a long-life span and minimum maintenance.

The composition and properties of the material will be verified by chemical analysis and non-destructive testing, which will be specified in detail in the tender documents.

For highly stressed fabricated, constructions, especially those subjected to headwater pressure, only materials fully normalized or of quenched and tempered quality and of high durabilty including low temperature ductility, age resistant will be used. After the fabrication, the construction will be subject to heat treatment for stress relieving.

Important steel castings, in particular for constructions subjected to headwater pressure, will be ductile, even at low temperatures. After completion of the heat treatment and the pre-machining, the casting will be subject to non-destructive testing.

Important forgings e.g. shafts will be of vacuum-degassed steel and forged thoroughly and through going. Forging to shape, heat treatment and pre-machining to raw dimensions will be completed before material testing.

9.1.3.2 Design Stresses

Factors of safety according to international codes will be used throughout the design. The design of all parts subject to alternating stresses, impact or shock will be designed particularly robust.

Under usual loading, the maximum equivalent stress will not exceed 50% of the yield strength.

Under unusual loading and hydrostatic test pressure, the maximum equivalent stress will not exceed 75% of the yield strength.

Stress analysis and fatigue analysis will be performed considering dynamic loads and earthquake loads.

9.1.3.3 Welding

Prior to its commencing, the welding method selected will be surveyed, especially regarding its applicability. Welding procedure specification (WPS) will be issued for all welding .

All welding materials used will be of the highest quality.

Prior to performing any welding, welders engaged will pass the welder's qualification tests according to international standards and codes.

9.1.3.4 Maintenance Work

All components of the equipment will be designed for easy maintenance work. Components subject to wear and tear will be accessible and replaceable. Necessary special tools for this maintenance and replacement work will be provided.

9.1.3.5 Spare Parts

Priority will be given to the use of standard components and equipment in order to facilitate keeping stocks, maintenance, replacement, inter-changeability and quick delivery, etc.

The Contractor will provide general spare parts, which will be necessary for 5 years of operation, in accordance with his experience.

The scope of supply of the spare parts will be specified, in detail, in the tender documents.

9.1.3.6 Pipe

Unless otherwise specified, pipe will be designed for PN 10. Standard pipe will be used.

Water pipe with a nominal diameter less than DN 200 will be of stainless steel, since corrosion protection inside of pipe cannot be carried out sufficiently, especially for repair work. For connection of water pipe with dissimilar metals, insulating unions will be provided to prevent the passage of more than 1% of the galvanic current, which would exist with metal-to-metal contact of dissimilar metals.

Pressured oil pipe systems will generally be stainless steel.

Water pipe connecting the operation water supply system will be executed via weld-in standard flange. For safety reasons, connections which have nominal diameters more than DN 200 upstream of the main inlet valves will be equipped with two shut-off valves in series arranged as close as possible to the connection.

9.1.3.7 Corrosion Protection

After manufacturing the components/equipment it will be carefully cleaned and protected by the application of an adequate corrosion protection system. For surfaces in contact with water the painting material will conform to the latest international and national environmental requirements.

Material subject to corrosion and/or abrasion will be designed to have 2 mm additional thickness.

In the tender documents methods for surface treatment and conservation, as well as the minimum required coating thicknesses of the painting material and its examination, will be specified in detail.

9.1.3.8 Quality Assurance, Control and Testing

During the manufacturing, assembly, erection and commissioning, control and testing will be carried out on all materials, components and assemblies to confirm that the material and the manufacturing method used as well as the equipment furnished are adequate and fully comply with the requirements and guarantees.

As a minimum, the following control and testing will be carried out in the workshop and at Site:

- Material test (chemical analysis, mechanical properties, crack detection on steel plates and castings)
- Weld inspection (liquid penetrant testing, magnetic-flux testing, ultrasonic testing, time of flight diffraction, radiographic testing if necessary)
- Pressure test at 150% of the design pressure (Pressure vessels, piping, etc.)
- Run-out control (shaft, etc., individual and together)
- Balancing control (turbine runner, individual and together with the generator rotor)
- Dimension and surface roughness measurement
- Function tests (equipment, individual and together)
- Turbine model tests (optional)

The tender documents will contain the detailed requirements about the scope of the inspection and testing to be performed by the Contractor in the workshop and at Site.

9.1.3.9 Control and Safety Devices

The power station mechanical equipment and its auxiliaries will be equipped with control, measuring and safety devices as well as indicating instrumentation, which are necessary for both manual and fully automatic operation. All conditions of importance for safe and proper operation will be monitored. In case of a failure, an alarm signal will notify the operator, and if a dangerous operating condition occurs, normal, quick or emergency shutdown of the turbine-generating unit will be initiated.

Pressure taps will be carried out in accordance with IEC 60041, Clause 11.4.3. A shut-off valve of stainless steel will be arranged as close as possible to the tap. Next to the pressure transmitter and indicator, a quick release coupling for connecting mobile manometer will be provided.

9.2 Francis Turbine

9.2.1 General

The powerhouse cavern of Tamakoshi V HEP will be equipped with three main turbine-generating units, each consisting of a Francis turbine and a synchronous generator rigidly coupled by vertical shafts. In Addition, a small hydro turbine-generating unit described in Subchapter 7.13 will be installed to exploit the minimum water flow discharged by UTK. The small hydro unit will only operate if at least one turbine is in standby.

The water coming from UTK will be conveyed through the connecting tunnel to the head pond of Tamakoshi V HEP. Then, water flows through the headrace tunnel (5.6 m inner diameter, about 8.2 km length), the steel-lined pressure shaft and high-pressure-tunnel (4.2 m inner diameter, about 180 m total length) as well as the upstream manifolds to the turbines. Downstream of the turbines, the water in the downstream manifolds goes through the tailrace tunnel (5.6 m nominal diameter, about 480 m length) to the outlet structure.

Each turbine will be equipped with a main inlet valve of butterfly type at the inlet of the spiral case. In addition, a butterfly valve will be arranged between the surge shaft and the pressure shaft. On the downstream side, each turbine can be isolated from the tailrace tunnel by a draft tube flap gate. At the outlet structure of the tailrace tunnel, fixed wheel gates will be installed for closing the tailrace tunnel.

9.2.2 Water Levels and Gross Heads

The following headwater levels will be used for the layout of the turbines of Tamakoshi V HEP:

- Maximum headwater level 1158.2 m asl
- Rated headwater level 1158.0 m asl
- Minimum headwater level 1155.0 m asl

The tailwater levels for the layout of the turbines are defined as following:

- Maximum tailwater level 986.35 m asl
- Rated tailwater level 984.55 m asl
- Minimum tailwater level 982.80 m asl

The maximum tailwater level is conditioned by the operation of UTK, while the minimum tailwater level is conditioned by the tailrace outlet structure.

Thus, the gross heads for the turbine operation are

	maximum	rated	minimum
Gross Head in (m)	175.40	173.45	168.65

9.2.3 Head Losses and Net Head

As described in Chapter 2.2, the total head losses are computed as 11.1 m for the rated condition. Thus, the rated net head is determined as (173.45 – 11.1 =) 162.35 m.

The maximum net head will be reached, if only one unit is operating with the minimum permissible discharge guaranteed by the turbine supplier. A maximum net head of about 175 m is expected.

9.2.4 Turbine Discharge

With all three turbine-generating units in operation, each turbine will be able to operate smoothly and continuously with the rated discharge of 22 m³/s at the rated head.

Each turbine will be able to operate smoothly and continuously from 105% to 40% of the rated discharge within the entire head range. The minimum discharge for continuous operation is expected to be below 8 m³/s, the final value will be guaranteed by the turbine supplier.

9.2.5 Rated Turbine Output

The rated turbine output can be calculated as follows:

$$P_T = \rho\, g\, H\, Q\, \eta_T$$

where:
- ρ = density of water = 999 kg/m³ according to IEC 60193;
- g = acceleration due to gravity = 9.79 m/s² according to IEC 60193;
- H = rated net head = 162.35 m;
- Q = rated turbine discharge = 22 m³/s
- η_T = turbine efficiency at rated condition = 92.8% (assumed).

Applying the values mentioned above, the rated turbine output will be 32.42 MW. The final value of the rated turbine output will be guaranteed by the Contractor.

With an assumed generator efficiency of η_G = 98.0% and a transformer efficiency of η_{Tr} = 99.5%, all three turbine-generating units will be able to deliver 94.83 MW at the high voltage side of the main transformers.

9.2.6 Rated Speed of Turbine-Generating Unit

Whilst selecting the rated speed of the turbine-generating units, the following constraints were considered:

- Higher speed yields smaller dimensions of the turbine-generating unit and leads to lower investment cost of the equipment and a smaller powerhouse cavern;
- Higher speed requires lower turbine setting due to ensure against cavitation. With respect to the powerhouse cavern, the elevation of the cavern (some meters higher or lower) has negligible influence on the total project investment cost;
- The turbine setting must be sufficiently slow to preventi water column separation in the draft tube;
- The turbine speed selected will be within the range of existing references, to ensure the feasibility of the equipment.

As shown in Figure 9-1, 600 min^{-1} is the most appropriate speed for Tamakoshi V HEP, which fulfils the above-mentioned criteria. The turbine size is feasible and has the smallest dimensions (Table 9-1). Thus, 600 min^{-1} was selected as the rated speed.

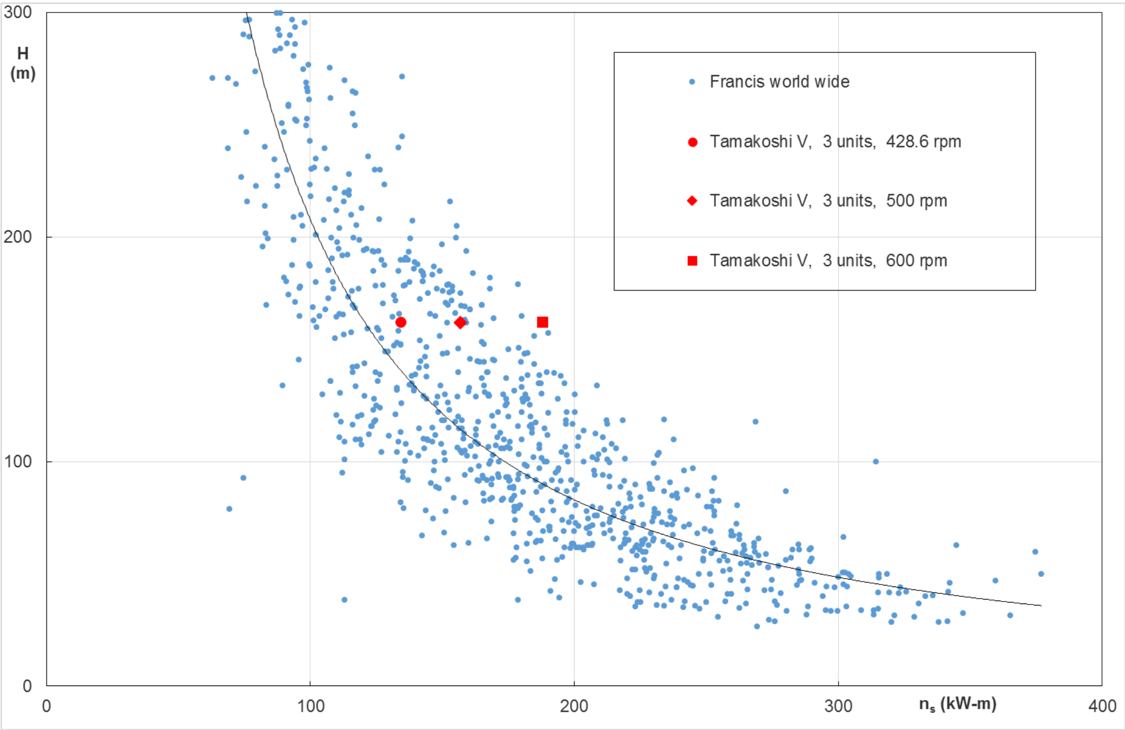

Figure 9-1: Comparison of turbine alternatives for Francis turbines worldwide in operation

Table 9-1: Comparison of turbine main parameters for different rated speeds

Item		Unit	Value		
Rated output (P_T)		MW		32.42	
Rated head (H)		m		162.35	
Rated discharge		m³/s		22.0	
Rated speed (n)		min^{-1}	428.6	500	600
Specific speed (n_s=n $P_T^{0.5}$ / $H^{1.25}$)		m-kW	133	155	186
Runner-Ø	Outlet	m	1.63	1.53	1.42
	Inlet	m	1.81	1.54	1.29
Spiral case:	inlet diameter	m	1.73	1.63	1.54
	Eccentricity	m	2.47	2.22	1.98
Draft tube:	depth	m	5.55	4.87	4.23
	length	m	9.11	8.07	7.14
Required suction head		m	-3.5	-6.5	-10.0
Turbine setting		m asl	980.5	977.5	974.0

9.2.7 Pressure Rise and Speed Rise

In the detail design the maximum pressure occurred in the spiral case and the minimum pressure occurred in the draft tube due to load rejection of all turbine-generating units and/or other circumstances. The final pressure will be finally calculated by the Contractor after optimisation of the turbine hydraulic design (e.g. by turbine model tests).

A preliminary transient calculation shows that a maximum of 2.1 MPa is expected in the spiral case, and in the draft tube, the pressure will not be lower than -48 kPa.

During the detail design, the maximum pressure occurred in the spiral case, the maximum and minimum pressure in the draft tubeThe runaway speed and the temporary speed rise of the turbine-generating unit will be guaranteed by the tenderers. Preliminary transient calculation will be included in each offer.

9.2.8 Turbine Main Parameters

The parameters of the turbine will be finally selected by the turbine supplier. Each turbine has the following main preliminary parameters:

- Type — Francis vertical
- Bearing arrangement — three guide bearings (2 for generator, 1 for turbine)
one thrust bearing (at generator)
- Frequency — 50 ± 2.5 Hz
- Rated head — 162.35 m
- Rated discharge — 22.0 m^3/s
- Rated output — 32.42 MW
- Rated speed — 600 min^{-1}
- Runaway speed — 1100 min^{-1}
- Turbine setting elevation (centre line spiral case) — 974.0 m asl
- Runner-Ø — Outlet 1.42 m
Inlet 1.29 m
- Guide vane — pitch circle Ø 1.75 m
height 0.30 m
- Spiral case: — inlet diameter 1.54 m
Eccentricity 1.98 m
- Draft tube: — depth 5.10 m
length 7.14 m
- Turbine Weight (estimated) — 60 t

In the final design, those dimensions may vary slightly depending on the selected supplier. Experience has confirmed that small deviations will not affect the basic layout and sizing of the powerhouse structures.

The rotating direction will be clockwise viewed from the top of generator.

9.2.9 Turbine Model Tests

Model tests are of benefit to identify early the characteristics and problems of the prototype and to do the necessary modifications. The verification of the guarantees is more accurate at the test rig than at Site. Conversely, model tests increase the project's cost. Thus, it is recommended to specify the turbine model tests as an option due to potential cost saving.

If the contractual turbine supplier has not performed model tests for Francis-turbines with specific speed equal to that of Tamakoshi V HEP, he will carry out model tests as specified below.

From the spiral case inlet to the end of draft tube, the model turbine will be fully homologous to the prototype. The model tests will determine all parameters, which are required for perfect hydraulic layout and design of the prototype, for obtaining the setting level and the main dimensions of the turbine as well as for safe and smooth operation.

The model tests and the model acceptance tests will be performed according to IEC 60193. The preliminary model test report will be submitted by the Contractor, latest, 28 days prior to the start of the model acceptance tests.

The turbine model tests will at least include the following items:

- Performance tests to determine the performance characteristics for various guide vane openings and at net effective heads corresponding to the various flow rates;
- Complete cavitation tests to show the cavitation and flow behaviour over the entire operation range;
- Runaway tests to determine the runaway speed under the operating heads;
- Hydraulic axial thrust over the entire operation range;
- Torque on the guide vane over the entire operation range;
- Pressure pulsations in the spiral case, between guide vane and runner, in draft tube (inlet, elbow and outlet) including frequency analysis;
- Torque oscillations and frequency analysis.

9.2.10 Main Components

Each turbine will consist of the following main components:

- Spiral case and stay ring;
- Guide vanes and regulating mechanism;
- Head cover and bottom ring;
- Turbine runner;
- Turbine shaft and intermediate shaft;
- Shaft seals;
- Turbine guide bearing including oil lubrication system;
- Turbine pit liner.

The assembly (including the first assembly) and dismantling of the turbine components will be carried out by dismantled intermediate shaft through the hatch from the turbine floor and to the machine hall floor, regardless of the generator. The design of the turbine components will consider this circumstance. For easy handling during the assembly and dismantling, an overhead handling system with sufficient lifting capacity will be provided in each turbine pit, rails will be installed for moving the turbine components from the turbine pit to the hatch.

Each turbine generating unit will have three bearings: one combined thrust-guide bearing above the generator rotor, one guide bearing below the generator rotor and one guide bearing next to the turbine runner.

The turbine runner will rotate "clockwise", looking from above.

9.2.11 Provision for Air Admission

Each turbine will be designed and fabricated with all the necessary air admission provisions, including pipes, valves and remote-control devices, to inject compressed air to the suction side of runner and/or draft tube during partial load and other operating conditions. The location of the air admission inlets, pipe and valve sizes will be finalized by the turbine supplier based on his experience.

If the shaft bore would be used for aeration of the suction side of runner, particular attention will be paid to prevent water leakage through the shaft coupling.

The final selection of the aeration method will be proposed by the Contractor and approved by the Employer/Engineer. The start and the end of the aeration will be monitored.

9.2.12 Instrumentation and Safety Devices

The turbine and its auxiliary equipment will be furnished with all instruments, control, etc., necessary for manual and automatic start-up, shut-down and supervision of the operation. All conditions of importance for operation will be monitored. On occurrence of a failure, an alarm signal will be given, and if dangerous operating condition occurs, normal or emergency shut-down of the generating unit will be initiated.

The following measuring devices as well as monitoring and indicating instruments will be provided as minimum. The Contractor will add instrumentation and/or safety device, where it is required.

- Pressure in the spiral case
- Guide vane position, guide vane safety elements
- Pressure fluctuations between guide vanes and runner
- Pressure below the runner
- Pressure in the draft tube liner
- For the turbine guide bearing as specified in the Subsection 7.2.10.8
- For the shaft seal as specified in the Subsection 7.2.10.7

A mechanical-hydraulic over speed protection device will be delivered as specified under the Subchapter “Turbine Governor”.

Each turbine-generating unit will be equipped with a vibration monitoring system supplied within the Computerized Control System of the power station. Provisions for fixing of the sensors will be provided.

9.3 Turbine Governor

9.3.1 General

Each turbine will be equipped with a governor, which will be composed of a digital governor, governor actuator, oil pressure unit and associated control and safety devices.

The governor actuator and the oil pressure unit will be arranged on the Turbine Floor, whilst the cubicle containing the digital governor will be next to the unit control board on the Machine Hall Floor.

9.3.2 Design Criteria and Operation Conditions

The turbine governor will be of the PID type, designed in accordance with latest relating IEC-Standards, with electric feedback between the guide vane servomotors and the digital governor. The design of the turbine governor will be state of the art and fully compatible with the Computerized Control System of the power station.

Whilst configuration of the governor, particular attention will be paid to the fact, that the operation of Tamakoshi V HEP is solely determined by the discharge released from UTK, which is designed as a power plant for peak load with six Pelton units.

All equipment will be provided for fully automatic operation. An automatic start-up control system will enable start-up and synchronizing of the turbine-generating unit within the shortest time permissible.

The positioning times for the set points of speed, power and guide vane position of the governor will be adjustable during commissioning in accordance with governor settings and requirements due to hydraulic transients at power changes. In order to attain a satisfactory optimisation of the governor, the amplification and integral action time will be as smoothly adjustable as possible over a large range.

The governor will ensure stable operation under all operating conditions. Therefore, a distinction will be made between parallel operation with/without other machines in the grid and isolated operation.

The digital governor will allow the following control modes:

- Speed control (PID),
- Power-frequency control (PI), and
- Opening control (PID).

Switch-over from one control mode to another and vice versa will be bump-free.

To achieve rapid correction of frequency fluctuations in parallel operation, power-frequency control will be performed by linear, adjustable power-frequency droop function, specially optimised for the purpose and conditions of the Tamakoshi V HEP.

In the parallel operation (power-frequency control) the digital governor will check permanently if the adjustable speed band is violated to guarantee the detection of grid disturbances in a reliable way. The width of the speed band will be adjustable. If the speed band is violated, the digital governor will switch automatically to the speed control mode using the isolated mode parameters.

The signal for power control will be obtained from the generator instrumentation. The power will be controlled at an accuracy of not less than 1%.

The speed signals will be obtained from a voltage transformer located on the output terminals of the generator and from a toothed wheel on the turbine shaft operating with proximity transducers.

The speed controller will respond to deviations in frequency less than 0.01 Hz. In addition to the set-point adjuster, it will be possible for a superimposed device to transfer binary signals of speed, power, deflector and needle positions. The set points of speed, deflector and needle positions, and power will not be affected by alterations of the speed droop.

The following functions will be included in the governor:

- Speed control at no load operation
- Automatic start and stop sequences, including automatic synchronization
- Manual start and stop by sequences of linked control actions
- Power output control; operation at output limitation with power feed back
- Frequency regulation in the parallel operation and in the isolated mode
- Load sharing between the units in "joint control" mode
- Quick shutdown in case of mechanical failures
- Emergency shutdown on electrical failures

The governor will monitor all conditions of importance for operation. On occurrence of a failure, an alarm signal will be given, and if dangerous operating condition occurs, normal or emergency shut-down of the generating unit will be initiated.

The opening and closing time of the guide vane servomotors will be adjustable.

The pressured oil system will be designed for maximum PN 160.

9.3.3 Main Components

Each turbine governor will consist of following main components:

- Digital governor
- Speed measuring devices
- Power measuring device
- Mechanical over-speed protection device
- Electric feedback transmitters for guide vane servomotors

- Governor actuator
- Oil pressure unit

All components will have state of the art design and conform to the relevant international standards.

The governor construction will enable all maintenance and repair work, which will require standstill of the turbine, within the shortest possible time. All governor components to be dismantled will be equipped with eyebolts, lugs and/or other devices to facilitate handling, installation and removal.

9.3.4 Instrumentation and Safety Devices

The turbine governor will be furnished with all instruments, control, etc., necessary for manual and automatic start-up, shut-down and supervision of the operation. All conditions of importance for operation will be monitored. On occurrence of a failure, an alarm signal will be given, and if dangerous operating condition occurs, normal or emergency shut-down of the generating unit will be initiated.

The minimum scope of measurement, monitoring and indicating instruments are specified in the relating Subsections describing the turbine governor components. The Contractor will add instrumentation and/or safety device, where it is required.

Control devices of the hydraulic governor will be installed on the oil sump tank as far as possible, easily accessible for maintenance and setting.

The supply will include the oil for flushing and first filling.

9.4 Main Inlet Valve

9.4.1 General

Upstream next to each turbine, a main inlet valve of through-flow butterfly type will be provided. It will have state of the art design according to international standards. The valve will have a service seal and a maintenance seal.

9.4.2 Design Criteria and Operation Conditions

The main inlet valve will isolate the related turbine from the upstream manifolds for inspection and maintenance without any influence on the operation of the other units. In addition, it will shut off the inflow to the turbine and into the cavern, if the turbine guide vanes could not be closed due to malfunction and/or if there would be pipe burst downstream of the main inlet valve. Therefore, the main inlet valve and its operation mechanism will be designed for closing under the design flow specified below.

The main inlet valve will be opened during starting procedure of the turbine-generating unit. It will be closed, when the unit stops the operation. Normally, opening and closing will be under approximately balanced condition with the turbine guide vanes in the closed position. However, the main inlet valve will be capable of closing against any flow up to the design flow.

The main inlet valve will be closed by closing weight(s) and opened by oil servomotor(s). It will be operated via the control systems or manually at the oil pressure unit of the turbine governor. The pressured oil for the servomotor(s) will be supplied by the pressured oil system of the turbine governor.

The main inlet valve will be so designed and constructed that no vibrations occur under the various operating conditions and that the working stresses will not exceed the maximum allowable values.

The opening and closing time of the main inlet valve will be independently adjustable between 30 s and 120 s. The closing time will be so adjusted that the momentary pressure caused at the valve will not be higher than the design pressure.

The main inlet valve will be installed/dismantled by using the powerhouse crane.

9.4.3 Nominal Diameter and Design Pressure

Generally, the nominal diameter of the main inlet valve will not be less than the inlet diameter of the turbine spiral case to avoid discontinuous flow entering the turbine. Thus, it will be finally selected by the turbine supplier. A nominal diameter of 1.54 m is expected.

The design pressure of the main inlet valve is identical to that for the turbine spiral case and can finally be set by the turbine supplier after the turbine model tests and transient calculation. According to the preliminary calculation, a design pressure of 2.1 MPa will be sufficient.

9.4.4 Design Flow

In the event of a burst of pipe downstream of the main inlet valve during the turbine operation, the main inlet valve will be safely closed against a hypothetical flow for preventing cavern flooding. That hypothetical flow is usually set to be double of the rated turbine flow. For Tamakoshi V HEP, the design flow of the main inlet valve is 44 m^3/s under the rated head.

9.4.5 Main Components

Each main inlet valve will consist of following main components:

- Valve body and support foundation
- Valve rotor and trunnions
- Trunnion sealing and bearing
- Service seal and maintenance seal
- Valve operation mechanism
- Extension pipe (on upstream side)
- Dismantling pipe (on downstream side)
- Bypass
- Local control system

9.4.6 Instrumentation and Safety Devices

The main inlet valve will be furnished with all instruments, control, etc., necessary for manual, automatic and supervision of the operation. All conditions of importance for operation will be monitored.

The following measuring devices as well as monitoring and indicating instruments will be provided as minimum. The Contractor will add instrumentation and/or safety device, where it is required.

- Pressure upstream and downstream of the main inlet valve
- Valve rotor position
- Oil pressure for operating servomotor and bypass
- End positions of the bypass operation valve

Non-intentional closing of the main inlet valve will cause a quick stop of the related turbine generating unit.

9.5 Draft Tube Flap Gate

9.5.1 General

Downstream of each turbine, a draft tube flap gate will be provided as integral part of the turbine draft tube. It will have state of the art design according to international standards.

9.5.2 Design Criteria and Operation Conditions

The draft tube flap gate will isolate the related turbine from the downstream manifolds for inspection and maintenance without any influence on the operation of the other units. In addition, it will shut off the inflow into the cavern if a pipe burst upstream of the flap gate.

Generally, the flap gate will remain in the open position, even whilst the standstill of the related turbine. It will be only closed for inspection and maintenance work on the turbine parts within the waterway or in the event of excessive water inflow into the powerhouse cavern. Opening and closing will be under approximately balanced condition with the main inlet valve in closed position.

To prevent excessive pressure on the flap gate, hydraulic interlocking will be implemented so that the flap gate can only be closed when the main inlet valve is completely closed, and the main inlet valve can only be opened when the flap gate is completely opened.

The flap gate will be closed by its own weight and opened by an oil servomotor. The pressurized oil for the servomotor will be supplied by the oil pressure unit of the turbine governor.

If the turbine spiral case inadvertently becomes pressurized and the flap gate is in the fully closed position, the flap gate will be automatically lifted up to prevent damage.

The flap gate will be designed and constructed so that no vibration occurs under the any operating conditions of the turbine-generating units and that the working stresses will not exceed the maximum allowable values.

In the open position, the area above the flap gate will be de-pressured by dewatering. Therefore, the tailwater pressure will be additionally effective to support the flap gate for keeping in the open position.

The flap gate can only be manually opened, after pressure balancing during manual opening of the main bypass. It can be closed both automatically and manually via the control system or manually via the oil pressure unit of the turbine governor.

9.5.3 Design Parameters

The dimensions of the flap gate are dependent on the draft tube size and will be determined by the turbine supplier and approved by the Employer/Engineer. The expected width and height of the flap gate are 4.0 m and 2.2 m, respectively.

The preliminary design pressure of the flap gate will be 1.0 MPa, if the transient calculation does not require a higher value.

The design flow will be 2 m^3/s corresponding to the rupture of a DN 350 water pipe.

9.5.4 Main Components

Each draft tube flap gate will consist of following main components:

- Gate frame
- Gate leaf
- Gate seals
- Gate operation mechanism
- Bypasses
- Local control system

9.5.5 Instrumentation and Safety Devices

The draft tube flap gate will be furnished with all instruments, control, etc., necessary for manual, automatic and supervision of the operation. All conditions of importance for operation will be monitored.

The following measuring devices as well as monitoring and indicating instruments will be provided as minimum. The Contractor will add instrumentation and/or safety device, where it is required.

- Pressure upstream and downstream of the flap gate
- Gate leaf position
- End positions of the oil-hydraulic operated mechanical locking device
- Pressure in the servomotor guide shaft
- Oil pressure of the pressured oil supply
- Redundant supervision of “lifting rod breaking”

Non-intentional closing of the flap gate will cause a quick stop of the related turbine generating unit.

9.6 Cooling Water System

9.6.1 General

The cooling water system will provide cooling water to all equipment installed in both the powerhouse and the transformer cavern, which requires cooling water for the operation. In addition, the cooling water system will provide water for the fire-fighting system and the domestic water supply system.

The schematic arrangement of the equipment is shown in the conceptual P&I diagram (drawing-No. 4700-R 3100). The cooling water pumps, automatic filters and headers will be arranged on the Auxiliary Floor.

9.6.2 Design Criteria and Operation Conditions

The cooling water system will be of the one-loop type for common use of all turbine-generating units. The cooling water will be taken from the turbine draft tubes, filtered by automatic filters, circulated by pumps to the consumers (generator air coolers, bearing oil systems, transformers, etc.) and returned to the turbine draft tubes.

The cooling water system will have four (3 for service + 1 as standby) main cooling water pumps and one small cooling water pump. A main pump will be started/stopped once a turbine-generating unit has started/stopped the operation. In the event thatall main units are in standstill, the small pump will maintain the cooling water supply to the transformers, the air condition system and if required, the small hydro unit, with the main pumps as standby.

The cooling water header to each turbine-generating unit and the cooling water return pipe to each turbine draft tube will be equipped with a motor-driven valve which will be opened/closed in connection with the operation of the related turbine-generating unit.

The cooling water system will be designed for PN 10. The nominal diameter of the cooling water pipes will be dimensioned so that the flow velocity is less than 2.5 m/s.

If the tailrace tunnel is completely dewatered, the cooling water will be taken from the extension pipe(s) of the main inlet valve. A pressure reducing device will reduce the headwater pressure to an adequate value. The cooling water will flow by gravity via the transformers and air condition system to the dewatering sump of the powerhouse cavern.

The raw water temperature varies between 2°C and 16°C. The temperature increase of the cooling water will not exceed 15°C for the main transformers and 5°C for all other consumers. The head losses of individual heat exchangers will not be higher than 50 kPa. Each heat exchanger will be equipped with a control valve for regulating the cooling water flow at the required rate.

The cooling water amount required by individual consumers will be finally determined by the equipment suppliers. Preliminary estimation yields the following values for one turbine-generating unit:

- Generator including bearings 45 l/s

- Main Transformer 3 l/s
- Turbine 5 l/s

Thus, the cooling water demand for a turbine-generating unit will be 53 l/s. In addition, 16 l/s will be necessary for the air conditioning system. Therefore, about 175 l/s cooling water will be totally required. This figure has been adopted for the current design.

The cooling water for the turbine shaft seals was considered in the above-mentioned amount. Alternatively, the shaft seal can be cooled by water directly taken from the extension pipe of the main inlet valve.

9.6.3 Main Components

The cooling water system will consist of following main components:

- Backwash filter
- Hydro-cyclone filter
- Cooling water pump
- Motor control centre
- Local control system
- Pipes and valves

Standard equipment will be applied for optimisation of investment cost and convenient maintenance work. All components will have state of the art design and conform to the relevant international standards.

9.6.4 Instrumentation and Safety Devices

The cooling water system will be furnished with all instruments, control, etc., necessary for manual, automatic and supervision of the operation. All conditions of importance for operation will be monitored.

At a minimum the following measuring devices as well as monitoring and indicating instruments will be provided. The Contractor will add instrumentation and/or safety device, where it is required.

- Pressure measurement at each cooling water pump outlet
- Pressure and temperature sensors of the cooling water in the headers
- Temperature gauge and flow meter for the cooling water at inlet and outlet of each consumer
- Operation monitor for cooling water pumps
- Valve end position monitor for the motor-driven valves

9.7 Drainage and Dewatering System

9.7.1 General

Using gutters and conduits of the powerhouse structure, leakage water of various equipment and liquids from the cavern floors will be guided through the coalescence type oil separator to the drainage sump located between the Units No 2 and 3, while the seepage water of the caverns, including the associated underground facilities, will be partly guided to the dewatering sump located between the Units No 1 and 2 and partly to the drainage sump.

If a turbine, the steel lined headwater way and/or the tailrace tunnel requires dewatering, the related water will be fed to the dewatering sump.

The drainage and dewatering system will bring the water from the both sumps to the outside of the cavern. The schematic arrangement of the equipment is shown in the conceptual P&I diagram (drawing-No. 4700-R 3102).

9.7.2 Design Criteria and Operation Conditions

The water in the drainage sump will be pumped to the dewatering sump via a drainage pipe. Two drainage pumps will be installed in the drainage sump, one for service and one as stand-by. Preliminarily, the capacity of each pump was set to be 15 l/s. The drainage system will be designed for PN 6 as minimum. The actual seepage and leakage amount will be verified after completion of the excavation of the caverns and then the pump capacity will be dimensioned accordingly.

The system will operate automatic. The drainage pump will be controlled by water levels in the drainage sump. The nominal diameter of the drainage water pipe will be dimensioned so that the flow velocity will usually be less than 2.5 m/s.

The water in the dewatering sump will be pumped outside via the dewatering pipe arranged in the Cable & Ventilation Tunnel.. Three dewatering pumps will be installed in the dewatering sump, two for service and one as stand-by. Given flood management considerations of the powerhouse cavern (pipe burst) and the dewatering of a turbine and the tailrace tunnel,the capacity of each pump will be set to 175 l/s. The dewatering system will be designed for PN 10.

The arrangement of the dewatering pipe in the powerhouse cavern is shown in the Album of Drawings, Part E of the Detailed Design Report, drawing 4700-R 3102). A transient calculation will be carried out by the Contractor for verification of the necessity of an expansion tank. In the event of inspection and maintenance of the dewatering pipe, the water in the dewatering sump will be conveyed to the draft tube of the turbine No.1 or No.2.

Both the drainage pumps and the dewatering pumps will be controlled by water levels in the related sump, either by float switches or by level measurement.

9.7.3 Main Components

Both the drainage and the dewatering system will consist of following main components:

- Pump-motor-units
- Motor control centre
- Local control system
- Pipes and valves

Standard equipment will be used to minimize cost and facilitate maintenance work. All components will have state of art design and conform to the relevant international standards.

9.8 Compressed Air System

9.8.1 General

A centralised low compressed air system will provide compressed air for

- the mechanical brake of the generators,
- maintenance seal of the turbine shaft,
- expansion tank of the dewatering pipe, if any, and
- service air for pneumatic tools at required points in the caverns.

The schematic arrangement of the equipment is shown in the conceptual P&I diagram in Part E Album of Drawings of the Detailed Design Report, (drawing-No. 4700-R 3101).

9.8.2 Design Criteria and Operation Conditions

The compressed air system will be designed for PN 10. The operation pressure will be about 0.8 MPa.

Two identical air-cooled compressors and two air receivers will be provided. The compressors will be automatically operated according to the air pressure in the receivers.

A single compressor will be able to fill the main air receiver from atmospheric pressure up to the operation pressure within two hours and to recharge it from the minimum to the operating pressure in less than five minutes.

The main air receiver, of minimum 2 m^3 capacity, will provide compressed air to the generator brake, maintenance seal of the turbine shaft and the expansion tank of the dewatering pipe, if any. The auxiliary air receiver, of minimum 1 m^3 capacity, will provide compressed air for miscellaneous consumers in the cavern.

9.8.3 Main Components

The compressed air system will consist of the following main components:

- Compressor
- Air receiver
- Motor control centre
- Local control system
- Pipes and valves

Standard equipment will be used to minimize cost and be for convenient maintenance work. All components will have state of the art design and conform to the relevant international standards.

9.9 Oil Handling Equipment

9.9.1 General

With the goal to reduce investment and maintenance costs, as well as the risks caused by oil storage, there will be no stationary oil handling system in the caverns. For treatment of the hydraulic oil (bearings, governors) of the turbine-generating units, two mobile oil filtration stations will be provided.

9.9.2 Design Criteria and Operation Conditions

Two mobile oil treatment units equipped with oil feeder pumps, strainers, filter(s), heater(s), water separator, vacuum pump(s), valves, pipes, measuring, control and supervisory equipment for automatic operation will be provided. The mobile units will be equipped with wheels for easy movement.

Each mobile unit will, at a minimum, be able to purify 1 m^3/h oil. The design pressure of the unit will be a minimum 0.3 MPa. The power cable will have about 50 m length and a plug compatible with the standard socket outlets and the prevalent distribution voltage of this project. The oil pipes connecting the oil tanks/reservoirs will be equipped with quick release coupling compatible with those installed on the oil tanks/reservoirs.

After the treatment, the oil for bearings and hydraulic governors will fulfil the requirements specified by the code 15/13/10, ISO 4406, and the water content will not exceed 40% of the saturation point.

Standard products of an international well-known manufacturer is preferred. All elements will be easily accessible and replaceable.

A mobile oil quality control unit will be provided.

9.9.3 Main Components

Each mobile oil treatment unit will consist of following main components:

- Oil pumps
- Heater
- Filter
- Water separator
- Control box

9.10 Powerhouse Crane

9.10.1 General

To facilitate assembly, erection and dismantling work of the equipment installed in the powerhouse cavern, a powerhouse crane will be provided. All mechanical and electrical equipment required will be provided, including cables, cable conduits and terminals.

9.10.2 Design Criteria and Operation Conditions

The powerhouse crane will be of overhead bridge, double girder type and designed according to the international standards likes EN, FEM, etc.

The crane will have a main and an auxiliary hook. The main hook will be used for lifting the heaviest component to be lifted, mostly the generator rotor, whilst the auxiliary hook will be utilized for erection and maintenance work of smaller parts like rotor poles, servomotors, heat exchangers, etc. Thus, the rated capacity of both hooks will be finally optimized by the Contractor and approved by the Employer/Engineer.

Preliminarilly, the powerhouse crane has the following main parameters:

•	Rated lifting capacity	main hook	80	t
		auxiliary hook	10	t
•	Span		10.4	m
•	Lifting height	main hook	14	m
		auxiliary hook	20	m
•	Rail base elevation		989.19	m asl.
•	Rail track length		69	m
•	Bridge travelling speed	normal	20	m/min
		creeping	0.6	m/min
•	Trolley travelling speed	normal	15	m/min
		creeping	0.6	m/min
•	Main hook lifting speed	normal	1.2	m/min
		creeping	0.1	m/min
•	Auxiliary hook lifting speed	normal	4	m/min
		creeping	0.5	m/min

The crane will be remotely controlled by using radio-operated portable control panel. Operator’s cabin is not necessary and will not be provided in order to save project costs.

The crane will be tested at 125% of the rated lifting capacity at Site. The testing load will be provided and removed after the testing by the Contractor. The maximum deflection of the bridge structure under the rated lifting capacity will not exceed 1/1000 of the span.

9.10.3 Main Components

The overhead bridge crane will have following main components:

- Runway rails
- Crane bridge
- Trolley
- Lifting equipment
- Brakes

- Power supply
- Control equipment
- Maintenance lifting platform (optional)

Along the crane runways as well as on the girders and trolley, handrails with provisions for safety will be supplied. Vertical height of the handrails above the footing plate will be not less than 1000 mm. There will be middle rails, approximately 450 mm above the footing, and breast boards of at least 70 mm height at the bottom of the handrails.

9.11 Cargo and Passenger Elevator

9.11.1 General

One combined cargo and passenger elevator for easy and quick moving and handling will be provided in the powerhouse cavern. The elevator will be state-of-the-art and designed according to international standards, e.g. EN 81, ASME A17.1.

9.11.2 Design Criteria and Operation Conditions

The elevator will be able to transport a pump of the cooling water system and drainage and dewatering system. Therefore, the lifting capacity of the elevator will be finally selected by the Contractor.

Preliminarily, the elevator has the following main parameters:

•	Rated capacity		1500	kg
	Equivalent passenger number		17	
•	Rated speed		1 - 1.5	m/s
•	Lifting height		approx. 17.2	m
•	Cabin inside dimensions	clear width	1.6	m
		clear length	2.1	m
		clear height	2.3	m
•	Cabin door (on both sides)	clear width	1.1	m
		clear height	2.3	m
•	Floors to be served	HVAC Floor	EL. 989.24	m asl.
		Administration Floor	EL. 985.64	m asl.
		Machine Hall Floor	EL. 982.19	m asl.
		Generator Floor	EL. 978.51	m asl.
		Turbine Floor	EL. 975.15	m asl.
		Auxiliary Floor	EL. 971.98	m asl.

The design of the elevator will meet the following requirements:

- Multiple Independent braking systems;
- Direction controls in cabin at start and end platforms;
- Security push button in cabin/platform at start and end platforms;
- Drive variable speed soft start/stop motors control package;

- Driven by electric hoisting machine including cable and counter weight, or similar;
- Cabin and shaft doors of automatic sliding type, access from opposite side possible;
- Smooth, however, rapid acceleration of cabin;
- Accurate floor landing/levelling without jerking and jumping;
- Minimum 240 starts per hour and operation without overheating of hoisting machine.

9.11.3 Main Components

The elevator will have following main components:

- Hoisting machine
- Suspension and guides
- Cabin
- Shaft doors
- Control equipment

9.12 Mechanical Workshop Equipment

The mechanical workshop will be equipped with machines, tools and associated facilities for the normal maintenance and repair of the electromechanical equipment.

The following tasks will be executed with the mechanical workshop equipment:

- Repair and maintenance of the power station vehicles
- Day-to-day maintenance like testing, lubrication, as well as minor repair work of the equipment in the power station
- Dismantling and erection of minor parts in the power station
- Perform site repair and maintenance on minor parts in the power station

The items and numbers of the equipment, which will be supplied, will be specified in the tender documents.

9.13 Small Hydro Unit

9.13.1 General

With the objective of utilizing the minimum water flow discharged by UTK, a small hydro turbine-generating unit will be installed in the powerhouse cavern. The arrangement of the small unit is shown in the Album of Drawings, Part E of this Detailed Engineering Design Report, drawing 4330 - Q 1300.

9.13.2 Design Criteria and Operation Conditions

The small unit will operate, if only one unit in UTK is operating in the operation mode “Spinning Reserve” with a discharge not less than 1.5 m³/s. In addition, the small unit will operate, if in Tamakoshi V HEP at least a main unit is out of service.

To limit the investment cost, the small unit will share the main transformer with the main unit 3. Thus, the output of the small unit is limited to 5 MW at the generator terminal, if it operates together with Unit 3 and another main unit operating at the rated condition. That means:

$$P = \rho \, g \, H \, Q \, \eta_T \, \eta_G = 5 \text{ MW}$$

where:
- ρ = density of water = 1000 kg/m³;
- g = acceration due to gravity = 9.8 m/s²;
- H = rated net head;
- Q = rated turbine discharge;
- η_T = turbine efficiency at rated condition = 92% (assumed);
- η_G = generator efficiency at rated condition = 97% (assumed).

H and Q of the small unit have to be iteratively determined. The net head is obtained by gross head minus total head losses, which also depends on the total discharge through the waterway (small unit + two main units). H = 168.6 m and Q = 3.39 m³/s are the result of the iterative calculation.

The small unit will be of horizontal Francis type. The rated speed of the unit will be 1000 min^{-1}. The unit will be arranged at the Turbine Floor. The maximum diameter of the runner and the inlet diameter of the spiral case is expected to be 0.8 m and 0.7 m, respectively.

The design pressure of the turbine spiral case and the turbine inlet valve is identical to those for the main turbine, which will finally be fixed later. According to the preliminary calculation, a design pressure of 2.1 MPa is expected.

The generator will have the following rated parameters:

• Rated power	6	MVA
• Rated voltage	11	kV
• Rated speed	1000	min^{-1}
• Rated frequency	50	Hz
• Power factor	0.85	

The rotating direction will be clockwise viewed from the turbine to the generator. Erection and dismantling of the small unit will be carried out by using the powerhouse crane.

9.13.3 Main Components

The small unit will consist of following main components:

- One horizontal turbine-generating unit
- One turbine governor

- One turbine inlet valve
- One turbine outlet valve
- Electrical equipment
- Unit control system

A standardised unit of a turbine supplier is acceptable.

9.13.4 Instrumentation and Safety Devices

The small unit and its auxiliary equipment will be furnished with all instruments, control, etc., necessary for manual and automatic start-up, shut-down and supervision during operation. All conditions of importance for operation will be monitored. An occurrence of a failure will trigger an alarm signal and if a dangerous operating condition occurs, normal or emergency shut-down of the generating unit will be initiated.

9.14 Water Level Measuring Devices

9.14.1 General

Measuring devices will be provided for measuring the water level in the:

- head pond,
- surge tank, and
- tailrace outlet structure.

The locations of the head pond, surge tank and tailrace outlet structure are shown in the Album of Drawings, Part E of the Detailed Engineering Design Report, drawing 4310-Q1201.

9.14.2 Design Criteria and Operation Conditions

Equipment for measuring the water levels with two different measuring methods will be provided, including all necessary auxiliaries. The measuring sensors will be protected by stainless-steel pipes to obtain stable and proper measurement. The data measured will be used for the operation of the turbine-generating units and the gates.

The measuring ranges will be:

- Head pond EL.1140 – EL 1160
- Surge tank EL.1130 – EL.1180
- Outlet structure EL. 974 – EL.1004

The measured data (4 – 20 mA) will be cabled to the related terminal boxes and then transferred to the Computerized Control System (CCS) of the power station. All cables including protecting pipes, terminal boxes and necessary auxiliaries will be provided.

For each of the above three locations, a water level staff gauge will be provided and installed. It will extend from the bottom to an elevation above the maximum water level and consist of standard gauge scales of durable plastic with subdivisions for every cm and intermediate divisions every 10 cm. The standard gauge scales will be firstly bolted to galvanized steel plates and then to the concrete. For every two meters, elevation numbers of about 200 mm high will be indicated.

9.14.3 Main Components

The devices and instruments will be of proven and sturdy design for the rough conditions prevailing at or near the water passages. The precision of the instruments will satisfy the relevant ISO standards.

The components of the pressure measuring points, the measuring pipes and the bleeder valves will be of stainless steel. The measuring points will include replaceable orifice inserts in accordance with IEC 60041. The supply will include temporary cover plates attached to the flush mounted measuring plates during installation and concrete works. The pipes will rise steadily to the bleeder valves in front of the instruments.

The submersible pressure type sensors will contain a relative pressure cell, which compensates the atmospheric pressure changes. The compensation will be accomplished by the vented sensor-bus cable. The measuring range and the accuracy will be duly selected.

The level sensors will be adequately protected against waves and other flow effects. The sensor case will be of stainless steel. The sensors will be connected through a sensor bus to the processing units. Each processing unit will include two programmable analogue outputs, two programmable limiting value outputs and one acceptance output. The processing units and sensor-buses will be provided with over-voltage protection.

9.15 HVAC Systems, Ventilation and Air Conditioning

9.15.1 General

The Ventilation and Air-Conditioning system shall be installed in all rooms and areas of the project according to the internal requirements. Some areas need to be air conditioned and some shall be ventilated only. The different requirement are summarized in the following description.

9.15.2 Scope of Work

The Scope of Works for the Ventilation and Air Conditioning System shall cover the complete engineering, preparation of construction drawings, shop drawings, approval drawings, procurement, manufacture, testing, transport, erection, installation, testing, commissioning and completion of the heating, ventilation and air-conditioning systems (HVAC) including motor control boards, switchgears, automatic control and associated cabling according to the requirements of the Bid Documents.

9.15.3 Design & Design Data

9.15.3.1 Outdoor Design Conditions

Design Condition in summer:

Air temperature	32°C DB
Relative humidity	35 %
Daily range	20 K (°C)

Design Condition in winter:

Air temperature	+2 °C DB

9.15.3.2 Indoor Design Conditions

The maximum indoor design conditions shall be provided in different rooms as identified below:

Room / Area	Maximum Condition
Powerhouse:	
Machine Hall	30°C DB
Central Control Room	23°C / 40% r.h. * (summer), 20°C (winter)
Office rooms, conference rooms:	25°C /20°C DB (summer/winter)
Corridors, locker rooms, showers:	30°C DB
Toilet rooms	temperature not controlled
Switchgear Rooms	30°C DB
Battery Room	25°C DB
Generator Floor	30°C DB
Cable Gallery	35°/40°C DB
Turbine Floor	30°C DB
Erection Bay, Workshops	30°C DB
Inspection/ Dewatering Gallery	temperature not controlled
Cable Gallery	up to 50°C DB

<u>Operation Buildings:</u>

Office rooms, exhibitions room, telecommunication room, reception:	25°C /20°C DB (summer/winter)
Corridors, tea rooms:	30°C DB
Toilet rooms	temp. not controlled in summer (in winter min. 15 °C)

<u>* Remarks</u>:

- The HVAC systems shall be stand-alone controlled and supervised through central control facilities
- For rooms such as computer room, electronic equipment room, control room etc., the design room conditions specified by the equipment manufacturers shall be considered by the air-conditioning system design if the requirements are more restrictive as specified above.
- The above table for Air Conditioning and Ventilation and the description for the Air Conditioning and Ventilation for each room in the Technical Specification Civil Works are complementing each other and in case of any difference in the interpretation the Contractor shall ask the Consultant for clarification and the Consultant's decision shall be final and binding.

9.15.4 Fresh Air Rates

The minimum fresh air rates for air conditioned rooms and areas shall be provided upon the following, whichever is greater:

Office rooms, conference room, launch room	40 m³/hr per person or
	Equal to transfer air/exhaust air volume
	All above + overpressure requirements
Control Rooms	40 m³/hr per person or
	equal to transfer air/exhaust air volume or
	10% of supply air volume
	All above + overpressure requirements
Workshops	40 m³/hr per person or
	Equal to exhaust air volume
	10% of supply air volume
	All above + overpressure requirements

9.15.5 Air Changes per Hour (ACR)

Exhaust systems shall be designed upon heat removal requirement or based on the following minimum air changes per hour, whichever is greater:

Toilets	6	ACR
Washrooms	6	ACR
Tea-kitchens	4	ACR
pump rooms	4	ACR
storage rooms	4	ACR
air-compressor rooms	4	ACR
technical rooms	4	ACR
water-treatment plants	4	ACR

9.15.6 System Requirements

- Where necessary, fire dampers and sound attenuators shall be provided to comply with the fire compartmentation and sound level requirements.

- The outdoor air intake arrangements shall include a rain preventing louver.

- All systems, designed to remove internal heat from machinery, lighting, etc., shall be based on the design temperatures and internal heat load. In absence of excessive internal heat, the systems shall be designed on an appropriate air-exchange rate as specified above.

- The ventilation system in rooms containing a flammable or explosive material shall be automatically stopped under emergency conditions.

- Ventilation systems in rooms with automatic fire-extinguishing systems shall be stopped upon fire alarm system via fire alarm panel and local ventilation control panel.

- The Ventilation and air-conditioning systems shall be designed to the rules of "Fire Services Requirements for Buildings". The ducting system shall be equipped with fire dampers and thermal or electro-thermal links. All the thermal insulation shall be incombustible and a fire-retardant product.

- For rooms with automatic gas-extinguishing system, the fire dampers and HVAC units of the affected room shall be controlled in the local control panel and the remote fire-alarm panel. When the fire alarm is initiated, all the fire dampers shall be closed and the HVAC unit (S) shall be stopped instantly.

- Automatic control system and control panel shall be provided for the ventilation and air-conditioning system. Audio-warning alarms and visible indication system of the air-conditioning systems shall be installed in the Central Control Room and the local control panel. Local control panel and power supply isolator for the air-conditioning system shall be provided.

- Location of welding areas in workshops or areas where special exhaust is needed shall be selected as such that provision of individual exhaust air/make-up air can be provided in order to limit the size of the A/C units.

9.15.7 Air Distribution System

The air-distribution systems shall be generally low-velocity systems with constant outside air rates.

The maximum air velocities in different applications shall be as follows:

Room, Equipment	max. Velocity m/s
Air-conditioned rooms:	
between 0.50 and 2.00 m height above floor level	0.25
In main air ducts	5 – 7
In main ventilation ducts	5 – 8
In branch ducts	2 – 5

For final design of the above-mentioned air flow velocities, the induction effects of the air distribution and the maximum allowable sound pressure level shall be considered too.

9.15.8 Chilled Water System

The chilled water system shall consist of the central chiller units and the chilled water distribution network. The cooling water temperature even of the closed circuit will be nearly all time cold enough to cool down the room temperatures. Therefore the cooling water can pass the chillers in a bypass for ecological and economic reasons. Only during extreme outside conditions, when the cooling water temperature (Closed circuit) will be higher than 20 °C. For these seldom conditions the chiller units shall be installed. By dividing the required capacity into two chiller units of the same dimension, there is enough redundancy for reliable operation of the main plants.

9.16 Sanitary Installations

For the power station personnel (and visitors) there shall be toilets and showers installed in the operation area. These installations will be connected by waste water pipes to a waste water pumping station on the valve floor. The waste water will be pumped up to a collection tank (of approximately 10 m³) above the stairwell at unit 1. This tank will have a pipe connection to the tunnel portal, so that the tank can be emptied with a special waste water tank truck. The room for this collection tank will be kept at low pressure by a permanent connection to the exhaust air system in order to prevent any smell permeating to the machine house floor.

There shall be one truck provided for the power house and the surface facilities of the project. Depending on the number of staff members or visitors the tank in the power house has to be emptied once a week or twice a week as per requirement.

The truck shall be emptied into the next waste water treatment plant located outside of the power house at a distance of at least 200 m from any access tunnel or occupied houses, respectively it shall be executed as a gastight installations.

9.17 Central / Emergency Service Stations

For health and safety as well as for the operational reliability it is a basic requirement to have these services available within a short distance from the project area. These essential services are:

a) Fire brigade, fire workers, well educated, experienced and trained in the power house during construction on site and as well as during normal operation.

b) Rescue teams well informed and familiar with the facilities in the power house and different technical requirements.

c) Health service including a well-equipped medical treatment facility and a doctor permanently available.

d) Central sewage treatment plant or the transport service to the next facility in the vicinity.

9.18 Fire Protection Systems

This fire protection concept shall serve as a guideline for the design and the implementation of all fire protection installations for the hydropower Project as required by the Contract and the pertaining Technical Specifications, relevant local and international Codes and Standards as well as the actual state-of-the-art for fire protection for hydroelectric generating plants.

The Civil Works and Hydro-mechanical Equipment Contractor’s scope of work for the fire protection installations shall include all required fire rated equipment for doors, walls, ceilings, gates and hatch covers. The Mechanical and Electrical Equipment this Contractor’s scope of work shall include all other installations for fire and smoke detection and firefighting systems.

All fire protection installations shall be designed, installed and commissioned in accordance with the international NFPA-Codes and Standards, mainly NFPA 850, Recommended Practice for Fire Protection for Hydroelectric Generating Plants, latest Edition.

The design of buildings, civil structures and all fire protection installations shall also be in accordance with local regulations and requirements from the authority having jurisdiction.

The fire protection installations shall be ready for operation before commissioning the first turbine-generating unit.

All the fire protection installations shall be designed, supplied, installed and commissioned in such a way to provide comprehensive fire protection for all buildings and areas of the Project.

These areas and installations are:

- Powerhouse Cavern
- Transformer Cavern
- Auxiliary Galleries
- Cable Galleries/Tunnels

The fire protection installations for these areas shall mainly comprise:

- Structural fire protection
- Fire detection and fire alarm systems
- Fire fighting pumps
- Fire service main and outdoor hydrants
- Standpipes with indoor hose stations
- Water spray fixed systems
- Mobile fire extinguishers
- Installation material like supports, hangers, sleeves, seals, bulk heads, fire stops, electrical cables etc. for the complete fire protection installations
- Spare parts and special tools.

The fire fighting and fire detection systems to be provided are indicated in the following table as a minimum. Number and location of the various installations and systems shall be determined by the Contractor during detail design, subject to approval by the Employer/Engineer. The Contractor shall, however, add and include any areas / items not listed in the table to cover fire protection for the entire Plant.

OVERVIEW OF FIRE PROTECTION SYSTEMS TO BE INSTALLED

BUILDING / AREA / EQUIPMENT TO BE PROTECTED	**FIRE FIGHTING SYSTEMS**	**FIRE DETECTION SYSTEMS**
Powerhouse Cavern, Machine Hall	Hose stations, Dry powder and CO_2 fire extinguishers	Smoke beam detectors, Manual fire alarm stations
Powerhouse Cavern, Generator Floors	Hose stations, Dry powder and CO_2 fire extinguishers	Smoke beam detectors Smoke detectors, Manual fire alarm stations
Powerhouse Cavern, Turbine Floors	Hose stations, Dry powder fire extinguishers	Smoke detectors, Manual fire alarm stations
Offices	Hose stations on each level next to each staircase, Dry powder fire extinguishers	Smoke detectors, Manual fire alarm stations
GIS Floor	Hose stations on the staircase	Smoke detectors, Manual fire alarm stations
Workshop Building	Outdoor hydrants, Hose stations, Dry powder fire extinguishers	Smoke detectors, Manual fire alarm stations
Generators	Automatic and electrically interlocked water spray fixed systems	Smoke and heat detectors, Generator differential relay (Generator disconnected from system)
Transformer Bays, Transformer Cavern	Hose stations, Dry powder and CO2 fire extinguishers	Smoke beam detectors, Manual fire alarm stations
Transformers and Reactors	Automatic water spray fixed systems (Transformer and reactor disconnected from system)	Sprinkler head heat detectors, Buchholz relay
Control, Computer, Relay and Telecommunication Rooms	CO2 fire extinguishers	Smoke detectors

BUILDING / AREA / EQUIPMENT TO BE PROTECTED	FIRE FIGHTING SYSTEMS	FIRE DETECTION SYSTEMS
LV and HV Switchgear Rooms	CO2 fire extinguishers	Smoke detectors
Cable Galleries, Cable Shafts and Cable Spreading Rooms	Automatic water spray fixed systems	Smoke detectors
HVAC Plant Rooms	CO2 fire extinguishers	Smoke detectors for the plant rooms and duct smoke detectors for the ventilation ducts
Oil Storage and Oil Treatment Plant Rooms	Dry powder fire extinguishers	Smoke detectors
Compressor, Chiller and Pump Rooms	Dry powder fire extinguishers	Smoke detectors
Mechanical Workshop Rooms	Dry powder fire extinguishers	Heat detectors
Electrical Workshop Rooms	CO2 fire extinguishers	Heat detectors
Storage Rooms	Dry powder fire extinguishers	Smoke detectors
Corridors and Lobbies	Dry powder fire extinguishers	Smoke detectors, Manual fire alarm stations

10 POWER STATION ELECTRICAL EQUIPMENT

10.1 General

10.1.1 Scope of Work

The purpose of the Detailed Engineering Design Studies was to identify and to dimension the principal electrical components required for safe and economic operation of the Power Station.

The dimensions and layouts of the equipment have been developed to a detail sufficient to form the basis for the elaboration of the Tender Drawings and the Technical Specifications.

In the present report, the concepts for the electrical equipment are described.

The main electrical equipment of the power station will comprise the following:

- Three 38 MVA, 11 kV synchronous generators with auxiliary, monitoring and protection systems
- One 6 MVA, 11 kV Small HP Unit (reference is made to the Chapter - Mechanical Equipment)
- Three static excitation systems with automatic voltage regulators
- Three generator busduct systems
- Three generator voltage switchgear assemblies with GCB, U3 also with GCB for the Small HP Unit
- Three main transformers
- 220 kV GIS and Take off Yards
- 220 kV cable systems
- LV AC auxiliary supply systems
- Auxiliary transformers
- Diesel generating unit
- 110 V DC and 230 V UPS systems
- Electrical protection systems
- Computerised Control System (CCS)
- Communication and security systems
- Earthing and lightning protection systems
- Power and control cable systems
- Lighting and small power installations
- Electrical workshop.

Part A3 Chapter 8 of the Detailed Design Report provides detailed information about the individual components, their main characteristics and the related design and construction requirements.

10.1.2 General Design Criteria

The dimensioning, design and layout of the various plant components and installations consider the following features and aspects:

- The ambient conditions at the project site
- Ratings to safely cope with all normal and fault conditions, avoiding any overstressing of material and equipment
- Equipment will be of standard design, providing highest degree of safety, reliability, availability and ease of operation
- Equipment arrangements to consider adequate space and access for transport, installation, commissioning, operation and maintenance.

At normal operating conditions the plant will be manned and operated in automatic mode, controlled from the Central Control Room (CCR) at TK V and/or UTK. Remote control from the National Load Dispatch Centre (NLDC) will also be possible.

The basic system configuration and equipment arrangements are shown on the general single line diagram and layout drawings.

10.1.3 Standards and Regulations

The layout and design of all equipment and installations will comply with the latest edition of the relevant IEC Standards.

10.1.4 Electrical System Data

The following system parameters have been considered in the design:

10.1.4.1 220 kV System

• Nominal voltage	kV	220
• Highest system voltage	kV	245
• Nominal frequency	Hz	50
• Number of phases		3
• AC withstand voltage	kV	460
• LI withstand voltage	kV	1050
• Rated short-time withstand current 3 s	kA	40
• Neutral earthing		solid

10.1.4.2 11 kV System

• Nominal voltage	kV	11
• Highest system voltage	kV	17.5
• Nominal frequency	Hz	50
• Number of phases		3
• AC withstand voltage	kV	38
• LI withstand voltage	kV	95
• Rated short-time withstand current 3 s	kA	50
• Neutral earthing		
System:		solid
Generator neutral:	resistor loaded earthing transformer	

10.1.4.3 400 V System

• Nominal voltage	V	400
• Nominal frequency	Hz	50
• Number of phases		3 / PE / N
• Fault level 3-phase, 1 s	kA	40
• Neutral earthing		solid

10.1.4.4 230 V System

• Nominal voltage	V	230
• Nominal frequency	Hz	50
• Number of phases		1 / PE / N

10.1.4.5 DC System

• Nominal voltage	V	110
• DC system		L+/L-/PE, isolated

10.1.5 System Configuration

Each generator will be connected to a main transformer (MT) with air insulated busduct systems and generator circuit breaker. The MT will be installed in the Transformer Cavern (TC).

A Small HP Unit will be considered in the Chapter Mechanical Equipment; this unit will be connected to the 11 kV generator voltage switchgear of Unit 3.

The HV side of the MT will be connected to the 220 kV GIS MT bays by means of single core XLPE cable systems. Four HV cable systems (Line In/Line Out - LILO arrangement) will connect the GIS transmission line (TL) feeder bays with the Take-off Yards (TOY1&2) and the double circuit TL, respectively.

The auxiliary power supply for the Power Station will be branched off from the generator voltage switchgears of Units 1 and 2 feeding two 11/0.4 kV station service transformers, each rated for 100% auxiliary load.

The 400 V main distribution board in the Powerhouse Cavern (PC) will be supplied by three station service transformers, two connected to the unit switchgears and one connected to the 11 kV switchgear located in the Terminal & Ventilation Building (TVB).

The 11 kV switchgear will be connected to the NEA grid with an isolating transformer. This feeder will serve as back-up but can, at the same time, be used for feeding the system from the PC. In case of power failure, a diesel generating unit (DGU) will supply power to the essential consumers.

The Valve Chamber (VC) and Outlet Structure (OS) will be supplied from the TVB distribution board by LV cable/OHL feeders.

The DC auxiliary supply systems for the PC and OB will comprise 110 V installations consisting of lead acid batteries, battery chargers, DC main and sub-distribution boards.

For supplying the CCS with the relevant peripheral installations, UPS systems will be provided in the PC and OB, respectively.

The basic configuration of the electrical systems is shown on the single line diagrams.

10.2 Generators

10.2.1 General

This Chapter describes the requirements for the 3-phase synchronous generators with auxiliary systems. The generators will be installed in generator pits built by the civil contractor. The generators will be directly coupled to FRANCIS turbines.

The generators will be totally enclosed, vertical salient-pole type, using air as the primary and water as the secondary coolant. They will be designed to run without undue noise or vibration over the full load range up to the maximum continuous rating, within the specified limits of frequency and voltage variation. The generators will comply with the relevant IEC Standards.

The generators will have a combined thrust and guide bearing above the rotor and a second guide bearing below the rotor.

The scope of work will comprise the following:

- Three generators
- Three sets of related auxiliary plant to provide complete, ready for service installations
- One set of special tools and appliances
- One set of special spare parts.

10.2.2 Main Characteristics

The generators will have the following main characteristics:

Number of units	no	3
Type of construction		IM8425/W41
Rated output	MVA	38
Rated speed	rpm	600
Runaway speed	rpm	1,100
Direction of rotation viewed from the top		clockwise
Rated current	A	1,995
Rated power factor		0.85
Rated frequency	Hz	50
Rated voltage	kV	11
Rated range of voltage regulation	%	±5
Flywheel effect approx.	tm^2	300
Insulation class Rotor/Stator		F/F
Stator winding connection		Star (Y)
Protection class		IP 44

Bearings	Upper combined thrust/guide and lower guide bearing
Braking system	Mechanical and electrical
Fire-fighting system	Total flooding clean agent fire extinguishing system or De Luge water spray system.

10.2.3 Design and Construction Requirements

The generators will safely withstand maximum stresses during normal operation, runaway-speed conditions, two phase- and three-phase short-circuit conditions, single phase earth fault, 120° out-of-phase synchronisation, magnetic unbalance at nominal speed with 50% of the poles short circuited, brake application etc. Seismic forces will also be taken into consideration for design of the generators.

The unit stresses under any operating condition will comply with requirements of the GTS. The thrust bearing will support the generator loads, the maximum hydraulic thrust loads and weights of the turbine rotating parts.

The generator foundations and housing will be a concrete structure provided by the civil contractor as per the Contractor’s requirements. Necessary sole plates, dowels, etc. required for erection of stator, brackets etc. will be supplied by the Contractor. The doors to the generator pit and necessary grated walkways in the generator pit will be provided by the Contractor. The construction of the generator will be such that the rotor poles, stator bars and generator air / water heat exchangers can be removed without disassembly of the upper bearing bracket. The design of thrust bearing housing will provide easy access to the bearing pads. Openings with removable covers will be provided for this purpose. Sectionalising of large and heavy components will be considered pursuant to transport limits.

The stator frame, core and winding will either be supplied in factory assembled condition to the maximum extent possible within the transport limits or assembled at site. The stator joints will not be near the phase terminals. The stator assembly and testing can also be performed in the generator pit. The rotor construction will facilitate the replacing the rotor poles. The poles will be interchangeable. The rotor will be supplied in the maximum possible assembled condition.

The design of the generator will consider the assembly and disassembly using the powerhouse crane. Necessary lifting devices will be provided under this Contract. The crane capacity and lifting details will be coordinated between the Contractors.

The slip rings will be made of stainless steel and will have helical grooved surface. They will be well insulated with epoxy glass insulation and designed for use with carbon brushes. The slip rings will be spaced sufficiently apart or separated from each other by a barrier to prevent any accidental short circuit. The slip rings will be easily accessible for inspection during operation and for maintenance. The brushes will consist of several carbon graphite and copper blocks placed in a massive ring. The arrangement and length of the field leads will enable to reverse the connections to the slip-rings without removing and dismantling the field leads or collector rings. Suitable carbon dust collectors and exhaust systems will be provided to prevent escape of carbon dust to the generator. Field leads from the collector rings to the field breaker cubicle will be routed in suitable ducts.

10.3 Excitation Systems

10.3.1 General

This section covers the detailed requirements for the excitation systems for three synchronous, hydraulic turbines driven generators, comprising excitation transformers, rectifier assemblies, field circuit breakers, field flashing and field suppression equipment, automatic voltage regulators, electrical braking, protection and control devices.

The scope of work will comprise the following:

- Three sets of excitation systems with excitation and braking transformers
- Three sets of related auxiliary plant to provide ready for service, reliable installations
- One set of special spare parts
- One laptop for testing, commissioning and setting of parameters.

The Small HP Unit described in the Chapter Mechanical Equipment will feature a brushless AC exciter with rotating diodes.

10.3.2 Main Characteristics

The main characteristics of the excitation system will be the following:

Ceiling excitation voltage at U_{rated}	p.u.	2.0
Ceiling excitation current	p.u.	2.0
Forcing duration, voltage/current	s	30/10
Excitation system voltage response time during forcing not more than	s	0.05
Accuracy of the voltage related to the set static characteristic	%	±0.5
Nominal voltage of AC auxiliary supply system	V	400
Nominal voltage of DC auxiliary supply system	V	110

The main characteristics of the excitation transformers will be the following:

Rating	kVA	by Bidder
Voltage ratio	kV/kV	by Bidder

10.3.3 Design and Construction Requirements

The excitation systems with the related parameters, performances, scope and methods of testing the excitation system components will be in conformity with respective IEC, IEEE or other approved Standards. The excitation system will be of the static type based on the latest technology and requiring minimum maintenance.

The equipment and will be installed in metal clad cubicles providing easy access for installation, inspection, maintenance, repair and adjustment. The proposed system will be a standard product of a competent manufacturer with proven, reliable operation record.

The rectifier assemblies will be directly fed from the dry type excitation transformer connected to the generator main circuit by means of cables. The excitation equipment will be designed for supplying without exceeding the continuous rating, ample excitation for the generator operating at its maximum capability at rated frequency, rated power factor and at 105% rated voltage.

Separate current and voltage transducers will be provided in the excitation cubicles to transmit the generator and excitation current/voltage values to the CCR.

10.4 Generator Main Circuits

10.4.1 General

For connecting the generators with the main transformers and to provide feeders for the excitation and station service transformers, busduct systems, generator circuit breakers and utility type 11 kV switchgear assemblies will be provided. A braking switch cubicle will be installed for each unit.

Neutral earthing cubicles will be provided for the generator neutral points.

The generator main circuit configuration with basic data is shown on the single-line diagrams. The typical equipment arrangements are shown in the Album of Drawings, Part E of the Detailed Engineering Design Report.

The scope of work will comprise the following:

- Three sets of 3-phase air-insulated busduct systems
- Three 11 kV generator circuit breaker assemblies
- One 11 kV generator circuit breaker assembly for the Small HP Unit
- Two 11 kV switchgear assemblies for SST feeders
- Three braking switch cubicles
- Three generator neutral earthing cubicles
- One set of special tools and appliances
- One set of special spare parts.

10.4.2 Busduct Systems

The connections between the generators and the generator voltage switchgear, generator circuit breakers and the MT located in the Transformer Cavern, will consist of three-phase, segregated air insulated busduct systems.

The proposed busduct design represents the current state of the art and provides a high degree of both reliability and safety.

The busduct systems will have the following main characteristics:

Type	3-phase, segregated, air insulated	
Nominal system voltage	kV	11
Rated voltage	kV	17.5
Rated frequency	Hz	50
Rated short-time withstand current (3s)	kA rms	50
Rated peak withstand current	kA peak	100
Rated current	A	2,500
Power frequency withstand voltage	kV rms	38
Lightning impulse withstand voltage	kV peak	95
Protection class		IP 65
Cooling		AN
Installation		Indoor

10.4.3 Generator Voltage Switchgear

Each generating unit will be equipped with an 11 kV generator voltage switchgear.

The switchgear will be type-tested, metal enclosed, utility type switchgear with single busbar arrangement, comprising the following:

- Generator circuit breakers with ancillary equipment
- Measuring and surge protection cubicles
- Riser and busduct termination cubicles
- Generator circuit breaker at Unit 3 for Small HP Unit
- SST feeder cubicles at units 1 and 2, with draw-out type SF6 or vacuum circuit breakers

The switchgear will have the following main characteristics:

Type	Metal-clad, indoor	
Circuit breaker	SF6 or Vacuum, draw - out	
Nominal system voltage	kV	11
Rated voltage	kV	17.5
Rated frequency	Hz	50
Rated power frequency withstand voltage:		
- To earth	kV rms	38
- Across open contacts	kV rms	45
Rated lightning impulse withstand voltage:		
- To earth	kV peak	95
- Across open contacts	kV peak	110
Rated current of bus	A	2,500
Rated current of GCB	A	2,500 & 400
Rated current of SST CB	A	400
Rated short-time withstand current (3 s)	kA rms	50
Rated peak withstand current	kA peak	100
Protection class		IP 42

10.5 Main Transformers

10.5.1 General

To raise the generator voltage to the grid transmission voltage each generating unit will be connected to a three - phase main transformer 230/11 kV (MT). The MT units will be installed in transformer bays located in the TC. The MT will be equipped with De Luge type water spray fire-fighting systems, provided under "Fire Protection Systems".

The scope of work will comprise the following:

- Three 3 - phase main transformers
- Three sets of related auxiliary plant to provide complete, ready-for-service installations
- One set of special tools and appliances
- One set of special spare parts.

10.5.2 Main Characteristics

The MT will have the following main characteristics:

Number of units	no	3
Type	3-phase, oil-immersed, outdoor	
Nominal rating		
- MT 1 & 2	MVA	40
- MT 3	MVA	44
Rated voltage:		
- HV	kV	230
- LV	kV	11
Voltage variation	%	± 2x2.5
Tap changer (NLTC)		Off-circuit
Rated frequency	Hz	50

Vector group		YNd11
Impedance voltage	%	10
Rated LI withstand voltage		
- HV	kV	1,050
- LV	kV	95
Cooling method		ODWF
Ambient temperature	°C	40
HV terminations		XLPE cable
LV terminations		Busducts

10.5.3 Design and Construction Requirements

The design of the main transformers will be based on the following conditions and requirements:

- The design of the transformers will be based on the site and service conditions with an ambient temperature of maximum 40°C.
- The rating of MT 1 & 2 will be 40 MVA, pursuant to the generator max. rating; the rating of MT 3 will be 44 MVA, pursuant to the generator plus the Small HP Unit nominal ratings.
- The maximum permissible temperature rise of the transformer oil (top oil) will not exceed 45 K; the average temperature rise of the winding will be limited to 55 K, and the winding hot spot temperature rise will not exceed 68 K. For the thermal evaluation of heat-run tests the hot spot factor (H) for power transformers with “core form type” core will be 1.5, and 1.7 for power transformers with “shell type” core.
- The transformers will be capable of operating continuously within the specified temperature rise limits at their rated power at 10% over - or under-excited operation.
- The hottest spot temperature at 115% excitation and for long-time emergency loading duties as per IEC 60076-7 will not exceed 130°C at 30°C ambient temperature under thermal steady state conditions.
- The transformers, completely assembled with bushings, cable boxes and/or flange connections, etc. will be designed and constructed to withstand without damages the effects of short circuits as per IEC 60076-5 for at least 3 s at rated conditions and after all loading conditions as specified in IEC 60076-7.
- The transformers will be designed with special attention to the suppression of harmonic voltages to eliminate wave form distortion and from any possibility of high frequency disturbances reaching such a magnitude to cause interference with communication circuits.
- Neutral points will be brought out by suitable means and will be earthed.
- The ODWF/OFWF cooling system will be equipped with redundant oil/water heat exchangers and circulating pumps.
- Appropriate measuring, monitoring and protection devices will be provided; the related control circuits will be connected to control and marshalling cabinets.

10.6 220 kV Switchgear and Take-off Yards

10.6.1 Gas Insulated Switchgear (GIS)

10.6.1.1 General

The generating units/MT will be connected to a 220 kV GIS. Four cable/transmission line feeders will be provided for connecting the Power Station to the UTK-KHIMTI Double Circuit TL by means of a LILO arrangement. The switchgear will be of the gas insulated, metal enclosed type with double busbar arrangement. It will comprise single-phase isolated switching devices and 3-phase enclosed main busbars.

The GIS with the related Local Control Cubicles (LCC) will be installed in a ventilated, purpose-built GIS room located at the upper floor of the TC.

The configuration and the basic arrangement are shown on the single-line diagrams and layout drawings, respectively.

The scope of work will comprise the following:

- Three generator / main transformer feeder bays with LCC
- Four transmission line, cable feeder bays with LCC
- One bus coupler bay with LCC
- One measuring and busbar earthing bay.

10.6.1.2 Main Characteristics

The GIS will have the following main characteristics:

Type	Gas insulated, double busbar	
Main Busbars	Three-phase enclosed	
Switching devices	Single phase gas insulated	
Nominal system voltage	kV	220
Rated voltage	kV	245
Rated frequency	Hz	50
Rated power frequency withstand voltages	kV	460
Rated lightning impulse withstand voltage	kV peak	1,050
Rated switching impulse withstand voltage	kV peak	850
Rated current of busbar	A	2,000
Rated current of feeder	A	2,000
Rated short time withstand current (3 s)	kA rms	40
Rated peak withstand current	kA peak	100
Partial discharge level at 1.5 x U/√3	pC	<5
Enclosure Material		Al/Al alloy
Conductor Material		Al/Al alloy
Protection class of enclosure		IP 65
Protection class of operating mechanism		IP 54

10.6.1.3 Design and Construction Requirements

The GIS will be the type tested standard product of a competent manufacturer with proven track record.

The GIS will be of modular design and factory assembled to shipping units of appropriate dimensions and weight. Future extensions will be easily accomplished by adding extra feeder bays without dismantling any major parts of the equipment. Components and assemblies will be of standard manufacture and inter-changeable.

Each switchgear component will be sectionalised in modular form to enable disconnection from the system for replacement or maintenance without loss of pressure in adjacent modules or long shutdowns. Busbar compartments will be sectionalised into appropriate lengths as can be handled by the crane or erection per-sonnel.

The various switchgear components will be preassembled in the Contractor's factory to the largest modular units compatible with the limitations of shipping and installation capabilities. The overall layout of the GIS and associated equipment will provide a compact arrangement facilitating installation and assembly with adequate access to any operational area for inspection and maintenance.

Ladders, platforms and gratings necessary for maintenance and operation of the equipment will be included if the equipment is not easily accessible from the ground level.

Suitable means of expansion will be provided in the metal enclosures to compensate thermal expansion and contraction of the SF6 equipment and to facilitate alignment of the switchgear assembly.

10.6.2 Take-off Yards (TOY1&2)

10.6.2.1 General

The 220 kV Double Circuit TL UTK-KHIMTI will terminate at the TOY1 (in) & TOY2 (out) and will be connected to the GIS by four HV cable systems. The TOY1&2 will be arranged adjacent to the CVT portal, as shown in the Album of Drawings, Part E of the Detailed Engineering Design Report.

The outdoor installations of the TOY1&2 will comprise the following:

- Cable sealing ends (reference is made to Chapter 3.7)
- Window type CT
- Disconnecting and earthing switches
- Surge arresters with surge counters
- Take-off gantry and supporting structures
- Termination of the OPGW to termination boxes in the TVB, interface and bus link with CCS
- LCC to accommodate the auxiliary power supply and control equipment.

10.6.2.2 Design and Construction Requirements

The principal layout of the TOY1&2 is shown in the Album of Drawings, Part E of the Detailed Engineering Design Report.

The outdoor equipment will be the standard product of a reputed manufacturer, appropriate for the site conditions.

10.7 220 kV Cable Systems

10.7.1 General

The connections between the GIS feeder bays, the MT and the TL at the TOY1&2 will be carried out with 220 kV XLPE type single core cable systems.

The cables between GIS and MT will be routed in the upper floor of the transformer cavern. The cables between GIS and the TOY1&2 will be routed in the Cable and Ventilation Tunnel (CVT).

The HV cable systems will comprise the following:

- Three sets of three-phase cable systems between the GIS feeder bay and the MT
- Three sets of GIS cable sealing ends for the MT feeder bays
- Three sets of MT cable sealing ends
- Four sets of three-phase cable systems between the GIS feeder bay and the TOY1&2
- Four sets of GIS cable sealing ends for the TL feeder bays
- Four sets of outdoor cable sealing ends for the TOY1&2.

10.7.2 Main Characteristics

The equipment will have the following main characteristics:

Type designation		2XS(FL)2Y
Nominal system voltage	kV	220
Rated voltage	kV	245
Rated frequency	Hz	50
Rated power frequency withstand voltage	kV	460
Rated lightning impulse withstand voltage	kV peak	1,050
Rated switching impulse withstand voltage	kV peak	850
Rated continuous capacity per system:		
- Link between GIS and MT	MVA	44
- Link between GIS and TL (In/Out)	MVA	540/660
Rated short-time withstand current (3 s)	kA rms	40
Rated peak withstand current	kA peak	100
Conductor material		Copper
Conductor cross section		By Tenderer
Insulation material		XLPE
Maximum conductor temperature:		
- At rated conditions	°C	90
- During short circuit 1 s	°C	250

10.7.3 Design and Construction Requirements

The 220 kV cables and accessories will be the standard product of a reputed manufacturer with a proven track record. The construction of the cables will comply with the specified requirements and the related ICEA and IEC Standards.

The single core, XLPE insulated cables will be longitudinal and radial watertight, suitable for installation in air, ground, embedded in lean concrete, on cable trays and ladders, in cable ducts and pipes.

The maximum continuous current carrying capacity of each cable system will be based on the maximum loading at site conditions with due consideration of the load reduction factors. The conductor cross-sections will be adequate to carry the fault currents at specified conditions without deterioration of the dielectric properties.

Following the TL loading limits, load shedding to 570 MVA will be necessary in case of failure of one system.

10.8 LV AC Auxiliary Supply Systems

The Power Station will be provided with a reliable, easy to operate and maintain AC auxiliary supply system consisting of 11 kV switchgear, station service transformers, LV main switchboard, LV distribution boards, diesel generating unit and LV cable systems. In this Chapter, mainly the requirements for the MV and LV switchgear are described; the requirements for the other equipment are described in the relevant Chapters.

The scope of work will comprise the following:

- One 11 kV switchgear installed in the TVB
- One LV main switchboard for the PC
- Three unit MCC
- One unit MCC for the Small HP Unit, as appropriate
- One distribution board for the TC
- One distribution board for the GIS Room
- Two distribution boards for the Workshops
- One distribution board for the TVB
- One distribution board for the OB
- One distribution board for the Valve Chamber (VC)
- One distribution board for the Outlet Structure (OS)
- One lot of sub-distribution boards
- One lot of related equipment to provide complete, ready for service installations
- One set of special spare parts.

10.9 Auxiliary Transformers

The station auxiliary supply system will be fed by three station service transformers (SST), each rated for 100% of the auxiliary load. The TVB, Ob and Employer´s Camp will be supplied by two distribution transformers.

The cast-resin transformers will be housed in sheet steel enclosures and installed adjacent to the LV switchboards. The system configuration is shown on the general single-line diagram.

The scope of work will comprise the following:

- Three station service transformers 1,000 kVA
- Two distribution transformers 400 kVA for the TVB
- Five sets of auxiliary plant to provide complete, ready-for-service installations
- One set of special spare parts.

10.10 Diesel Generating Unit

An emergency diesel generating unit (DGU) will be provided at the TVB. The DGU will be installed in an enclosed and ventilated purpose-built engine room which will contain the complete installations including the day tank, cooling system, control equipment, starting battery and battery charger. The storage tank will be of the underground type.

The scope of work will comprise the following:

- One DGU rated 400 kVA, 0.4 kV, 1,500 rpm
- Related auxiliary plant to provide complete, ready for service installations
- One set of special tools and devices

- One set of special spare parts.

10.11 DC and UPS Auxiliary Supply Systems

For the safe and reliable operation of the control, protection and communication equipment 110 V DC auxiliary power supply systems will be provided at the PC and OB.

UPS systems will be provided at the PC and OB.

The basic system configuration of the DC auxiliary supply and UPS systems is shown on the single line diagrams. The systems will be supplied by battery chargers and batteries, each rated for 100 % of the total auxiliary load. The DC equipment will be installed in purpose-built equipment rooms. The locations and tentative layouts are shown on the layout drawings.

The ratings of batteries, battery chargers and inverters are not finally selected and will be adapted in accordance with the approved power demand calculations to be elaborated by the Contractor during the construction design, based on the data of the selected equipment.

10.12 Protection Systems

Protection systems will be provided to isolate faulty systems in the shortest time technically possible, to limit damage and to maintain healthy systems in stable operating conditions. The systems will feature a high degree of selectivity and discrimination between faulty and healthy circuits. The protective relays will be of the high speed, numerical type arranged in protection cubicles including all ancillary devices such as interposing transformers, relays, test facilities and power supply units.

The scope of work will include the following protection equipment:

- Three sets of unit protection systems to provide the complete range of electrical protection for the generators, busduct systems, main transformers and HV cable systems
- Four sets of protection systems to provide the complete range of electrical protection for the 220 kV transmission line feeders including tele-protection & communication UTK-TK V-KHIMTI
- One set of protection system to provide the complete range of electrical protection for the 220 kV busbar systems and the bus coupler
- One set of unit protection systems to provide the complete range of electrical protection for the Small HP Unit
- Protection systems to provide the complete range of electrical protection for the 11 kV switchgear, SST, distribution transformers and DGU
- One set of special tools and appliances
- One set of special spare part.

10.13 Computerised Control System

The Power Station will be provided with a state of the art control system, comprising:

- One Power Station Supervisory, Automation and Control System - in the following referred to as 'Computerised Control System - CCS'.

The scope of work will include all required studies, calculations, design, engineering, manufacturing, supply, submission of drawings and documents, shop and site testing, erection, adaptation to related equipment, commissioning and training services for the CCS and its components.

The CCS will be fully developed, debugged, commissioned and tested for manual and fully automatic control of the Power Station. The CCS will principally be composed of the following systems, components and ancillaries:

10.14 Communication and Security Systems

The communication and security systems will comprise the following:

- Telephone system
- Telecommunication system with interface equipment OPGW to UTK, KHIMTI & NLDC
- Radio communication system
- Public address system
- Clock system
- Closed circuit television system (CCTV)
- Access control system
- One set of special spare parts.

10.15 Earthing and Lightning Protection Systems

Earthing and lightning protection systems will be provided for the entire Power Station with related structures and buildings.

The scope of work will comprise the following:

- Sub-grade earthing systems
- Above ground indoor and outdoor earthing systems
- Potential gradient control systems
- Lightning protection systems.

10.16 MV, LV and Control Cable Systems

Power and control cable systems will be provided for the entire Power Station with all related buildings and structures. The cable systems will comprise all MV and LV power, communication, control and instrumentation cables, the necessary accessories such as cable terminations, cable trays, conduits, supports, fixing and mounting materials to provide complete, ready for service systems.

The scope of work will comprise the following:

- 11 kV single core power cable systems
- LV power cables for AC and DC systems
- Communication, control and instrument cables
- Fibre optic cables for external/internal data transfer systems
- Self-supporting areal cable for power and control circuits between OB and the 220 kV LILO by-pass switching station
- Material for supporting, laying and terminating of the cables
- One set of special spare parts.

10.17 Lighting and Small Power Installations

Indoor and outdoor lighting systems with small power service installations will be provided for the entire Power Station, related roadways and service areas.

The scope of work will comprise the following:

- Lighting and small power installations for the entire Power Station with all pertinent buildings and structures
- Lighting and small power installations for the service areas and associated roadways, extending up to 500 m from the Power Station
- Portable and rechargeable lamps with charging units
- One set of special spare parts.

10.18 Electrical Workshop

The Power Station will be provided with an electrical workshop. The basic outfit of the electrical workshop will comprise the following:

- Work benches, tool cabinets and chairs
- Tool machines and electrical portable tools
- Standard tools for electrical and mechanical fitters
- Instruments and appliances.

11 TRANSMISSION LINE

11.1 Introduction

NEA, for power evacuation from Tamakoshi V HEP recommended pie connection of Tamakoshi V to Upper Tamakoshi – Khimti 220 KV Transmission line at its switchyard. The existing 220 KV double circuit line from Upper Tamakoshi to Khimti Substation, constructed using twin Bison ACSR conductor was initially designed to carry 456 MW power. With the additional 100 MW power from Tamakoshi V HEP, the total power to be transmitted will become 556 MW. Using the Japanese code, the maximum allowable current for Bison ACSR was calculated and it was observed that the double circuit Bison conductor in twin bundle configuration is found to be adequate for transmission of a total power of 556 MW and, hence, the conductor size, for the design of loop-in-loop connectivity of the Tamakoshi V HEP is suitable, without any alterations to the existing Upper Tamakoshi – Khimti Transmission line.

This conclusion is consistent with the recommendations of NEA's Transmission System Master Plan from 1998.

The purpose of the Detailed Engineering Design was to elaborate the design in accordance with the approved design criteria, explain worst case probabilities and loading conditions, elaborate design methodology and freeze major project parameters. The transmission line design engineering report describes:-

- Selection of conductor, insulators and towers for transmission line,
- Route alignment / realignment
- Plan profile, angle pole locations and type of pole,
- Sag tension calculations, tower schedule, final plan profile, BOQ and cost estimate,
- Design of towers, tower earthing and tower foundations
- Drawings and technical specifications.

Engineering drawings were prepared in accordance with the engineering design and show the general outline and enough detail regarding the structures, material and equipment to enable the contractors and suppliers to prepare and submit competitive bids.

This chapter provides summary information on the transmission line design. Details about the design are compiled in Part A3 Chapter 9 of the Detailed Design Report.

11.2 Components of 220 kV Transmission Line

The following are the components for the transmission line:

- Tower & Tower Accessories
  - Suspension and Angle Towers
  - Phase Plate
  - Number Plate
  - Danger Plate

  - Circuit Plate
  - Tower Earthing (Pipe / Counter poise)

- Earth Wire

- Conductor

- Insulators

- Accessories for Conductor and Earth wire
  - Performed Armour rods
  - Mid span compression joints
  - Vibration Dampers for ACSR
  - Repair sleeves
  - Flexible copper bonds
  - Bundle spacers
  - Suspension clamp for earth wire
  - Tension clamp for earth wire

- Insulator String Hardware (Disc Insulator String)
  - Anchor Shackle
  - Chain Link
  - Ball Clevis
  - Arcing horn holding plate
  - Yoke plate
  - Socket clevis
  - Arcing horns
  - Corona controlled ring/grading ring
  - Clevis Eye
  - Free centre type/Armour grip suspension clamp for suspension strings
  - Compression type dead end clamp
  - Sag adjuster
  - Balancing weight

11.3 General

11.3.1 Service Conditions

Following are considered service conditions for 220 KV transmission line.

•	Maximum ambient temperature	Degree C	50
•	Minimum ambient temperature	Degree C	0
•	Annual Average Temperature	Degree C3	2
•	Maximum Relative Humidity	Percent range	(10-100)
•	Maximum altitude above mean sea level	Meter	2000
•	Maximum Wind Velocity	m/sec	47
•	Isoceraunic Level	(Days/year)	60
•	Rainfall	mm/annum	1000
•	Atmospheric Pollution	Light to medium	

11.3.2 System Parameters

The following system parameters have been considered in the design.

•	Rated voltage	kV	220
•	Highest system voltage	kV	245
•	Conductor Size	sq. mm	431.3
•	Thermal rating of conductors	Amp.	
•	Basic Insulation Level	kV(peak)	1250

11.3.3 Clearance requirements

The following system parameters have been considered in the design:

<u>Electrical Clearances</u>

•	Minimum Ground clearance	m	7.0
•	Between Phase and Earth	m	7.0
•	Between Phase to Phase	m	7.0
•	Clearance of Power Conductor over the highest flood level in case of non-navigable river	m	5.1
•	Minimum mid span vertical clearance between OPGW and the nearest conductor for 245 KV system	m	8.5
•	Minimum clearance between power line to power line crossing	m	4.58

- Minimum clearance between power conductor crossing
- Telecommunication line (mm) for 245 KV system m 3.05

Live Metal Clearance

The minimum live metal clearance to be provided between the live parts and steel work of tower are as given below.

Table 11-1: Live Metal Clearance

Description	Swing Angle	Live Metal Clearance (2 m Elv)
	0 Degree	2400 mm
Suspension String	15 Degree	2230 mm
(Single / Double)	30 Degree	2060 mm
	45 Degree	1885 mm
Tension String		
(Single/Double)		2400 mm
	0 Degree	2400 mm
Jumper	10 Degree	2400 mm
	20 Degree	2060 mm

11.3.4 Composition and Cross Section of Conductor

The ACSR Bison conductor will generally conform to the following:

- Stranding and wire diameter 54 / 3.00 mm Aluminium + 7/3.00 mm steel
- Number of strands
  - Steel core 01
  - 1st steel layer 06
  - 1st Aluminium layer 12
  - 2nd Aluminium layer 18
  - 3rd Aluminium layer 24
- Sectional area of Aluminium 381.8 Kg
- Total sectional area 431.3 Kg
- Overall diameter of the conductor 27.0 mm
- Approximate mass of the conductor 1.444 Kg/km
- Calculated DC resistance of the conductor 0.0753 Ohm/Km
- Minimum UTS 120.9 KN

11.3.5 Composition and Cross Section of Earth Wire

• Stranding and wire diameter	07 /3.35 mm Steel
• Number of strands	
- Steel core	01
- Outer steel layer	06
• Total sectional area	73.65 Kg
• Overall diameter	10.05 mm
• Approximate mass of the conductor	0.55 Kg/km
• Calculated DC resistance of the conductor	2.5 Ohm/Km at 20 degree C
• Minimum UTS	68.4 KN

11.3.6 Standards and Basic Design Documents

The design will be in accordance with the lines, definitions and criteria set forth in standards, reports and recommendation issued by the following institutes- Priority is in the order mentioned. The latest issues of the pertaining documents will prevail unless otherwise stated.

NS	The Nepal Bureau of Standards and Metrology
IS	The Indian Standards Institution
BS	The British Standards Institution
ASTM	The American Society for Testing of Materials
IEEE	Institute of Electrical & Electronics Engineers
IEC	International Electro-technical Commission

11.3.7 Codes and Standards for Design

The overall design standard used for the design and construction of the transmission line currently under construction from Upper Tamakoshi to Khimti and further to Dhalkebar line is the Indian Standard:

- IS-802 Use of Structural Steel in Overhead Transmission Line Towers – Code of Practice, Part 1 Materials, Loads and Permissible Stresses

  Section 1 (1995) – Materials and Loads

  Section 2 (1992) – Permissible Stresses

In preparation of IS-802 the Bureau of Indian Standards has derived assistance from various publications including ASCE-52 and IEC-826 being the standards recommended used in the former studies for the 220 kV Gongar – Khimti transmission line.

11.4 Transmission Line Route

11.4.1 General

NEA, through its Project Development Department, carried out a detailed survey of the transmission line for the Tamakoshi V HEP from:

- **A;** Tamakoshi V HEP Power House to Singati 132 KV (under construction) Substation consisting of 4.5 Km line in Dolkha district, Jankpur zone of Nepal.
- **B;** Tamakoshi V – HEP Power House to nearest angle point of 220 KV Upper Tamakoshi – Khimti section consisting of 1.5 Km in Dolkha district of Nepal.

The objective was to determine the most suitable and shortest route alignments and carry out a detailed route survey of the 220/132 KV transmission line section as per sections A and B.

The methodology for this work included basic data collection, desk study, a reconnaissance survey and a detail alignment survey.

Based on the desk study, the 220 KV Upper Tamakoshi – Khimti line was chosen as the most suitable alignment from the nearest angle point (AP 17). The detailed survey for the transmission line route alignment followed the final route recommended during field reconnaissance survey.

The engineering services of NEA's Project Development Department recommended Pie connections for Tamakoshi V to upper Tamakoshi – Khimti 220 KV transmission line at Tamakoshi V Switchyard.

Based on a power evacuation study report, the possibility of a 132 KV transmission line as an alternate option was evaluated and our findings are presented in Annex A.

It can be seen that power evacuation, using a 132 KV transmission network from Singati and Lamosangu, is not technically feasible due to the conductor loading limitations and mandatory N-1 reliability requirements as per grid code rules. Also, there is a space constraint in the Singati substation and difficulties in constructing line in hilly terrain. A third option for transmitting power on a 132 KV network requires the construction of a 55 Km long DC line from Tamakoshi V to Barbise, expansion of 132 KV bay at Barbise and the installation of a 132 / 400 KV transformer for further connection to the 400 KV UTK- Barbise line (currently under construction) near Lamosangu. This third option is not economical.

We are, therefore, in agreement with NEA's recommendations that a 220 KV line route with pie connectivity of the Tamakoshi V HEP is the best and most economical option

Based on the available data and field assessment reports the most appropriate route was selected and is shown below. The route is situated in mountainous regions and there is no cultivation in this area. The land is forest and bushes (Bairon Land) and is partly accessible by small roads. There are no road crossings and river crossings along the selected route.

To evacuate power from Tamakoshi V HPP to 220kV Khimti Substation Tamakoshi V HPP must construct:

- About 1.6 km long 220 kV double circuit transmission line from AP17/0 to Upper Tamakoshi HPP GIS bus bar (loop in) and;

- About 1.9 KM long 220 kV double circuit transmission line from Tamakoshi V GIS bus to the Upper Tamakoshi Tower No AP-18 (loop out).

Salient Features

• Route Length	3.5 Km
• No. of Angle Points	18
• No. of Road Crossings	Nil
• No. of River Crossings	Nil
• Highest Elevation along the Line	1,773.93 m
• Lowest Elevation along the Line	1,005.17 m

11.4.2 Description of Transmission Line Route Alignment

While the 220 KV transmission line loop-In connectivity of Tamakoshi V HEP starts from angle pole location AP 17/0 of existing Upper Tamakoshi – Khimti Line and uses TOY 1 for interconnecting cables from GIS busbars, Loop-out connectivity with existing line terminates on angle pole location AP 18/0 and uses TOY 2 for interconnection with outgoing cables from the GIS busbars.

The route extends along ~~on~~ the rolling terrain of the Mahabharat range. More than 80 % of the line route traverses hilly terrain covered with bushes and trees. The route was especially selected and surveyed to eliminate river and road crossings. The highest angle point deflection along the route is 118.65 degrees.

It is further envisaged that, if a fault in the PowerStation area occurs and the need arises that a bypass of Tamakoshi V is required, this will be possible without g disturbing the grid operations. This will be accomplished withan outdoor type switching station constructed near ToY 2 structure. During this period Tamakoshi V will be isolated from the transmission line and supply can be restored by closing the disconnecting switch in the switching station.

Due to steep elevations, this short length of transmission line will have approximately 18 tower locations. These locations will have the following span lengths based on the observed suitability of locations for the tower placement from the survey report.

• up to 350 meter	8
• up to 200 meter	8
• up to 150 meter	5

11.4.3 Line route accessibility

Between angle points AP 17/0 and AP 18/0 of the existing Gongar – Khimti line of the selected route there is presently no road and, hence, the line route will have to be approached on foot only leading to head loading of equipment and material from roads to the actual construction site.

The following parameters, which is required for transmission line design, was considered.

• Average ambient Temperature	32 Degree C
• Maximum ambient temperature	50 Degree C

•	Minimum ambient temperature	0 Degree C
•	Relative humidity	96%
•	Maximum rainfall	2,000 mm/annum

11.4.4 Outdoor Switching Station

Should there be a fault or major outage in the Powerhouse area, the transmission line looping connection at Tamakoshi V may require a bypass arrangement to restore grid power in the minimum possible time. With this view, a switching station in the outdoor area of Tamakoshi V will be constructed. This switching station will allow bypassing of the Tamakoshi V project (LILO arrangement).

It will comprise the following:

- Cable sealing ends
- Disconnecting and earthing switches
- Surge arrestor with surge counters
- Take-off gantry and supporting structures
- Termination of the OPGW to termination boxes in the TVB, interface and bus with CCS
- LCC to accommodate the auxiliary power supply and control equipment.

11.5 Selection of Conductor

11.5.1 General

Consistent with the Transmission System Master Plan of Nepal, NEA designed and constructed a 220 KV double circuit transmission line from Upper Tamakoshi to Khimti Substation using ACSR Bison conductors in a twin bundle configuration. This line was originally designed to transmit 456 MW power from Upper Tamakoshi. Tamakoshi V HEP is designed for approximately 100 MW power output. Grid connectivity of Tamakoshi V HEP is proposed by making LILO arrangements of Upper Tamakoshi to the Khimti 220 KV double circuit line at its switchyard. The most suitable route for pie connections was found to be from angle towers AP 17/0 and AP18/0.

The design compatibility characteristic of transmission line with the existing grid lines was verified using the-Japanese code. It was proven that bison twin bundle double circuit configuration is adequate to transmit 556 MW power. The maximum allowable current of ACSR bison in twin bundle configuration is 521 Amp. The calculations are given in the following.

Double circuit Bison conductors with twin cores can transmit 2x376 MW = 752 MW which can easily transmit the total power of 556 MW (456 MW of Upper Tamakoshi +100MW Tamakoshi V).

The design of the Bison twin core double circuit transmission line can safely transmit the total power from Upper Tamakoshi and Tamakoshi V HPPs. The computed allowable current of 521A is low compared to the values given in the conductor parameters details (595 A) andThis shows that the Bison twin core conductors, in each of the two circuits have, much more transmission capacity than required by the design requirement.

11.5.2 Transmission Line Capacity

11.5.2.1 General

The transmission line capacity of 521A equals ~~to~~ 288 MW for a single core Bison conductor and for double core it is 376 MW.

Double circuit Bison conductors with twin cores can transmit 2 X 376 MW = 752 MW which can easily transmit the total power of 556 MW (456 MW of Upper Tamakoshi +100MW Tamakoshi V).

The design of the Bison twin core double circuit transmission line is safe to transmit the total power from Upper Tamakoshi and Tamakoshi V HPPs. The value of the calculated allowable current of 521A was in lower than the values given in the conductor parameters details (595 A) and this shows that Bison twin core conductors in each of two circuits have much more transmission capacity than required by the design to transmit the combined power from these two power plants to 220kV Khimti Substation.

11.5.2.2 Sub conductor spacing

Optimizing sub conductor spacing is important because it influences the surge impedance loading of the line, corona and radio interference losses and the weight of the towers. The cost of series and shunt compensation, due to change in the reactance of the line, is also affected to some extent. A phenomenon called sub-span oscillation is also related to sub conductor spacing. All the above factors affect the cost per MW capability of line.

From the experience and practices adopted by different countries for 400 /500 KV lines, it was found that the optimum sub conductor spacing varies between 30 to 45 cm. With increased interphase spacing and reduced sub-conductor spacing, the transmission capability is reduced due to increased inductive reactance and reduced capacitive reactance of the line.

The tower weights are influenced with the change in the sub-conductor spacing due to the following reasons:

- For given configuration of towers, and specified angle of shield, the cross arm length increases with the increase in sub-conductor spacing. This involves an increase in the height and hence the weight of towers.
- With an increase in the cross arm length, the torsion load increases, under broken wire conditions, resulting in an increase in the weight of the bracings.
- The I^2R and corona losses also are affected with a change in sub conductor spacing due to change in the surge impedance loading and surface gradient.
- Sub conductor spacing is also related to the low frequency and high amplitude oscillations under severe wind conditions and cause clashing of sub conductors in the mid span and thus resulting in damages to the suspension fittings, spacers and to conductors. Through research and experience it has been shown that a sub conductor spacing to diameter ratio should be between 14-17. With 450 mm sub-conductor spacing and conductor diameter of 27 mm the ratio is 16.66 and hence acceptable.

In view of the above, the 220 KV Tamakoshi V transmission line, ~~the~~ spacing between sub-conductors was designed to be 450 mm in a horizontal configuration

11.5.3 Selection of Overhead Ground Wires (GSW AND OPGW) and Angle of Shield

The earth wires are the conductors arranged above the phase conductors and grounded at every tower. They are mainly provided for protection against direct strikes and distributing the current in two or more paths, thus reducing the voltage drop.

As per IS 5613 (Part 2), low resistance earth wire at an angle of 30 degrees to the vertical will be provided for a 220 KV transmission line system. However, in the Himalayan foothills of India, a transmission line designed for large power evacuation, it is recommended to reduce the shielding angle to 20 degrees. Consequently, for the Tamakoshi V transmission line, an angle of shielding of 20 degrees was considered. In India an earth wire of diameter 9.45 mm (7/3.15) is normally used for 220 KV systems. However, NEA uses 10.05 mm diameter earth wire for its 132/220 KV lines. Consistent with the existing 220 KV transmission line from Upper Tamakoshi to Khimti, two ground wires GSW and OPGW, each with a diameter 10.05 mm (7/3.35) and 11.5 mm respectively were designed to run horizontally on top of the towers and conductors at a shielding angle of 20 degrees. While the GSW ground wire is for protection against direct lightning strikes, the OPGW ground wire has the additional responsibility to ensure the availability of broad band communication between substations for all power system applications.

11.6 Selection of Insulator

11.6.1 Creepage requirements

For a 220kV transmission line network, the length of the insulator or insulator string is determined based on their “power frequency withstand voltage” and “lightning impulse withstand voltage”. Power frequency withstands voltage capabilities of the power line insulators mainly dependent upon the creepage distance of the insulators used. IEC 60815 [3] gives information regarding the selection of insulators in polluted conditions and the minimum required creepage distance depends on the environmental pollution level. The pollution categories relevant to specific creepage distances and terrain details which assist in identifying the right pollution category is available from IEC-60815 [3].

The transmission line for Tamakoshi V HEP is likely to pass through less inhabited areas and is away from the industrial towns and Light 1 (light pollution) applies. The minimum creepage distance required for lightly polluted conditions for the highest system voltage of 245 KV as per IEC 60815-3 = 16 X 245 = 3920 mm. Creepage distance of a disc insulator = 292 mm. nos. of discs required to achieve the minimum creepage distance requirements = 3920 / 292 = 13.42.

Therefore,the minimum no. of discs required for suspension and tension tower locations will be 14 and 15 respectively.

11.6.2 Insulator string configuration and Altitude Correction Factor

Survey data for Tamakoshi V line indicates the maximum elevation of tower location is more than 1700 meter above mean sea level. Since the towers of this line are required to be used for future lines, it is prudent to design the towers for an altitude of 2000 meters (altitudes normally not associated with snow fall) from MSL.

The minimum live to metal clearance distance for 220 KV lines with zero degree swings as per IS 5613 = 2130 mm.

Applying altitude correction factor of 1.25 % for every 100 meter over and above 1000 meter, the live to metal clearance for 2000 meter altitude = 2130 X (1.0125)^9 = 2382 meter. In order to meet this clearance, a suspension disc insulators string, comprising of 16 no. of 145 mm size discs (minimum creepage of 292 mm) and for tension towers 17 no. of such discs, is required.

11.6.3 EMS Strength

The rating of the insulators for a suspension string must be selected considering the criteria that the loads on the insulators do not exceed 20 % of the rating under everyday conditions and 70 % of the rating under maximum load conditions.

For the mechanical rating of the tension insulators the weakest point considered are the conductor joints which are designed for 95 % of UTS of the conductor. The insulators are generally designed 10 % more than that their rating, taking into account the larger reduction of insulator strength ~~reduction~~ due to aging.

The minimum UTS of twin bundle Bison, as per BS-215, part 2 is 120.9 KN, ~~and~~ The average mass/span equals 1444 kg/km. Considering the above parameters and the proposed weight span limits, a single insulator string of 70 KN EMS discs for suspension towers and two strings comprising each of 120 KN EMS discs for angle towers meets the strength requirements.

Since the transmission line is not foreseen to have phase transposition, a requirement of single tension strings were not envisaged.

11.6.4 Insulator Assemblies

11.6.4.1 Suspension Insulator Hardware

Fittings will be provided with an attachment to the cross arms, arcing horns at both live end and earthing end of the string and the helical type performed armour rods. Mechanical strength of a complete set of the suspension insulator assembly will not be less than 120 KN. The suspension clamp will be suitably equipped with armour rods.

11.6.4.2 Tension Insulator Hardware

Fittings will be provided with an attachment to the cross arms arcing horns at both live end and earthing end of the string and dead end clamp for the conductor. The dead clamp will consist of a compression type galvanized steel core with eye end, and a compression type aluminum dead end body and an aluminum jumper connector. Breaking strength of the tension insulator set will be not be less than 120 KN or 160 kN except the dead end clamp. The breaking strength of the dead end clamp will be not less than 95 %, and the jumper connector will be not less 30 % of the ultimate tensile strength of the conductor, respectively.

The electrical conductivity and current carrying capacity of the dead end clamp will be not less than that of the equivalent length of the conductor.

11.7 Towers for Tamakohi V Transmission Line

220 KV Transmission Line for power evacuation from Tamakoshi V HEP will have following type of towers.

- Double Circuit Towers (DA, DB, DC & DD)
- Special Towers (using DA, DB, DC & DD towers with location specific reinforcements)

The towers will be self-supporting, hot dip galvanized, latticed steel type and designed to carry the line conductors with necessary insulators, earth wires and all fittings under all loading calculations.

The towers will be fully galvanized using mild steel and / or high tensile steel sections. Bolts and nuts with spring washers re to be used for connections.

All types of towers will provide body extensions in order to obtain different tower height above the standard tower height and be so designed to make tower erection possible on sloping terrain by using various leg extensions.

Further details about the tower design can be taken from Part A3 Chapter 9 of this report.

11.8 Sag Tension Calculation

Extensive sag tension calculations were carried out for the transmission line of Tamakoshi V HEP. The results are presented in detail in Part A3 Chapter 9 of this report.

11.9 Types of Foundations

Design of the transmission line foundations will be conducted by the Contractor after completion of a thorough and detailed soil investigations of the sites.

The tower foundations are generally chosen to correspond, first, to the soil conditions where the tower is erected, and second, in terms of maximum reliability and cost effectiveness.

Proper drainage around the foundation area must be ensured.

The foundation will be designed to resist reactions resulting from the loading in the normal and broken wire conditions including uplift, compression and horizontal forces. The safety factor of the foundations will be according to code requirements. See for example IEEE Guide (2001) and EN 50341-1: 2012.

Several different types of tower foundations were considered for the construction of the Tamakoshi V HEP transmission line. Details about the various types are presented in Part A3 Chapter 9 of this report.

11.10 Considerations related to Power Evacuation at 132 kV Level via Singati S/S

Here below the advantages/disadvantages of evacuating power from Tamakoshi V HEP through Singati S/S. are reviewed.

A 132/33 kV substation is being constructed by NEA at Singati of Dolakha district, which is about 15 km from the proposed site of Tamakoshi V HEP. This station comprises of 4 no. 132 kV line feeders, and one 132/33 kV, 30 mVA transformer. A 132 kV double circuit transmission line with BEAR conductor is also under construction between Singati S/S and Lamosangu S/S. Single line diagram, plot plan of Singati S/S and the transmission line in the vicinity are shown in Figures 1, 2 and 3 below.

1. Several hydropower projects are under construction in the vicinity of Singati substation. Their total capacity is about 140 MW as shown in the following table. All these projects have already completed PPA with NEA with their connection point at Singati substation.

Table 11-2: Hydropower Projects under Construction in the Vicinity of Singati Substation

Name of Project	Status	Capacity (MW)
Sipring Khola	Operation	10.00
Khani Khola-1	Under Construction	40.00
Khani Khola Dolakha	Under Construction	30.00
Lower Khare	Under Construction	11.00
Khare Khola	Under Construction	24.00
Singati Khola	Under Construction	25.00
TOTAL		140.00

The transmission line under construction between Singati and Lamosangu is a double circuit line with BEAR conductor whose thermal capacity is 142 MVA (for 1 circuit). As per the rules of NEA Grid Code, only a maximum of 142 X 1.2 = 170 MW is allowed under contingency (n-1) criteria. If Tamakoshi V HEP is connected to Singati substation, the total power evacuation requirement will be 142 + 96 = 238 MW which is far above maximum allowable power. Hence, the evacuation of power from Tamakoshi V HEP through Singati substation can be ruled out.

2. There is no possibility of further expansion at Singati S/S due to space constraints as it is surrounded by hills on three sides and Singati River on one side. Refer to attached plot plan.

3. If a new 132 kV line is constructed from Tamakoshi V to Lamosangu, this power will need to be evacuated through Lamosangu - Bhaktapur 132 kV transmission line (double circuit BEAR conductor). The total load on this line will be 245 MW (Khimti P/S 60 MW, Bhotekoshi P/S 45 MW, power from Singati 140 MW), plus various other hydropower stations which are connected at Lamosangu S/S, even when Tamakoshi V is not connected. Hence there is no scope for any further addition to this load on Lamosangu - Bhaktapur 132 kV transmission line. Thus, this option also can be ruled out.

4. A new 400 kV transmission line from UTK to Barabise (near Lamosangu) and Barabise to Lapsiphedi (near Kathmandu) is under construction. Another option is to connect Tamakoshi V to Barabise with 132 kV double circuit Line. The length of this transmission line is estimated to be about 55 km.

   For this option, the following facilities need to be constructed:

a.	132 kV double circuit (BEAR conductor) line	55 km
b.	132 kV transmission line bay extension at Barabise	1 no.
c.	132/400 kV transformer bay at Barabise	1 no.

   This option is much more expensive as compared to the previous proposal, hence can be ruled out.

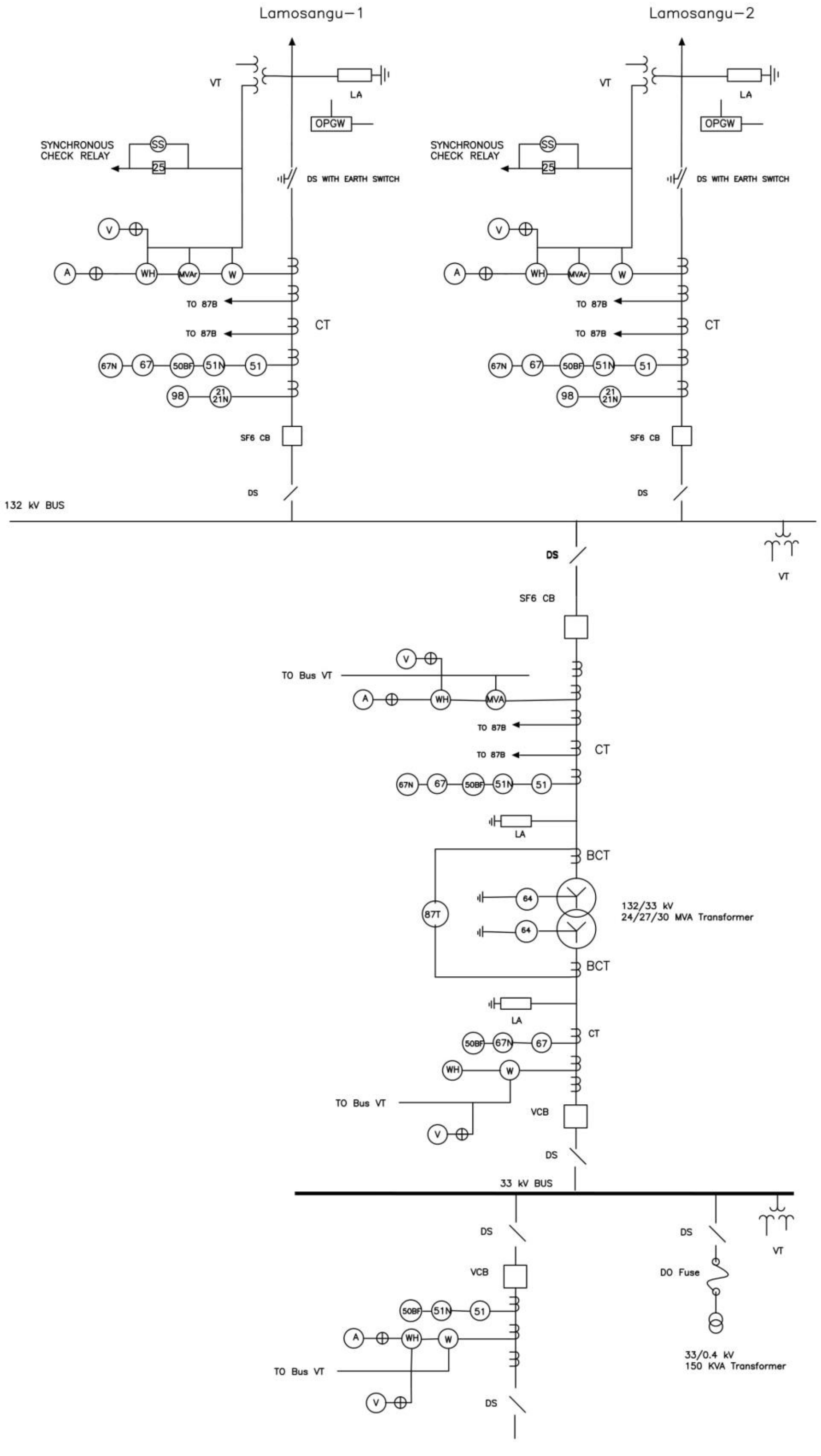

Figure 11-1: Single Line Diagram

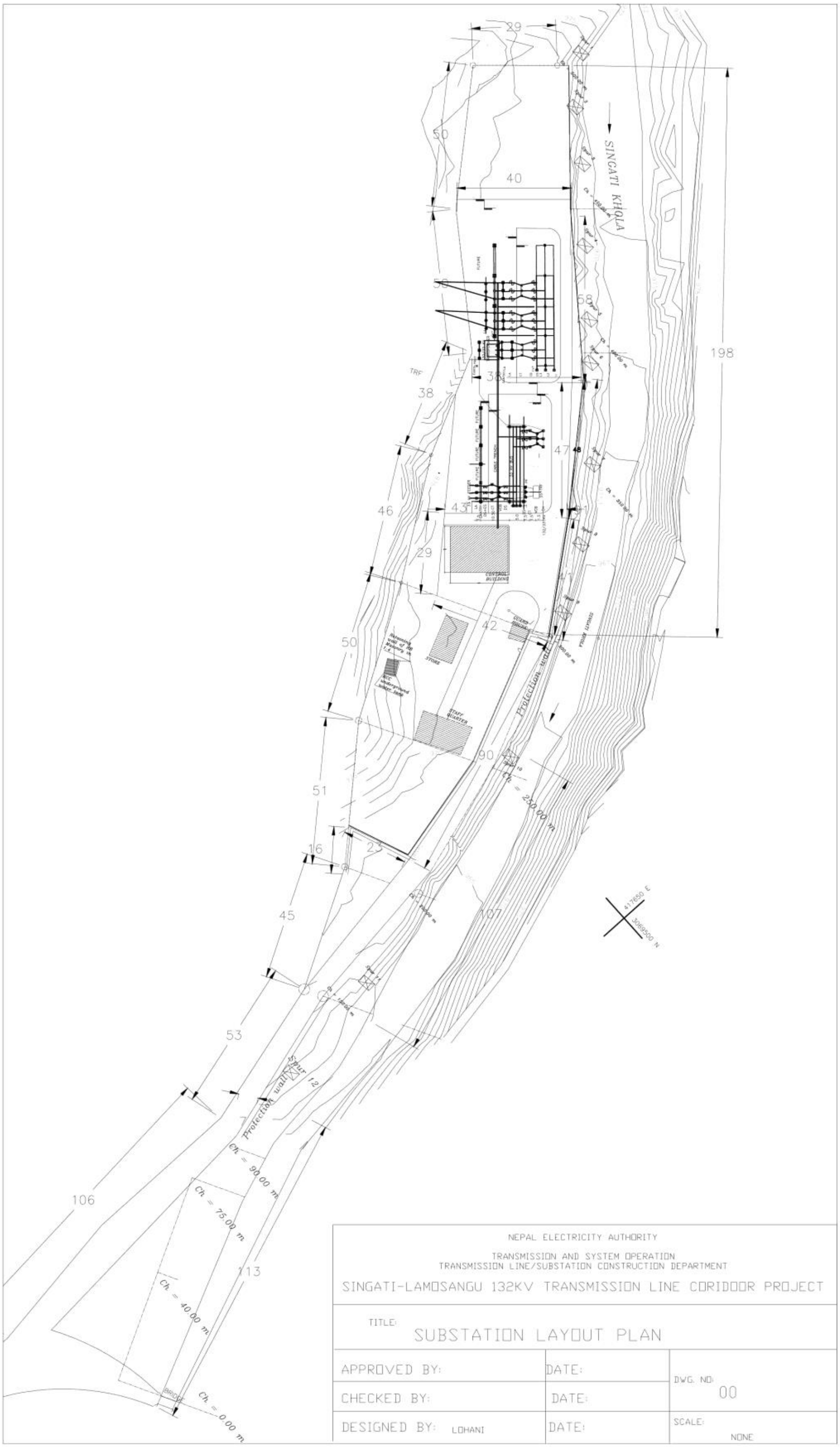

Figure 11-2: Layout Plan

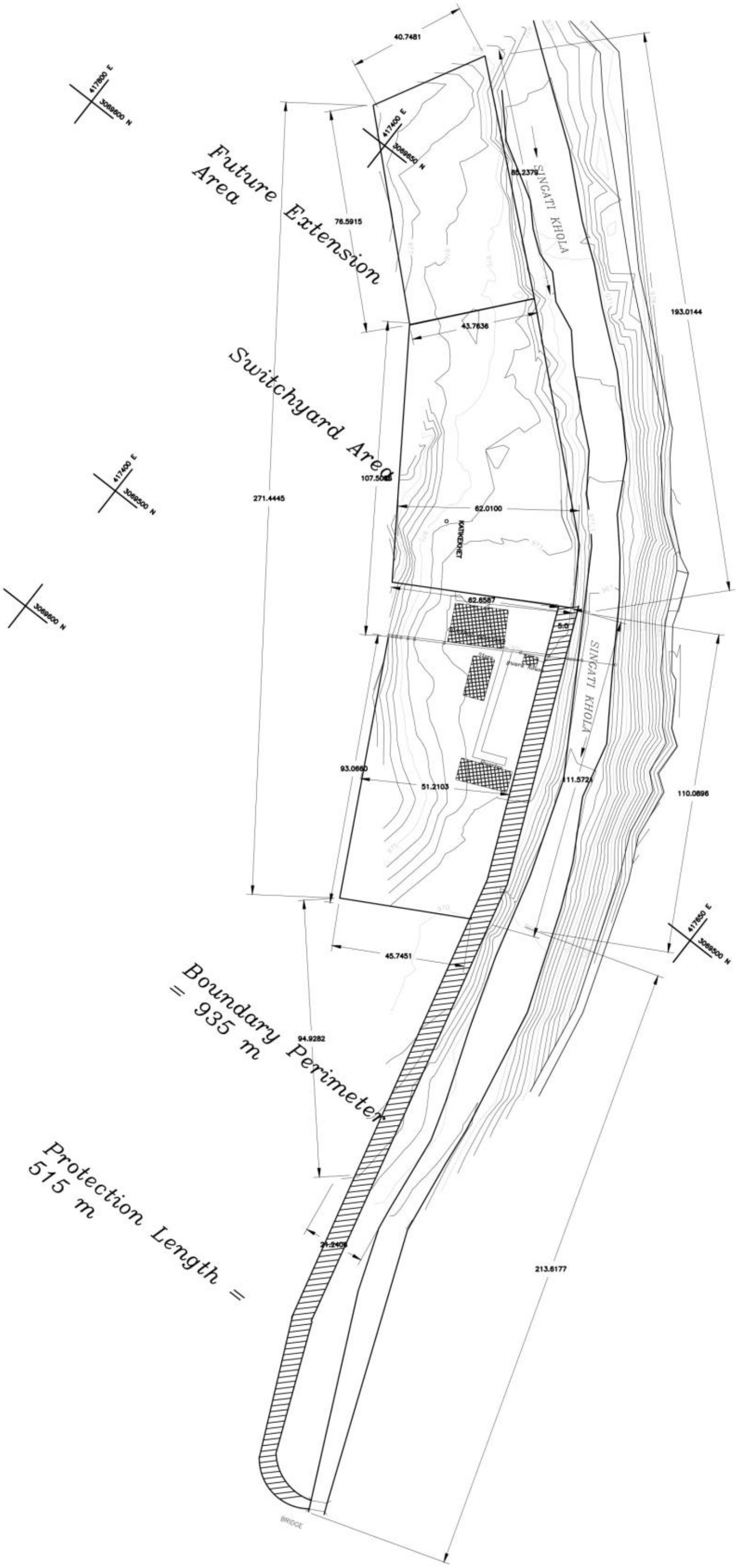

Figure 11-3: Layout Plan w/ Transmission Line Route Indication

12 EMPLOYER'S & ENGINEER'S CAMP AND ROADS

12.1 General Information

12.1.1 Specifics of the Camp Site

The permanent camp is located at Bigu Village Development Committee Ward Number 3 and is situated at right bank of Tamakoshi River and is adjacent to and north side of Jamune Bridge. The existing Singati – Lamabagar Public Road is connected with the camp at the right bank of Tamakoshi River. The area of the camp is about 3.5 hectares.

The site is in sloppy terrain and elongated in north-south direction. Camp consists of offices, living quarters of staffs, canteen, grocery store, ware house, Fire Brigade and site clinic in about 3.5 Ha. Road distance of the camp from the powerhouse is about 2.5 km. Public access is denied in camp with the help of checkpoint with Guard post and fencing.

There is also secondary access to the camp through an existing suspension bridge. As the site is elongated the buildings are placed in linear fashion. Due to small site area compact planning is done to adjust the given requirements. Greenery and open spaces are dually utilized as buffer space or barrier. Greenery and open spaces are maintained throughout the site so that it doesn't feel congested. The RL of buildings is maintained as per slope of site so that the amount of cut and fill in the site is minimal. The access road inside the camp acts as a kind of buffer between buildings and the Tamakoshi River.

The camp area is located on the cultivated terraces. The hill slope on the west side of the camp is a forest land which is composed geologically of thin ancient colluvial deposit and bed rock of Augen Gneiss. A geological report of the permanent camp is attached hereto.

Landscaping of the camp area with roads, buildings, street light, water supply & sewage system and greenery areas are provided. The camp is surrounded by security fencing with necessary gates.

12.1.2 Electric Power Supply to the Camp

During construction period electric power supply to the camp shall be brought from Singati through 11 kv line (4 km). And after completion of the project power supply shall be from Tamakoshi V powerhouse.

12.1.3 Water Supply and Sanitary System

A design period of 20 years is adopted for the demand calculation. Geometric progression approach for projections of design population with annual growth rate of 3% is adopted. Continuous water supply for 24 hours a day is adopted. A minimum of 5 m head is maintained at all service nodes.

The water supply to the camp is to be brought from Oran Khola and is situated at about 3 km from camp and lies on the right bank of Tamakoshi River. The scheme of water supply to the camp is to have right bank intake at Oran Khola and bring water through 3 km long main pipe line with necessary infrastructures for achieving drinkable water.

A RCC barrier is constructed across the Oran Khola so that the percolated water is collected through perforated small concrete channel buried upstream of the barrier. The water thus collected is passed through a water collection well & a sedimentation tank and is then collected in a 25 m3 capacity RCC reservoir tank. Through the reservoir tank, water is distributed to permanent camp under pressure by means of main pipe line. The main pipe line has a diameter of 90 mm and the material of the pipe is HDPE in colluvial deposit and GI in rock cliff areas. The collection well, sedimentation tank and reservoir tank are protected by masonry walls at the river bank. The flood water will not disturb the barrier as it is constructed below the river bed level.

The sewage coming out from buildings is passed through a 200 mm diameter RCC sewage pipe line and then to a RCC septic tank & soak pit for treatment. The slope of the sewage pipe is maintained 1: 100 to 1:20. Two sewage treatment plants containing septic tank (8 m3) and soak pit are provided to collect sewer coming out from two groups of buildings. The minimum velocity of sewer is kept at 0.75 m/s to maintain the self cleaning of the solid particles. Manhole of diameter 1 m is provided at suitable junctions to collect the sewage of buildings.

12.2 Design of Camp Buildings

Architectural Design

Buildings covered in this Document

The following is a list of buildings that were designed by the Architect.

1. Building Type A
2. Building Type B
3. Building Type C
4. Building Type D
5. Building Type E Guest House
6. Building Type F Employer’s Office
7. Building Type G Engineer’s Office
8. Building Type H Canteen Block
9. Building Type J Warehouse
10. Building Type K Guard House
11. Site Clinic
12. Grocery Store
13. Fire station
14. Guard Post

Specific Design Requirements

The architectural design related to space requirements was based on the Client’s specifications transmitted to the Architect through the document “Facilities of Camp”. The actual layout of the individual buildings. The design details were based on the reference book “Architectural Time Saver Standards, Technical Data for Professional Practice.” The design aesthetic is neo-classical; simple and clean. The finishing details were designed as per specification of the Owner.

The following input data was considered:

1. There are 14 types of building structures and a total 21 buildings inside the camp.
2. Total area of Permanent Camp including road length of 680 m is 3.5 hectares.
3. The water supply to the camp is supplied from Oran Khola. The intake in Oran Khola is about 1.7 km from the camp. The water diverted from the Khola passes through a sedimentation tank and collected in a reservoir tank. The water from the reservoir tank is chlorinated and then distributed to the camp with minimum water head of 10 m.
4. Waste water from all buildings is collected in two treatment plants through manholes and sewer pipe lines. The water coming out from the treatment plant is percolated beneath the ground near the bank of Tamakoshi River.
5. Electricity to camp will be tapped from NEA supply. Inside the camp electric poles with street light will be used for the distribution of electricity to each building.
6. The road inside the camp will be asphaltic road with street lighting system, surface drainage system and proper protection by retaining walls.
7. Because of small area available for the camp, helipad is situated near the powerhouse area which is 2.5 km. from the camp area.
8. The landscaping of the camp is as shown in the layout drawing. For this, hill slope cutting in 1(vertical): 2(horizontal) slope in alluvial soil and 10(vertical): 1(horizontal) slope in rock cut is assumed. The rock cut slope shall be protected by rock bolts and fibre shotcrete. And the cut slope in alluvial soil shall be protected by proper turfing.
9. Greenery area shall be allocated as indicated in the layout plan of the camp.
10. An existing suspension bridge is situated near the cliff. At the location a guard post is placed at the main entrance to the camp area. On the south side that goes to site clinic is kept open for public.
11. Fencing around the camp area and Helipad area is to be done with necessary entry gates

The synthesis of Owner’s requirements and the resulting architectural design is presented in the Drawing Album, Part E of the Detailed Design Report.

Structural Design

The following part of this Subchapter deals with the structural analysis and design of different buildings proposed for the project. There are fourteen different types of buildings in the Permanent Camp.

The building structures were designed on the basis of the dimensions and occupancy planned according to the Detailed Design Drawings. Among the 14 building types planned, the Canteen, Guard Post and Warehouse are reinforced concrete framed structures with steel roof trusses with and without RCC frame support structures. The rest of the buildings are RCC frame structures with slabs at each floor level and topped with a roof.

In the following sections the buildings are presented with their architectural designs (layouts, elevations) and FEM models, as applicable. Details about the dimensions and the assumptions for, and results of the calculation are not included here; they can readily be taken from Part A3 Chapter 10 of the Detailed Design Report and Part E Album of Drawings.

12.2.1 Building Type A Project Managers' Quarters

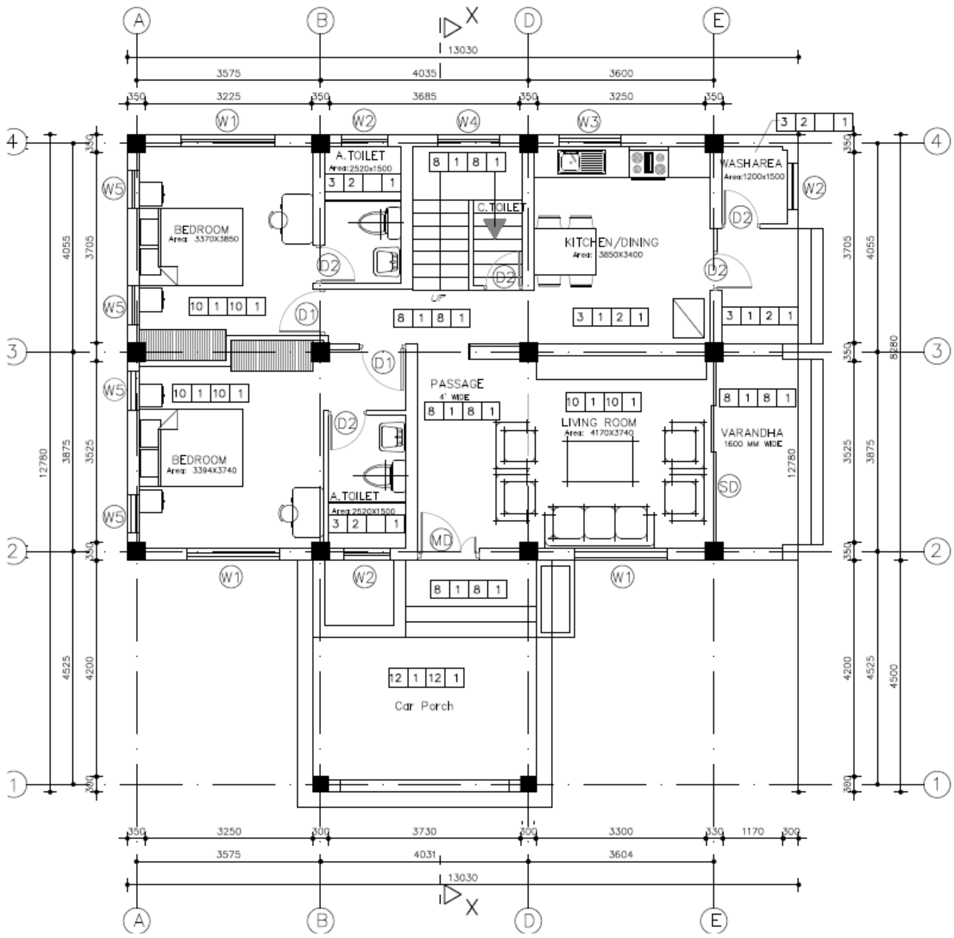

Figure 12-1: Plan of Type A Building

Figure 12-2: Front Elevation of Type A Building

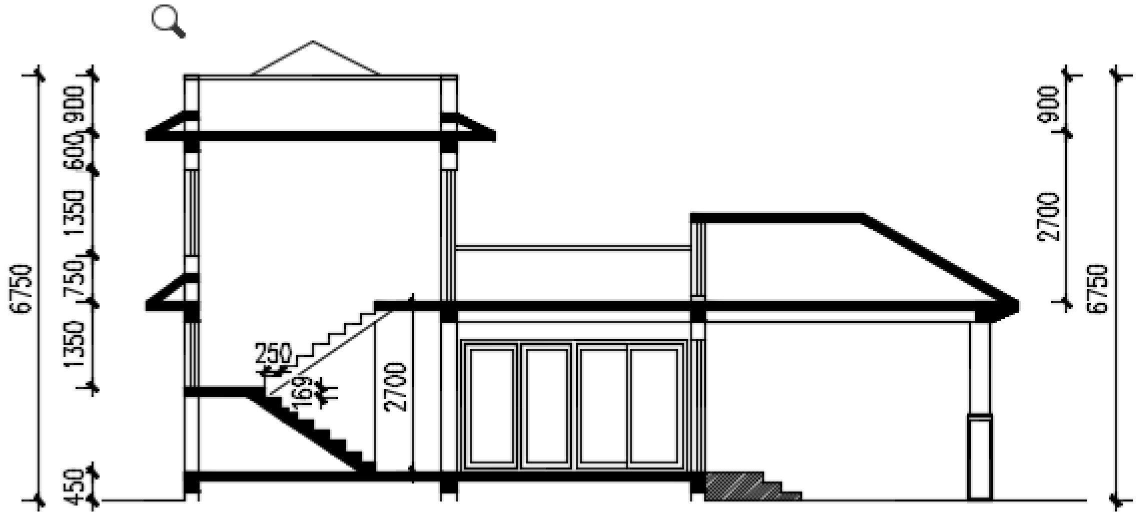

Figure 12-3: Section of Type A Building

12.2.1.1 Specific Design Criteria

Input Data for Design

Materials

- Grade of concrete for RCC frame is taken as M25 or C20/25;
- Unit Weight of Reinforced Concrete = 25 KN/m^3;
- Modulus of Elasticity of Concrete Ec = 5000 $\sqrt{f_{ck}}$ = 25000 N/mm^2; and

- Grade of steel is Fe 500 (according to IS 1786-2008).
- Structural steel E250BR (according to IS 2062-2011)

Assumptions and preliminary dimensions

The following basis were made for preliminary dimension of structural elements,

- Size of columns were determined on the basis of preliminary calculations of dead and live loads
- Size of beams were determined on the basis of effective span
- Slab dimensions were taken from the critical panel with highest occupancy loads and maximum span
- Bearing capacity of soil for ultimate limit state design was taken as 300 KN/m^2.

Design Loads

The following loads have been considered for the design of the buildings:

- Self-weight of the structure (DL);

  Dead load includes the self-weight of frame structure, wall and partition loads and loads due to plaster and floor finish works
- Live load or occupancy load (LL)

  Live load includes all load as per planned use of the building
- Earthquake load (EQx, EQy, EQz)

  Includes the loads acting on the building due to earthquake based on its dead load and occupancy
- Wind load

  Load acting on building due to design wind

Load Combinations

The analysis was performed for the following load combinations and the structural members were designed using Limit State Method of Design to ensure an adequate degree of safety and serviceability. The structure was designed based on the most critical limit state and was checked for other limit states of serviceability. (Refer to table 18 IS456-2000 and Cl. 8.3.2 IS 1893 (Part4) 2015).

Limit State of Collapse

Combination 1: 1.5 DL: 1.5 (Dead + Live)

Combination 2: 1.2 DLE: 1.2(Dead + Live ± $EQ_{x, y, \text{ or } z}$)

Combination 3: 1.5 DE: 1.5 (Dead ± $EQ_{x, y, \text{ or } z}$)

Combination 4: 0.9 DE: 1.5 Dead ± 0.9 $EQ_{x, y, \text{ or } z}$

Limit State of Serviceability

Combination 5:1.0 (Dead + Live)

Combination 6:1.0*(Dead ± $EQ_{x,y\text{ or }z}$)

Combination 7:1.0*(Dead + 0.8 Live + 0.8 $EQ_{x,y\text{ or }z}$)

When wind was considered as severe load earthquake load on the load combination can equivalently replace the wind load case.

12.2.1.2 Design Method Applied

Analysis

The following procedure was adopted for analysis:

- Dead load of frame structures were self-generated by FEM model
- Live load was assigned to the FEM model as per IS 875 part 2.
- Appropriate finishing loads (load of screed, plaster, and tiling) were calculated and assigned to the FEM model (see Part A3 Chapter 10 of this report).
- The wall load with plastering is calculated and assigned to model
- SAP software was used to carry out the analysis of frame structure;
- Analysis was carried out for all the loads and combinations as specified above.
- Calculation of horizontal seismic coefficients

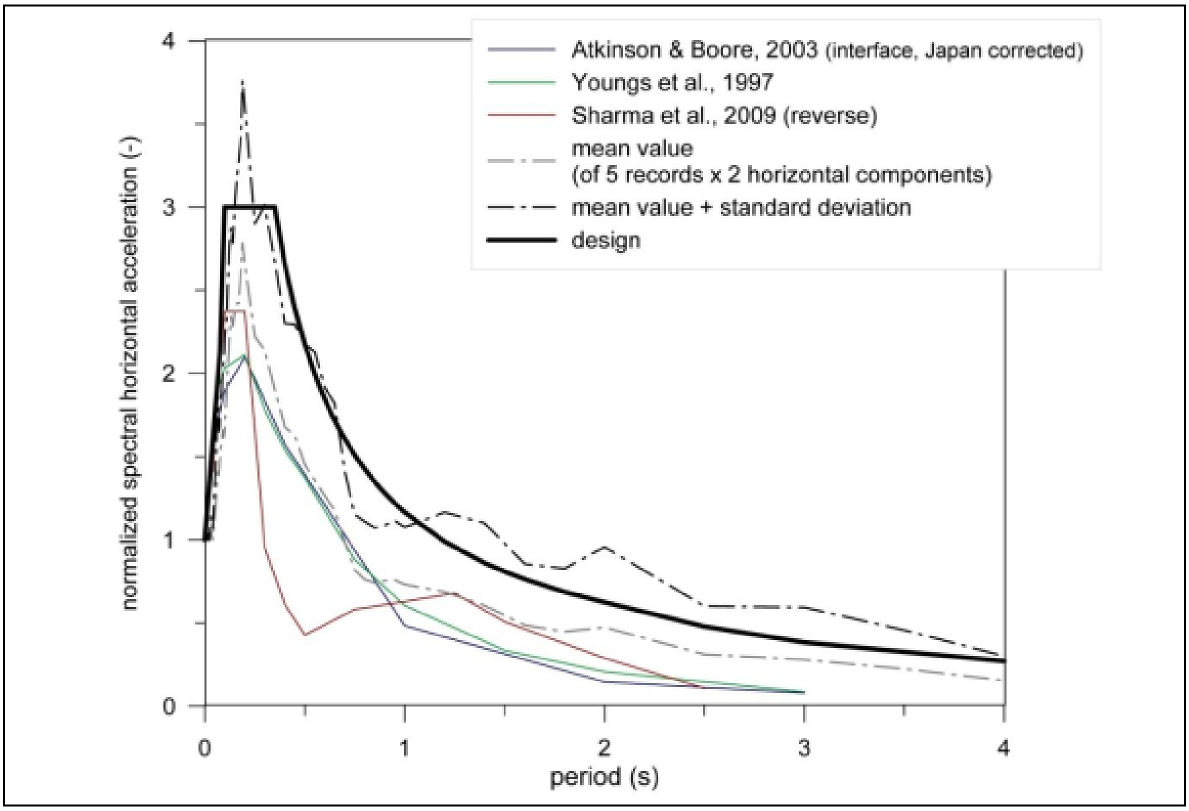

Figure 12-4: Design Response Spectrum Curve as per Seismological Investigation

$$A_h = \frac{Z}{2}\frac{S_a}{g}\frac{I}{R}$$

Z = 0.36

S_a/g = 3.0 (as per Figure 12-4**:** Design Response Spectrum Curve as per Seismological Investigation

)

I = 1.0

R = 5

A_h= 0.108

V_h= 0.9*A_h

- The basic time period was calculated using modal analysis with FEM software.
- The number of modes were considered such that the sum of model mass was greater than 90 percent of total seismic mass, as per IS 1893, 2016 part 1.
- Wind load was calculated on the basis of IS 875 part 3,
- The design was based on lateral loads due to earthquake since the building is located in an active seismic zone.
- Limit state of serviceability was checked and designed for limit state of collapse.

Modeling of Structure in FEM Software

3-D model of building was prepared on FEM software the basic data for analysis were adopted as discussed above.

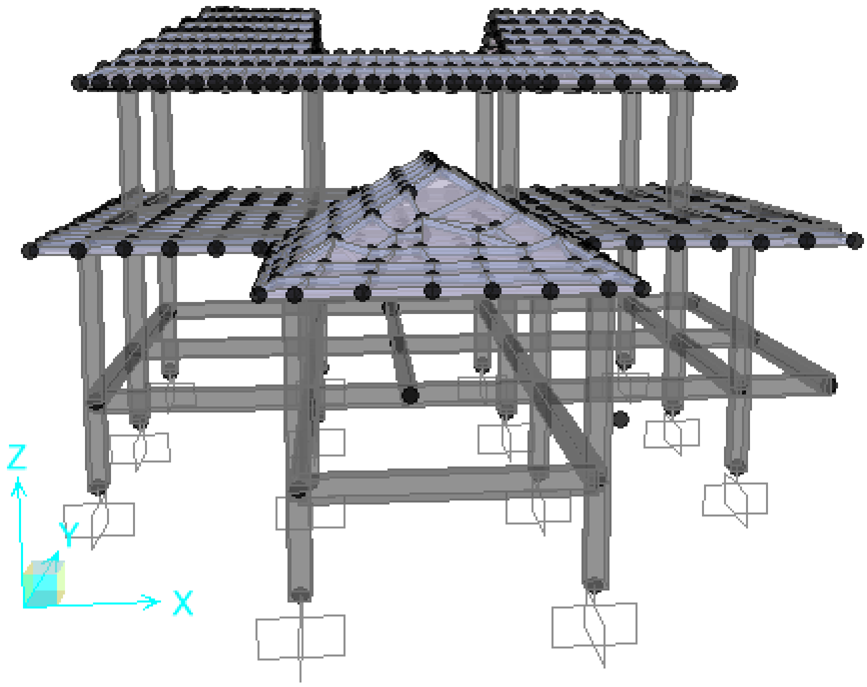

Figure 12-5: FEM Model of Type A Building

12.2.2 Building Type B Senior Engineers' Quarters

The basic data and dimensions were taken from the Detailed Design Drawings.

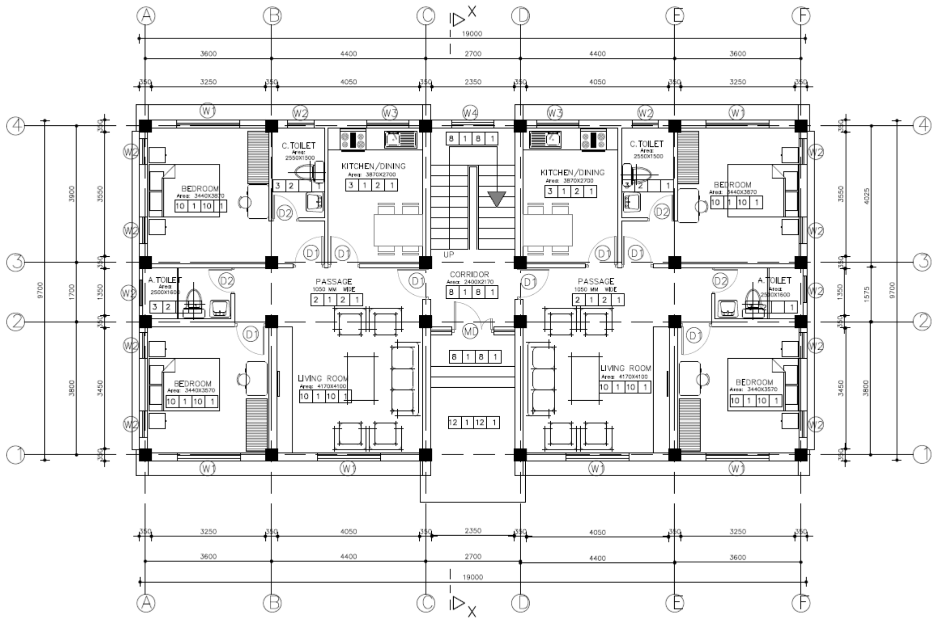

Figure 12-6: Plan of Type B Building

Figure 12-7: Front Elevation of Type B Building

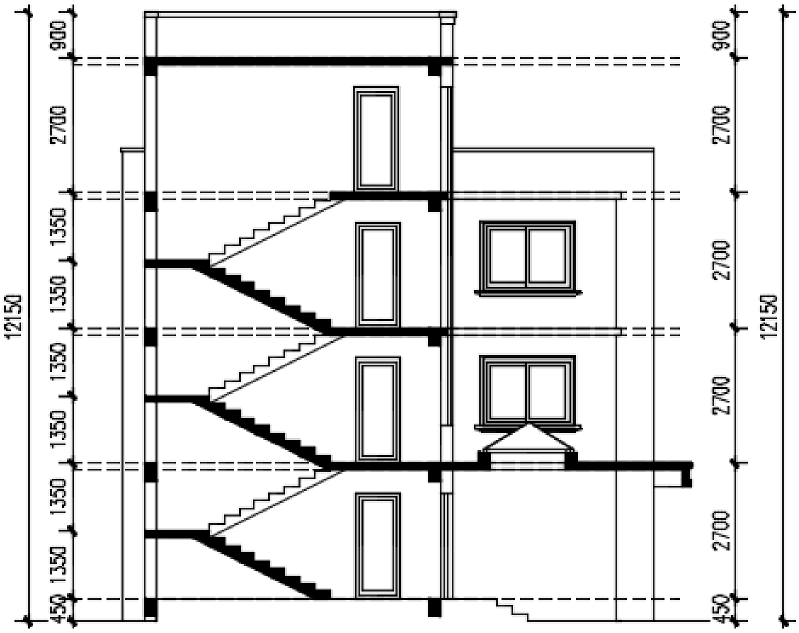

Figure 12-8: Section of Type B Building

12.2.2.1 Specific Design Criterial

Please refer to 10.2.1.1

12.2.2.2 Design Method Applied

<u>Modeling of Structure in FEM Software</u>

3-D model of building was prepared with FEM software. The basic data for the analysis were adopted as discussed above.

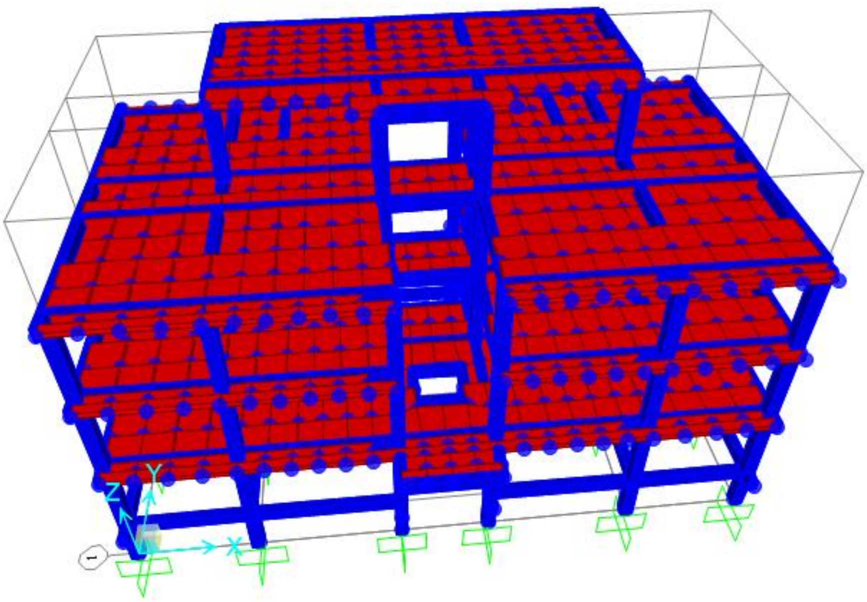

Figure 12-9: FEM Model of Type B Building

12.2.3 Building Type C Engineers & Officers Quarters

The basic data and dimension were taken from Detailed Design Drawings.

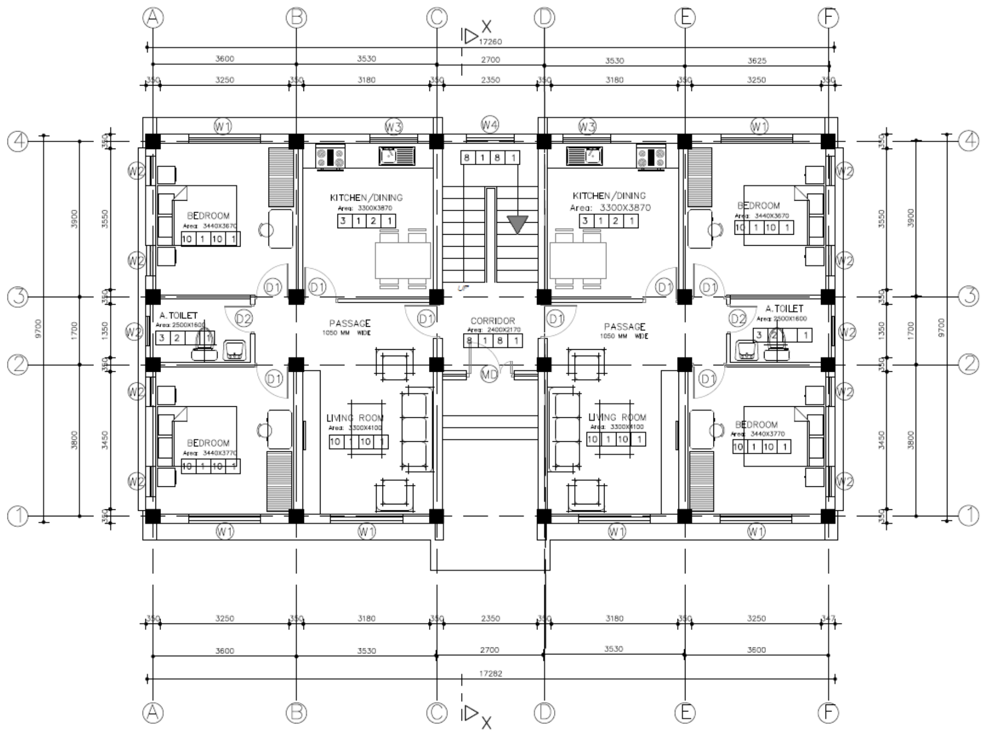

Figure 12-10: Plan of Type C Building

Figure 12-11: Front Elevation of Type C Building

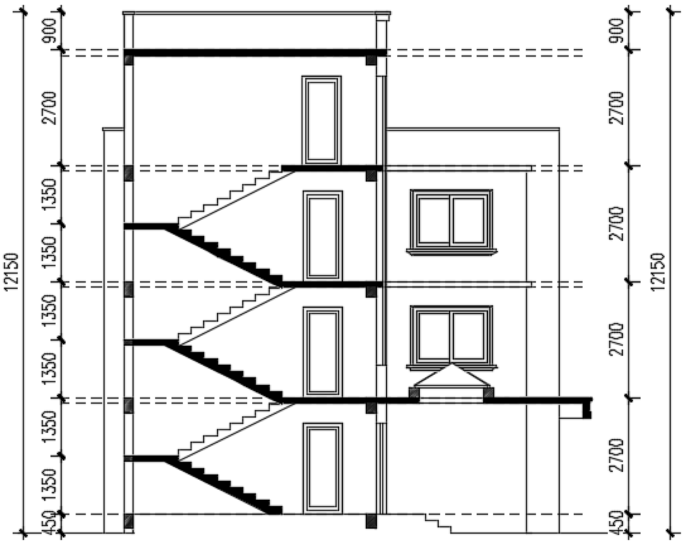

Figure 12-12: Section of Type C Building

12.2.3.1 Specific Design Criterial

Please refer to 10.2.1.1

12.2.3.2 Design Method Applied

Modeling of Structure in FEM Software

3-D model of building was prepared with FEM software the basic data for analysis were adopted as discussed above.

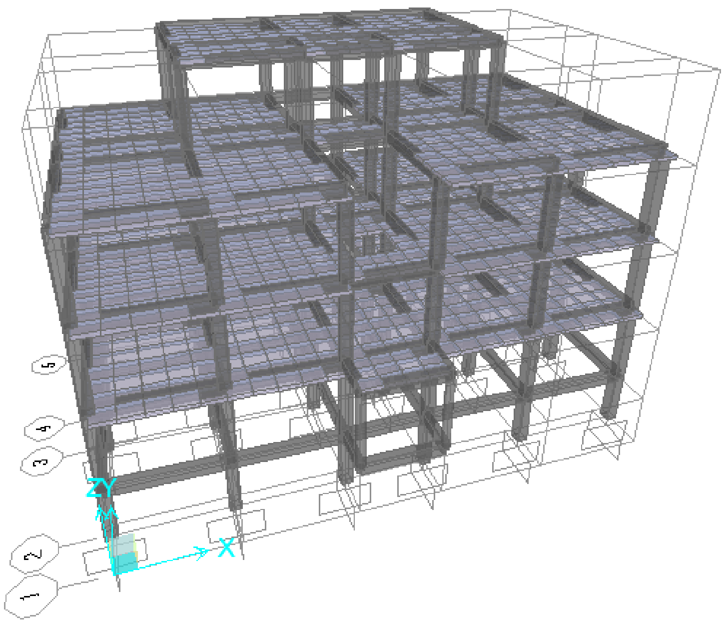

Figure 12-13: FEM Model of Type C Building

12.2.4 Building Type D Supervisor Quarters

The basic data and dimension were taken from Detailed Design Drawings.

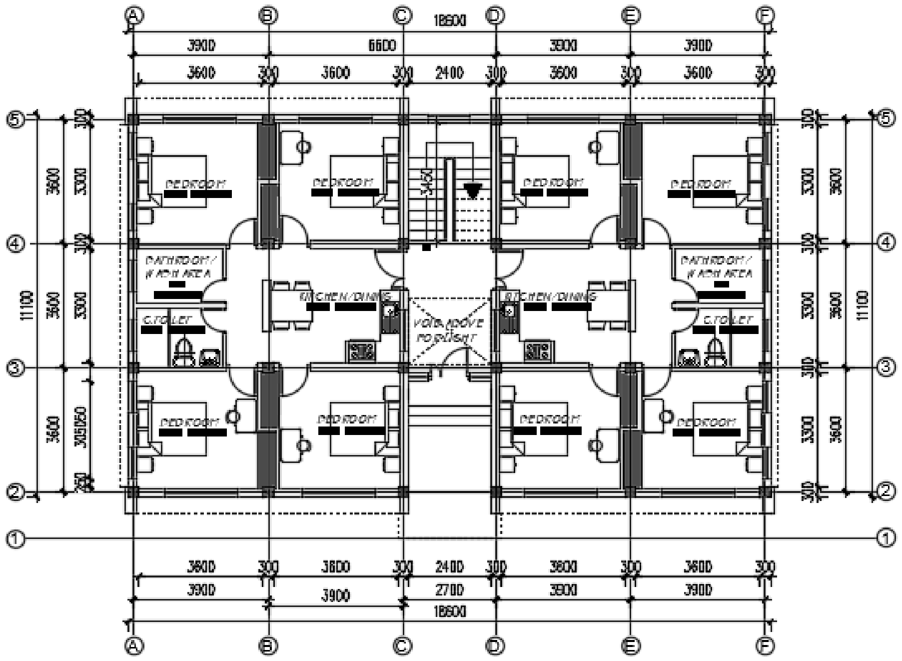

Figure 12-14: Plan of Type D Building

Figure 12-15: Front Elevation of Type D Building

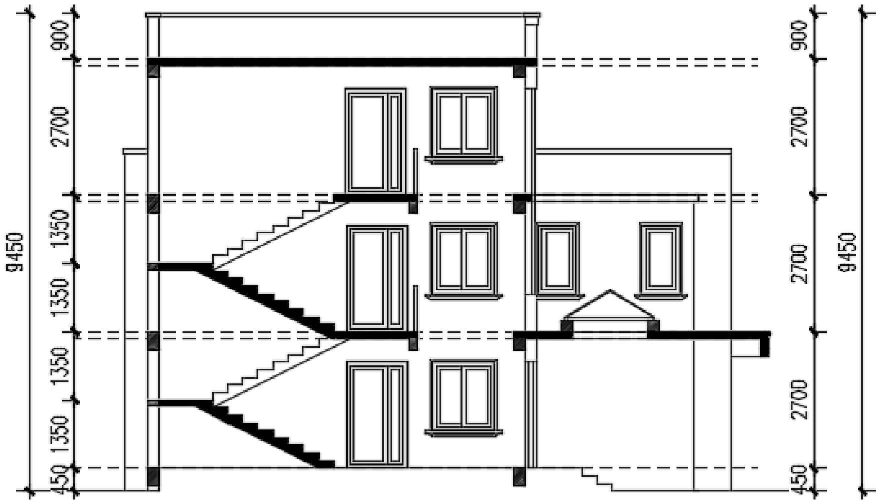

Figure 12-16: Section of Type D Building

12.2.4.1 Specific Design Criterial

Please refer to 10.2.1.1

12.2.4.2 Design Method Applied

<u>Modeling of Structure in FEM Software</u>

3-D model of building is prepared on FEM software the basic data for analysis were adopted as discussed above.

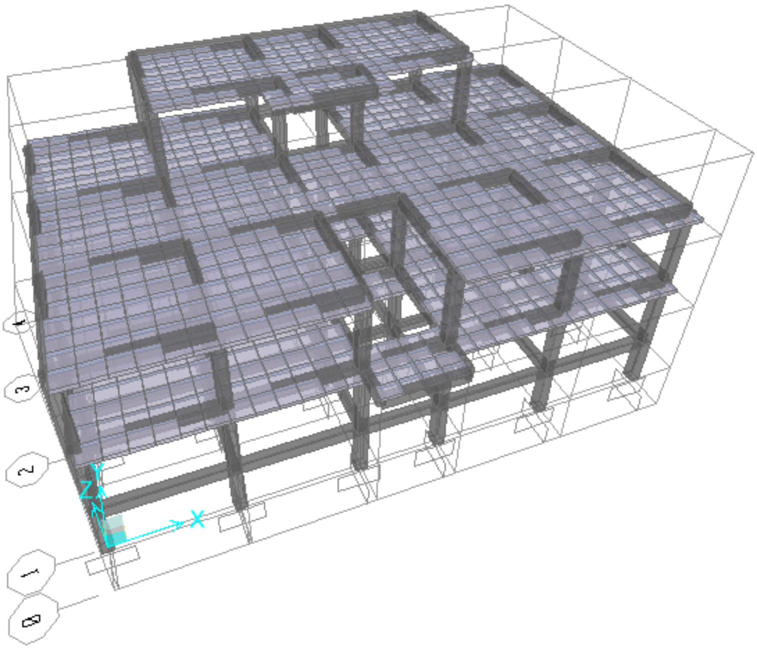

Figure 12-17: FEM Model of Type D Building

12.2.5 Building Type E Guest House

The basic data and dimension were taken from architectural planning.

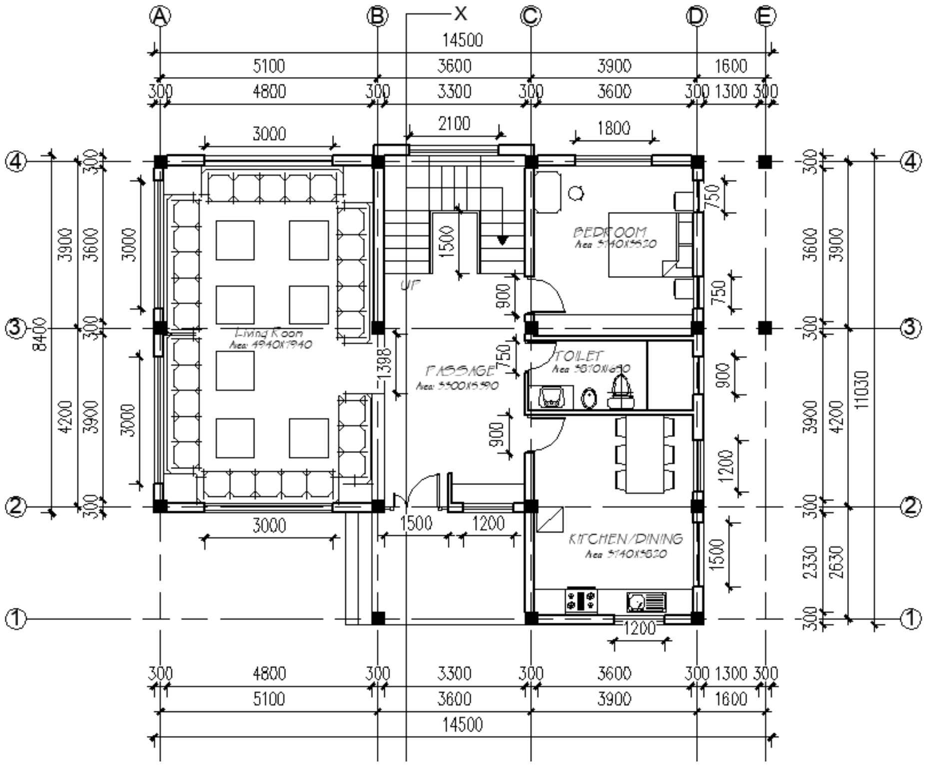

Figure 12-18: Plan of Type E Guest House

Figure 12-19: Front Elevation of Type E Guest House

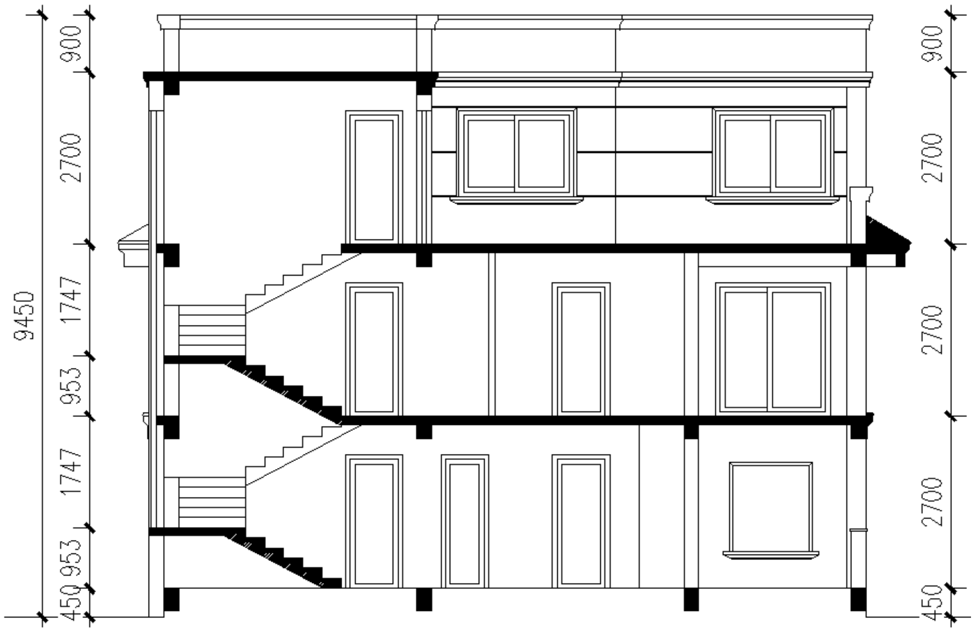

Figure 12-20: Section of Type E Guest House

12.2.5.1 Specific Design Criterial

Please refer to 10.2.1.1

12.2.5.2 Design Method Applied

Modeling of Structure in FEM Software

3-D model of building is prepared on FEM software the basic data for analysis were adopted as discussed above.

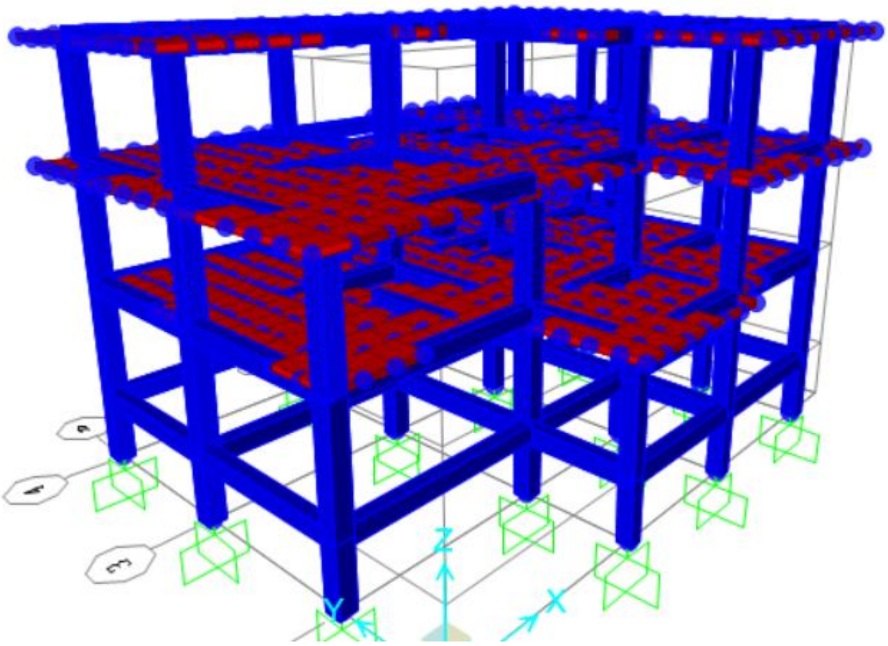

Figure 12-21: FEM Model of Type E Guest House

12.2.6 Building Type F Guard House

The basic data and dimension were taken from Detailed Design Drawings.

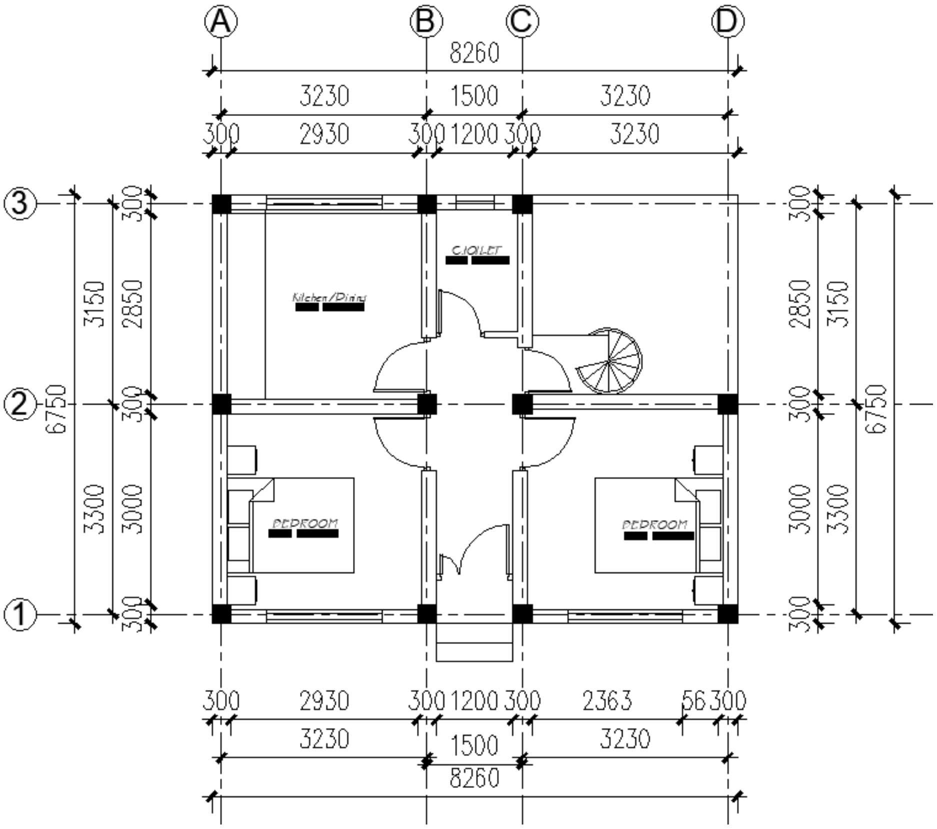

Figure 12-22: Plan of Type F Guard House

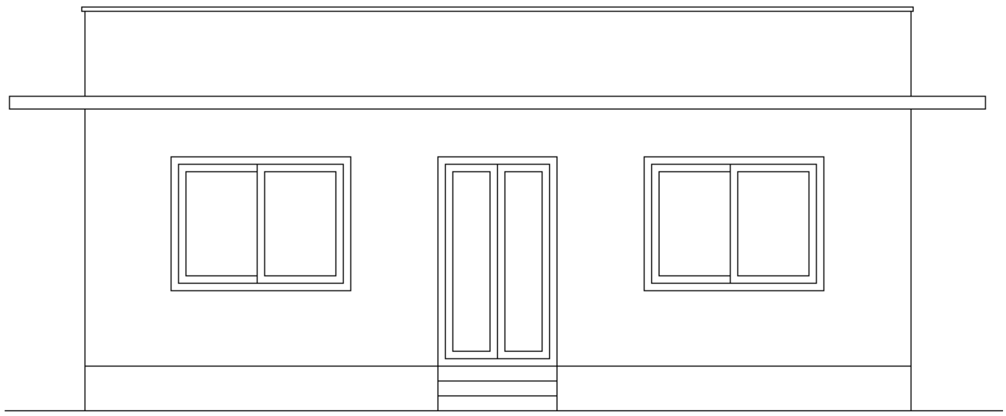

Figure 12-23: Front Elevation of Type F Guard House

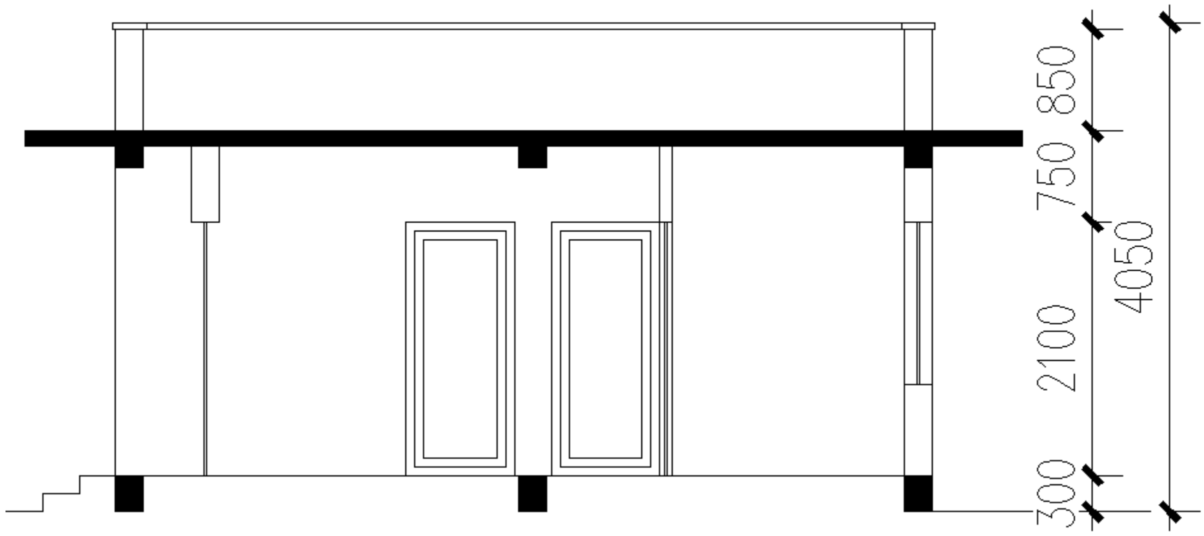

Figure 12-24: Section of Type F Guard House

12.2.6.1 Specific Design Criterial

Please refer to 10.2.1.1

12.2.6.2 Design Method Applied

<u>Modeling of Structure in FEM Software</u>

3-D model of building is prepared on FEM software the basic data for analysis were adopted as discussed above.

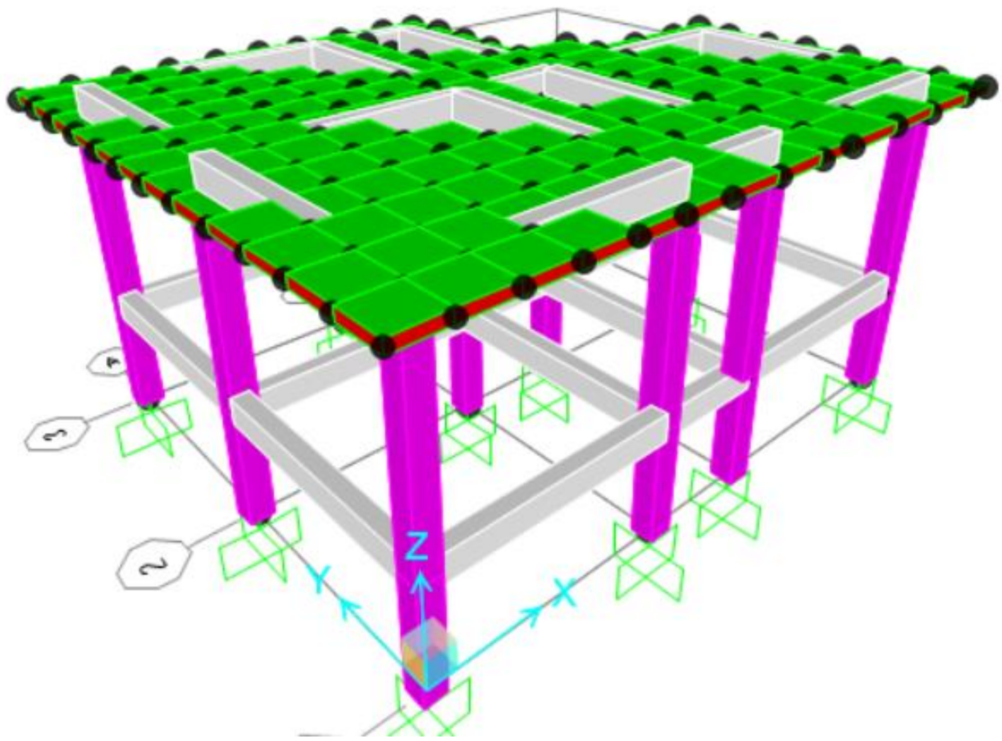

Figure 12-25: FEM Model of Type F Guard House

12.2.7 Building Type G (Employer's Office)

The basic data and dimension were taken from Detailed Design Drawings.

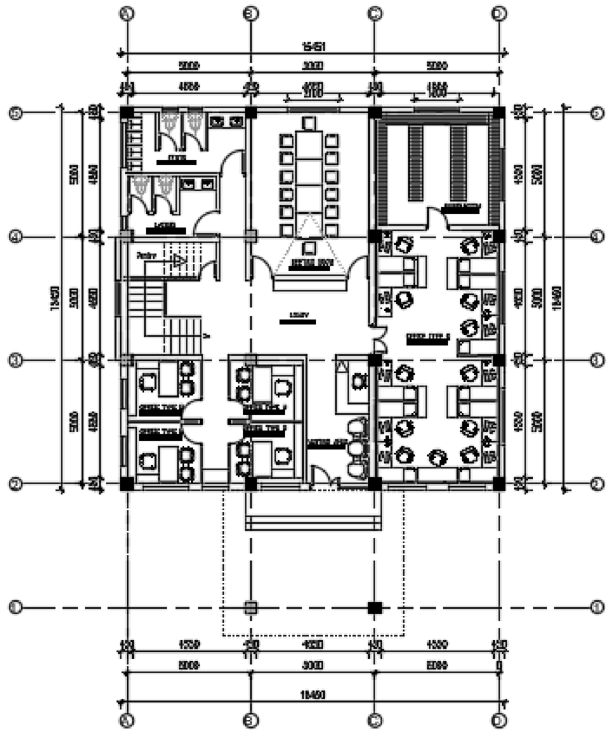

Figure 12-26: Plan of Employers Office (Type G)

Figure 12-27: Front Elevation of Employers Office (Type G)

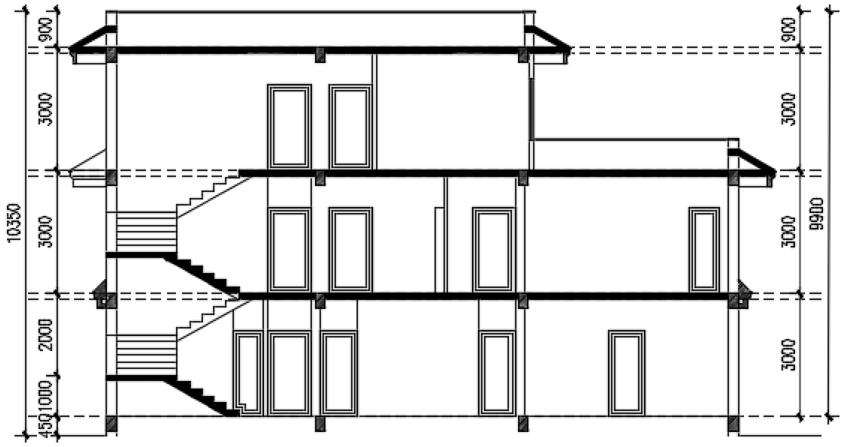

Figure 12-28: Section of Employers Office (Type G)

12.2.7.1 Specific Design Criterial

Please refer to 10.2.1.1

12.2.7.2 Design Method Applied

Modeling of Structure in FEM Software

3-D model of building is prepared on FEM software the basic data for analysis were adopted as discussed above.

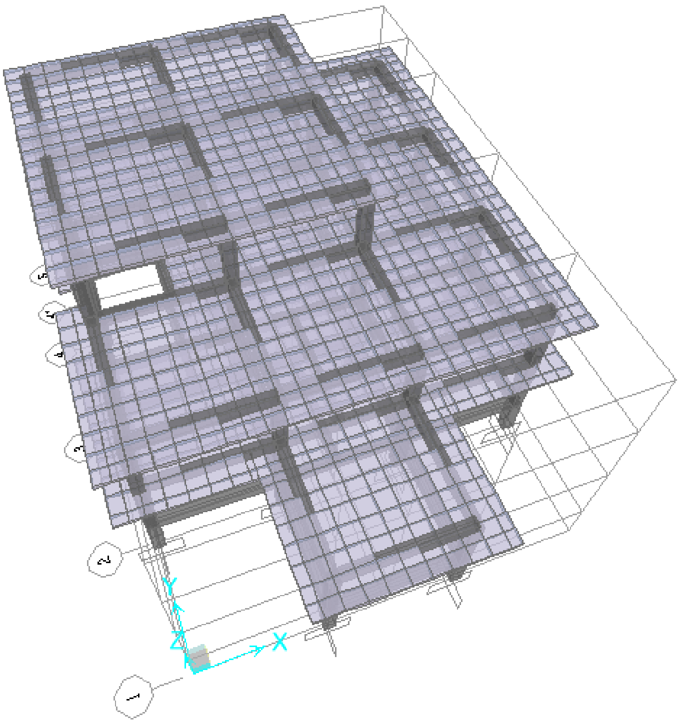

Figure 12-29: FEM Model of Employers Office (Type G)

12.2.8 Building Type H Engineers Office

The basic data and dimension were taken from Detailed Design Drawings.

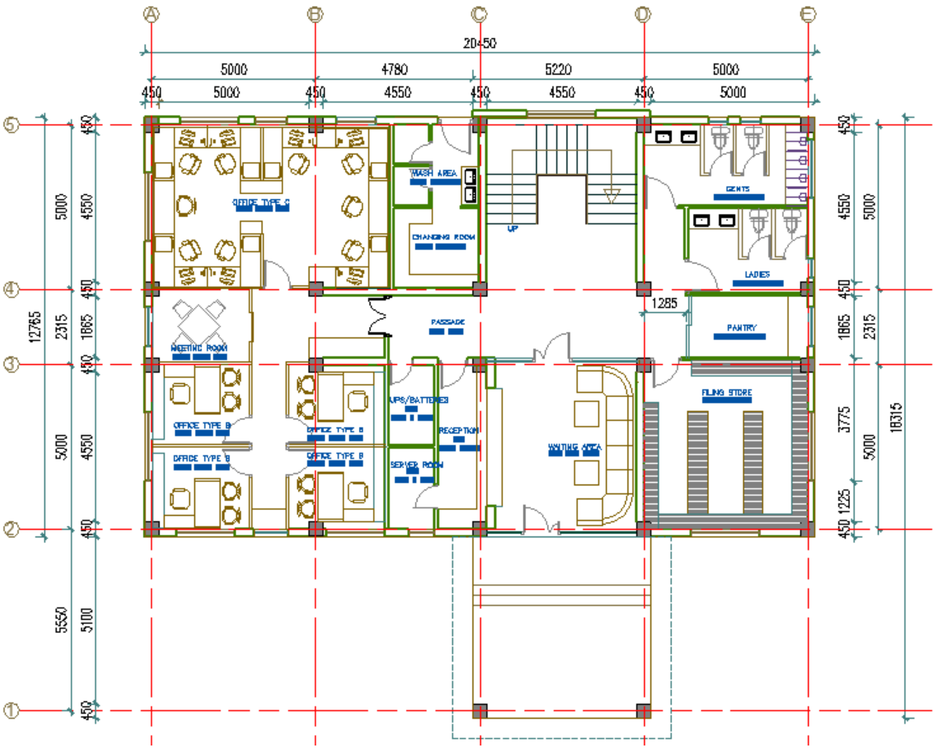

Figure 12-30: Plan of Type H Engineers Office

Figure 12-31: Front Elevation of Type H Engineers Office

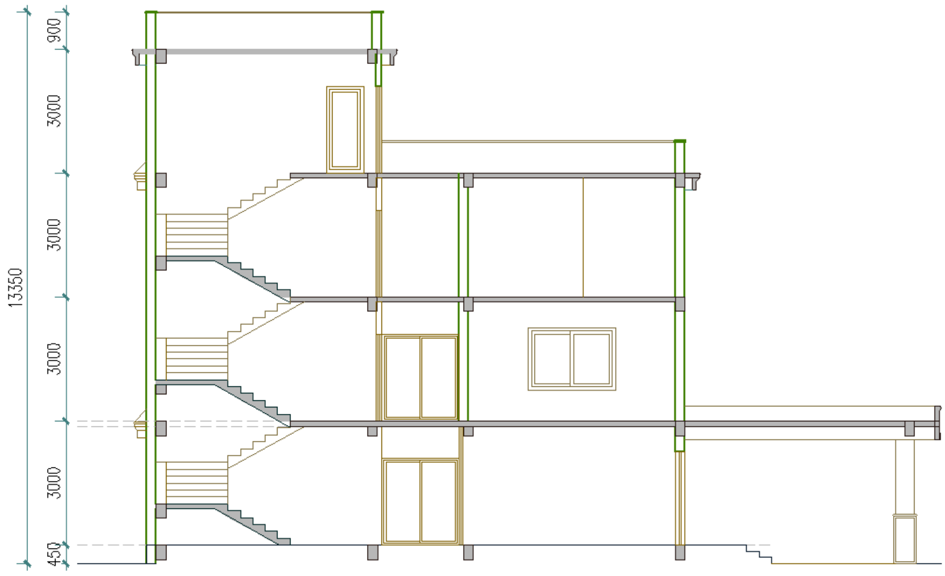

Figure 12-32: Section of Type H Engineers Office

12.2.8.1 Specific Design Criterial

Please refer to 10.2.1.1

12.2.8.2 Design Method Applied

Modeling of Structure in FEM Software

3D model of building is prepared on FEM software the basic data for analysis were adopted as discussed above.

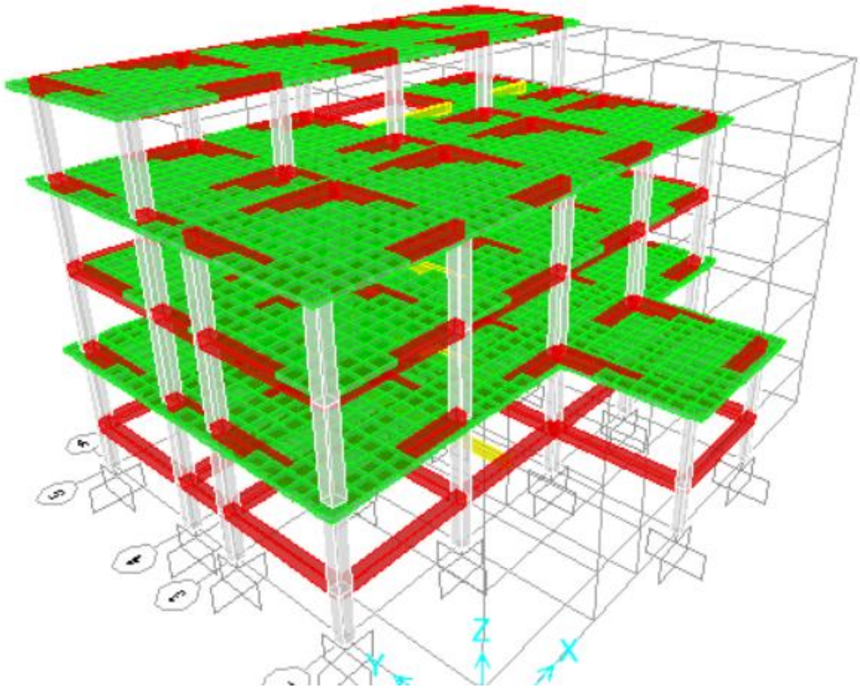

Figure 12-33: FEM Model of Type H Engineers Office

12.2.9 Building Type I Canteen Block

The basic data and dimension were taken from Detailed Design Drawings.

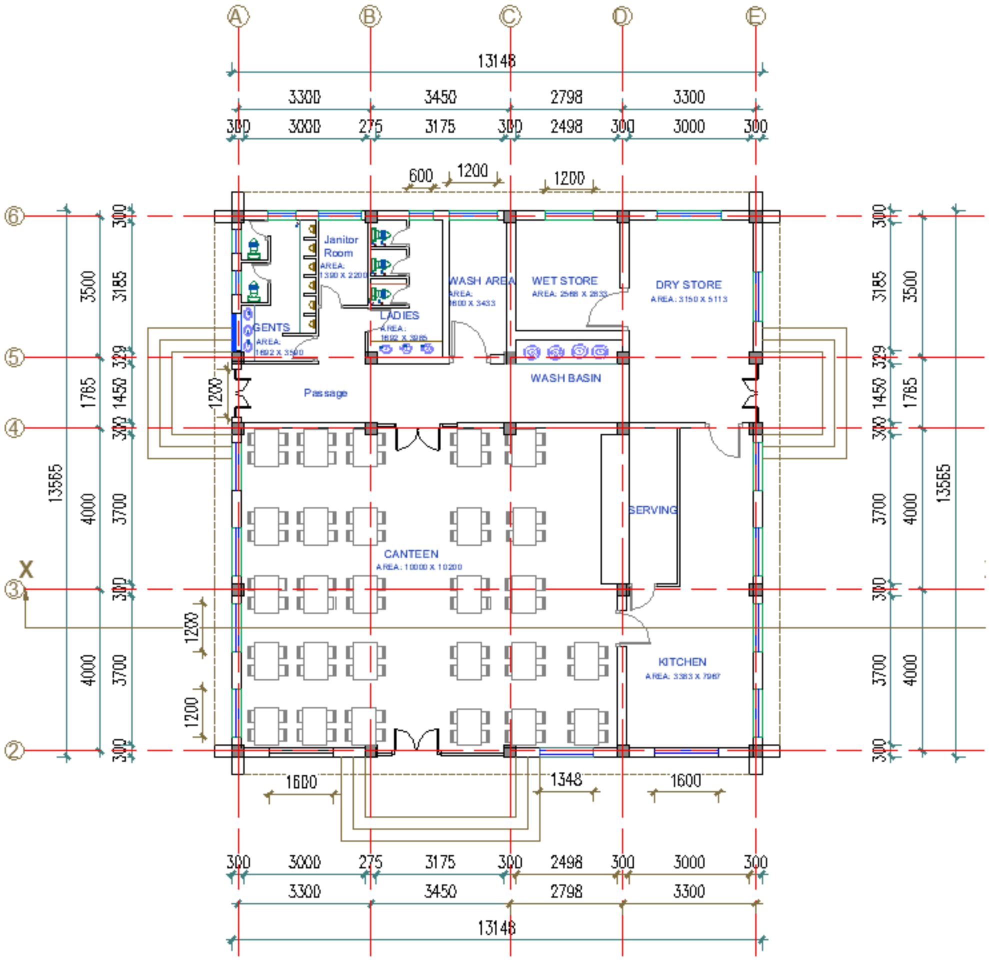

Figure 12-34: Plan of Building Type I Canteen Block

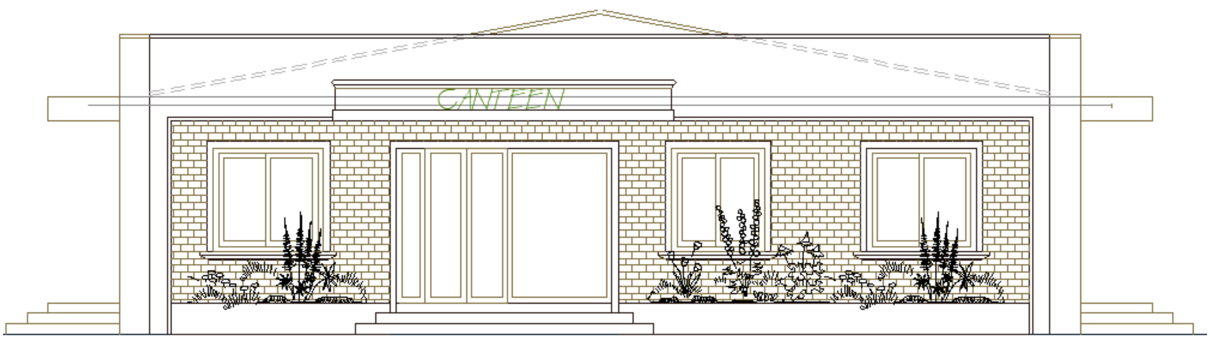

Figure 12-35: Front Elevation of Building Type I Canteen Block

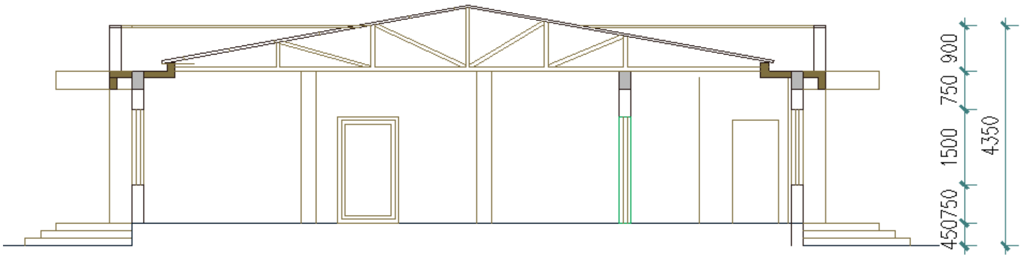

Figure 12-36: Section of Building Type I Canteen Block

12.2.9.1 Specific Design Criterial

Please refer to 10.2.1.1

12.2.9.2 Design Method Applied

Modeling of Structure in FEM Software

3-D model of building is prepared on FEM software the basic data for analysis were adopted as discussed above.

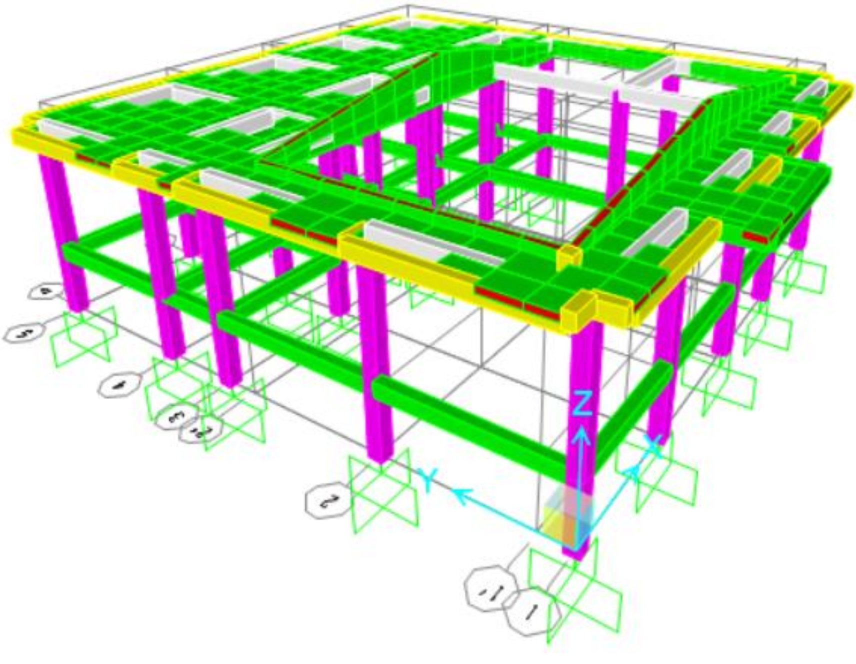

Figure 12-37: FEM Model of Building Type I Canteen Block

12.2.10 Building Type J Electrical Building

The basic data and dimension were taken from the Detailed Design Drawings.

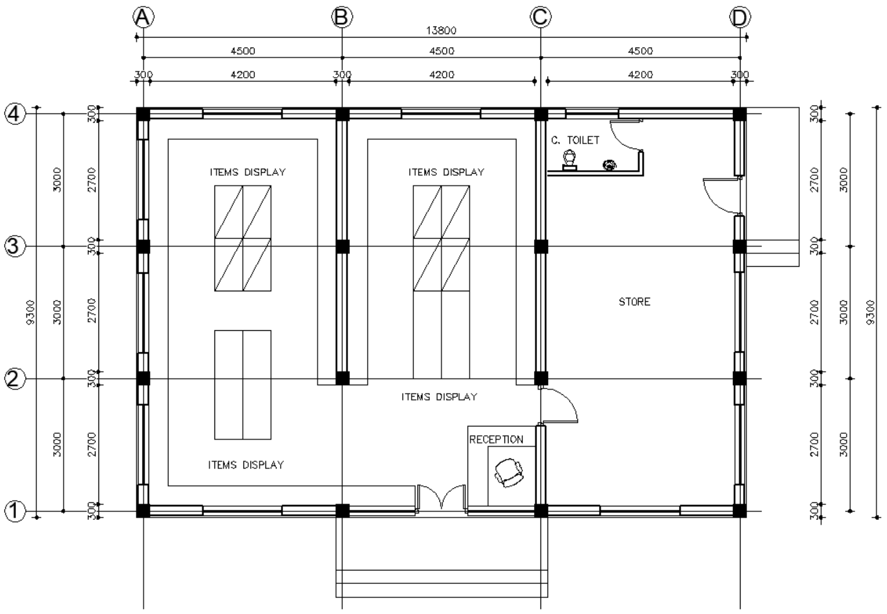

Figure 12-38: Plan of Electrical Building

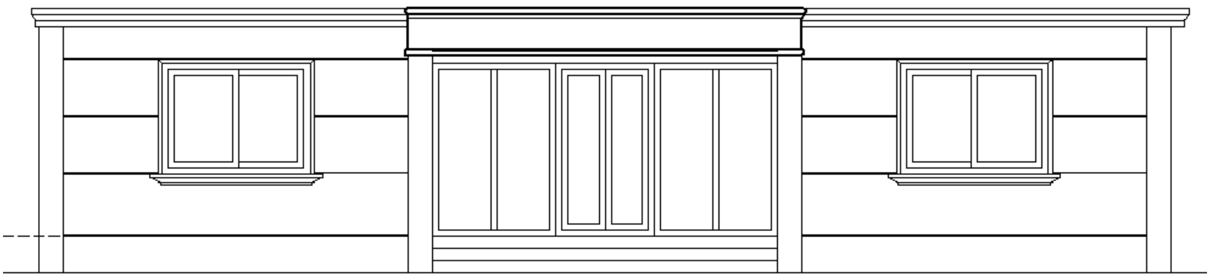

Figure 12-39: Front Elevation of Electrical Building

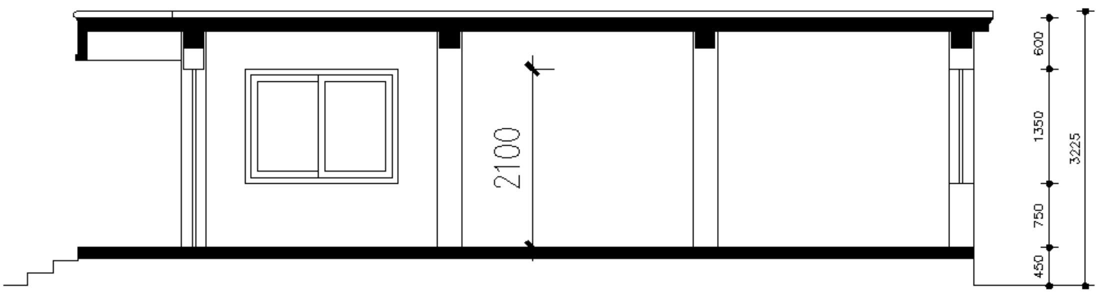

Figure 12-40: Section of Electrical Building

12.2.10.1 Specific Design Criterial

Please refer to 10.2.1.1

12.2.10.2 Design Method Applied

Modeling of Structure in FEM Software

3-D model of building is prepared on FEM software the basic data for analysis were adopted as discussed above.

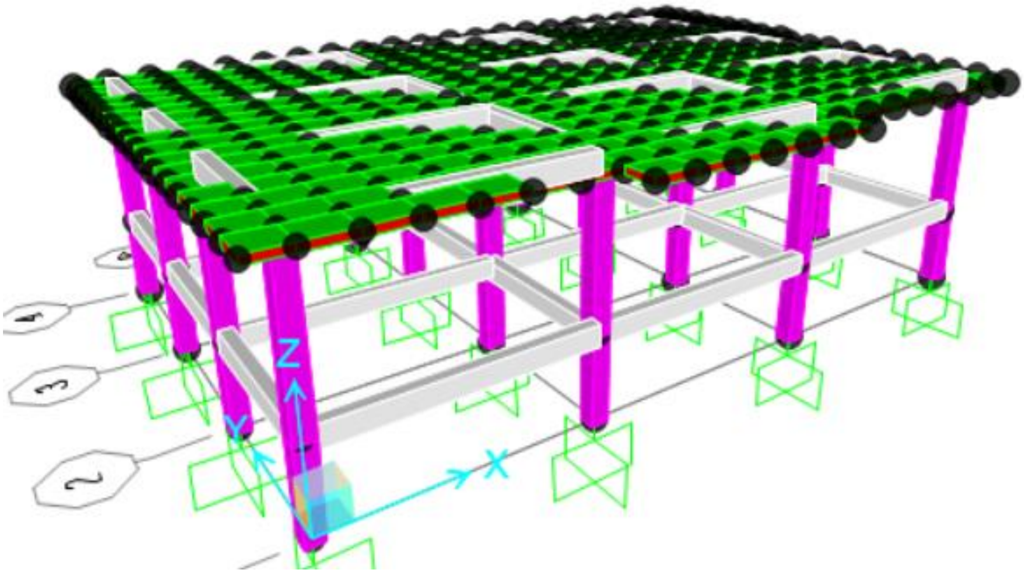

Figure 12-41: FEM Model of Electrical Building

12.2.11 Building Type K Warehouse

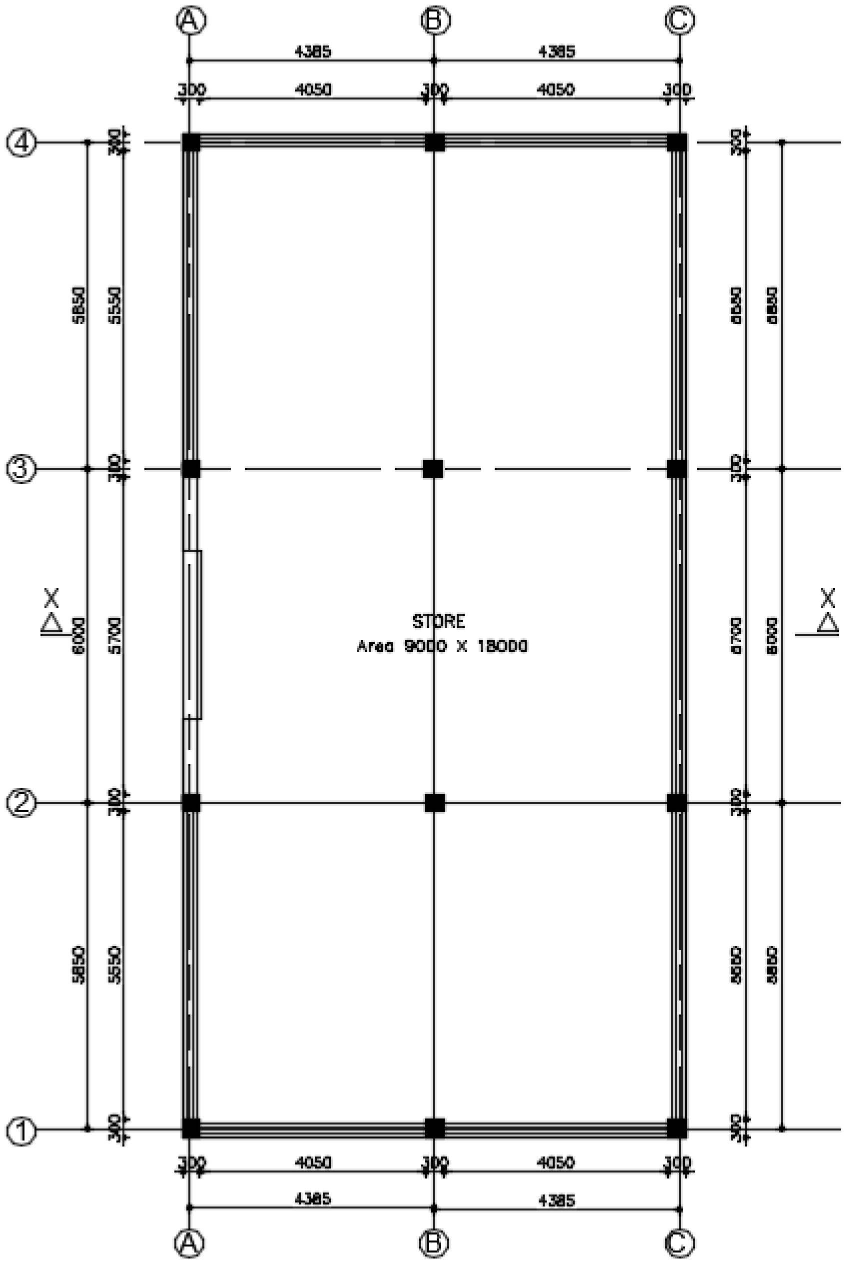

Figure 12-42: Plan of Type K Warehouse

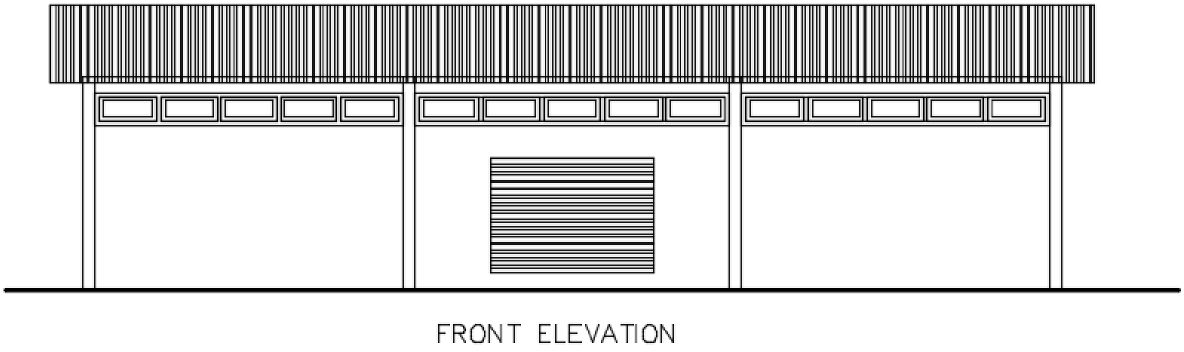

Figure 12-43: Front Elevation of Type K Warehouse

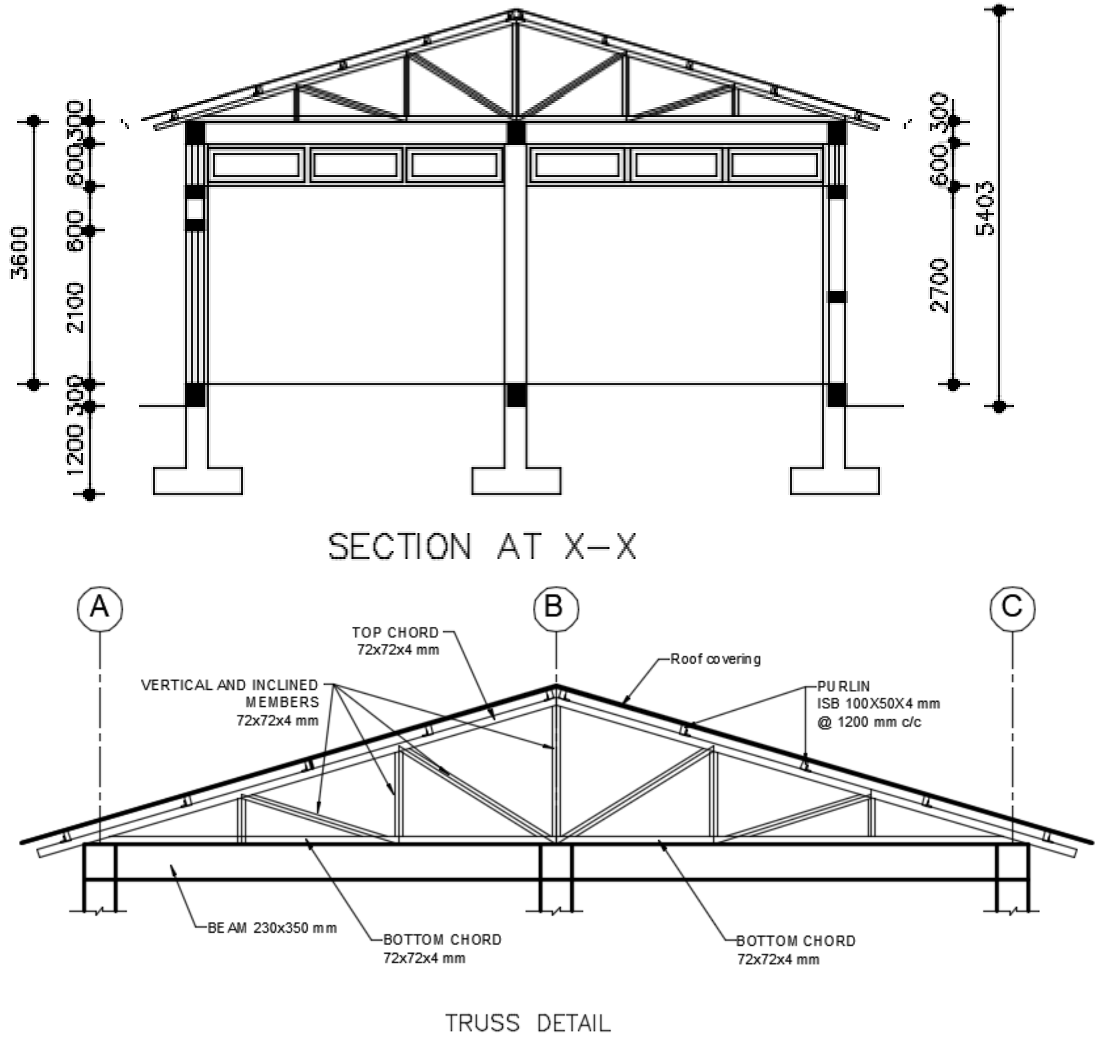

Figure 12-44: Section of Type K Warehouse

12.2.11.1 Specific Design Criterial

Please refer to 10.2.1.1

12.2.11.2 Design Method Applied

Modeling of Structure in FEM Software

3-D model of building was prepared with FEM software the basic data for analysis were adopted as discussed above.

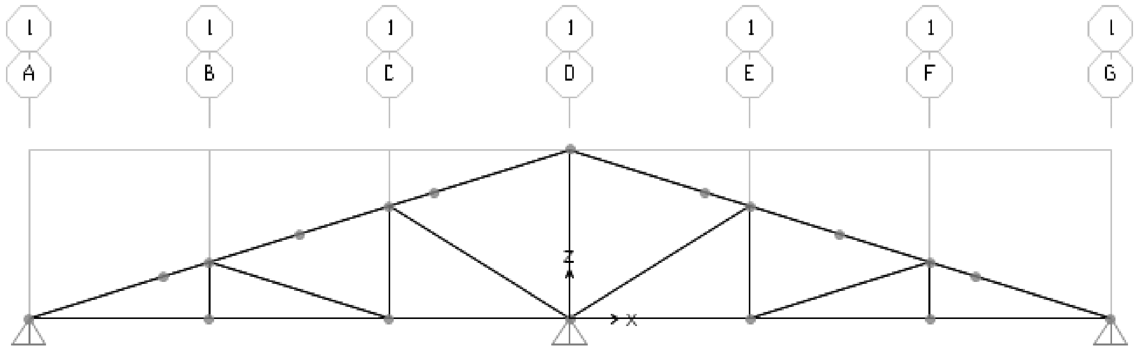

Figure 12-45: FEM Model of Type K Warehouse

12.2.12 Building Type L Fire Station

The basic data and dimension were taken from Detailed Design Drawings.

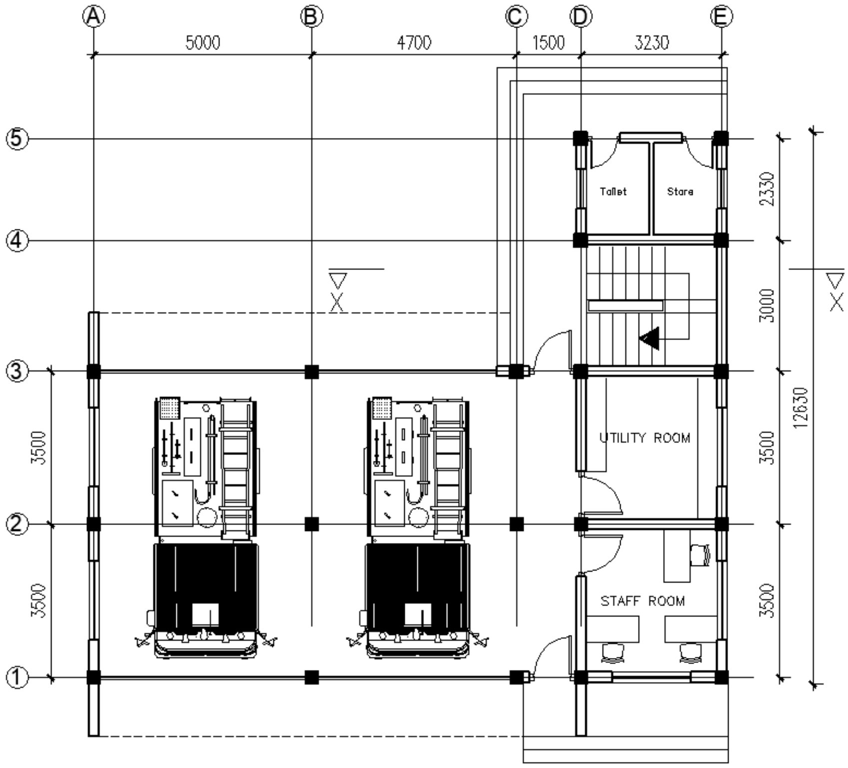

Figure 12-46: Plan of Fire Station

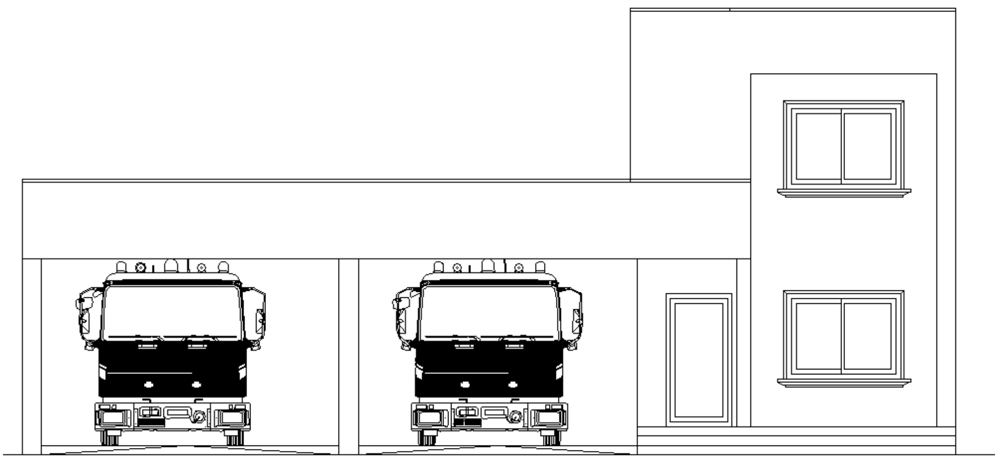

Figure 12-47: Front Elevation of Fire Station

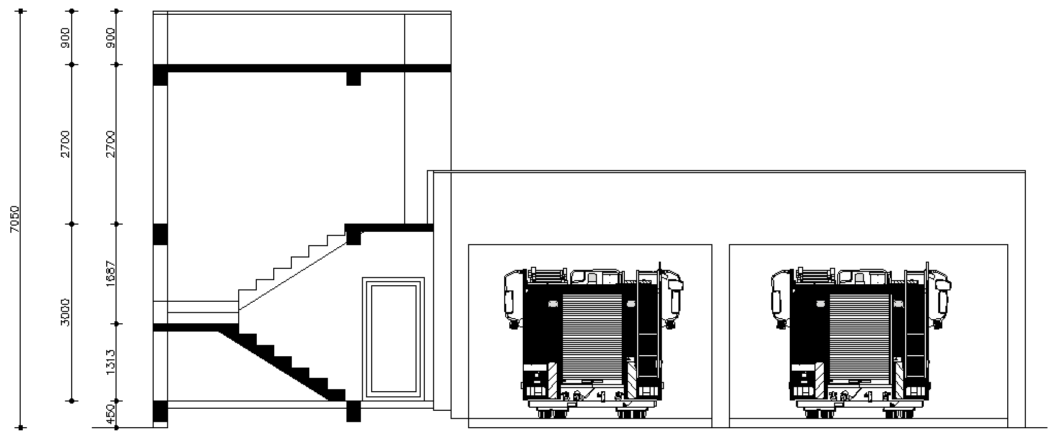

Figure 12-48: Section of Fire Station

12.2.12.1 Specific Design Criterial

Please refer to 10.2.1.1

12.2.12.2 Design Method Applied

Modeling of Structure in FEM Software

3-D model of building is prepared on FEM software the basic data for analysis were adopted as discussed above.

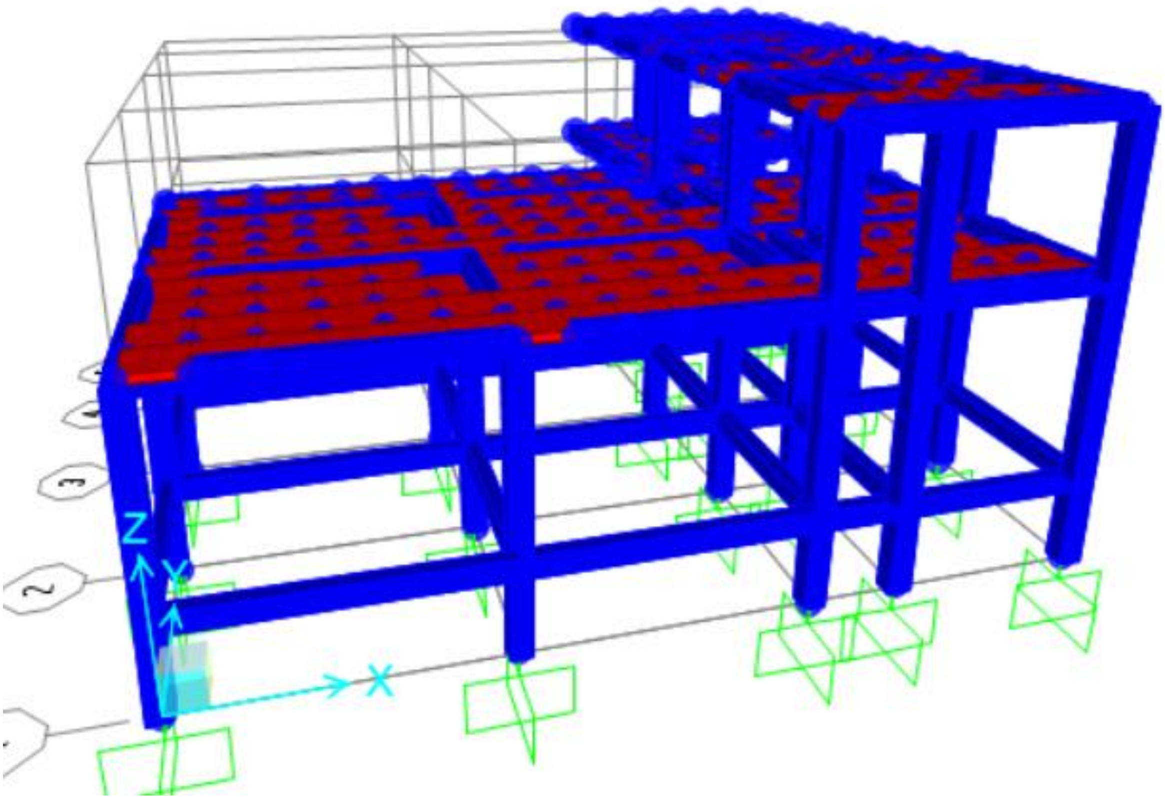

Figure 12-49: FEM Model of Fire Station

12.2.13 Building Type M Site Clinic

The basic data and dimension were taken from Detailed Design Drawings.

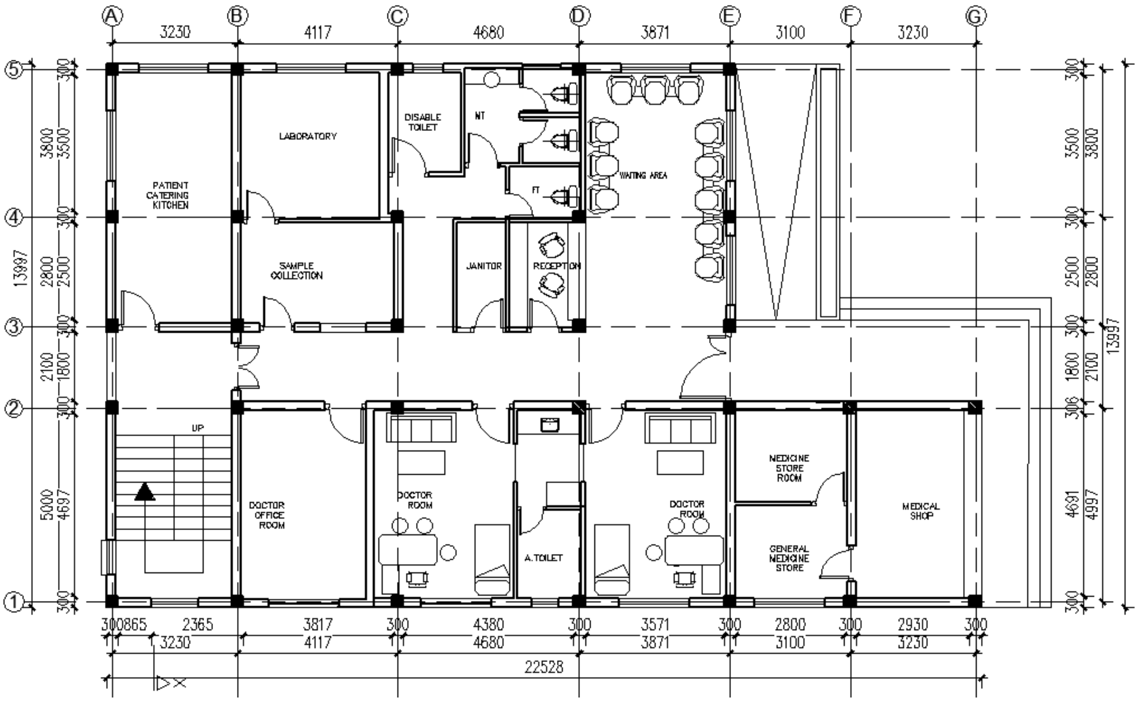

Figure 12-50: Plan of Site Clinic

Figure 12-51: Front Elevation of Site Clinic

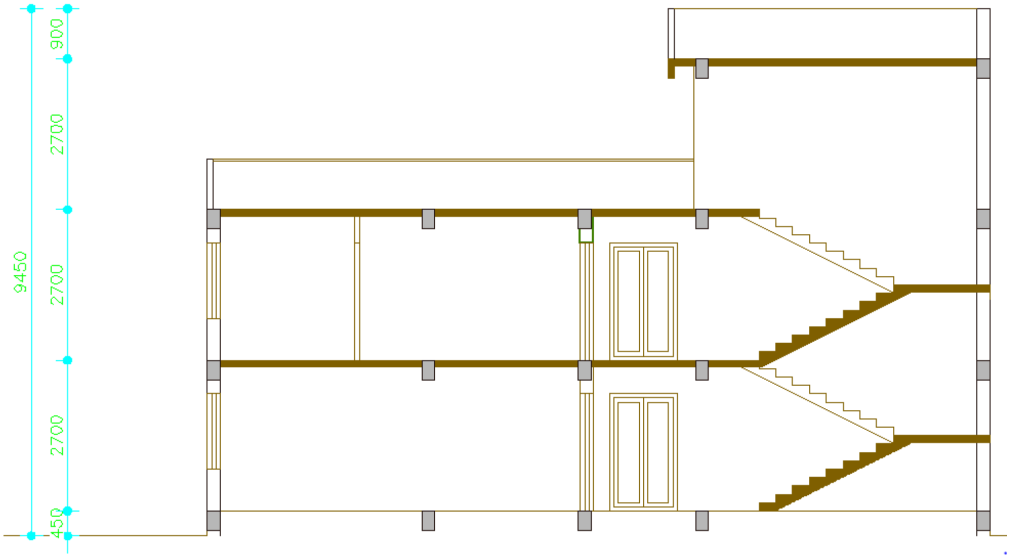

Figure 12-52: Section of Site Clinic

12.2.13.1 Specific Design Criterial

Please refer to 10.2.1.1

12.2.13.2 Design Method Applied

Modeling of Structure in FEM Software

3-D model of building is prepared on FEM software the basic data for analysis were adopted as discussed above.

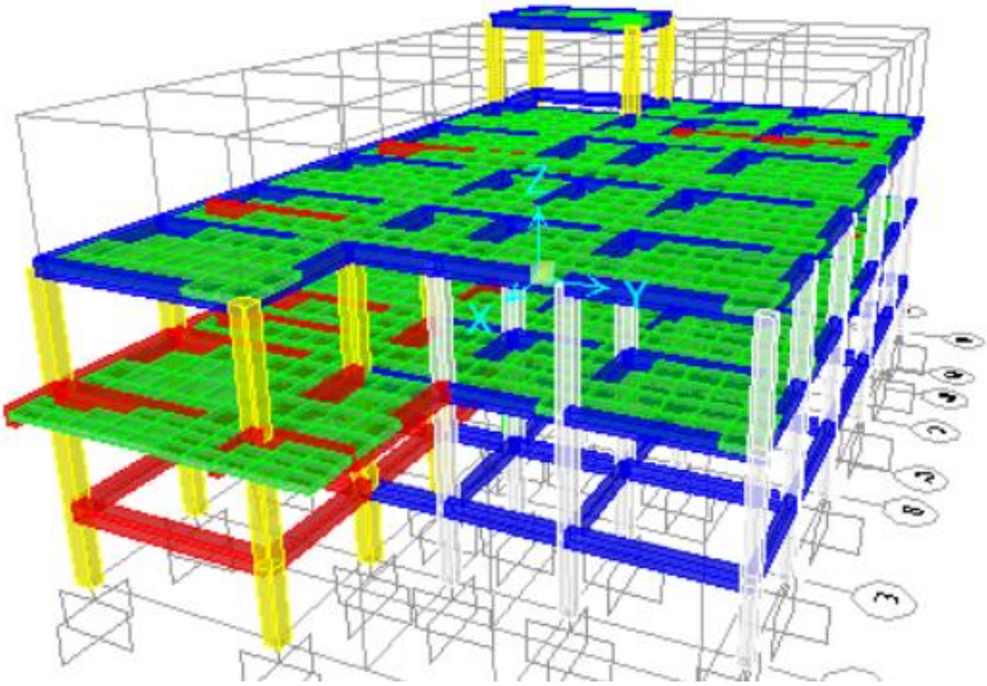

Figure 12-53: FEM Model of Site Clinic

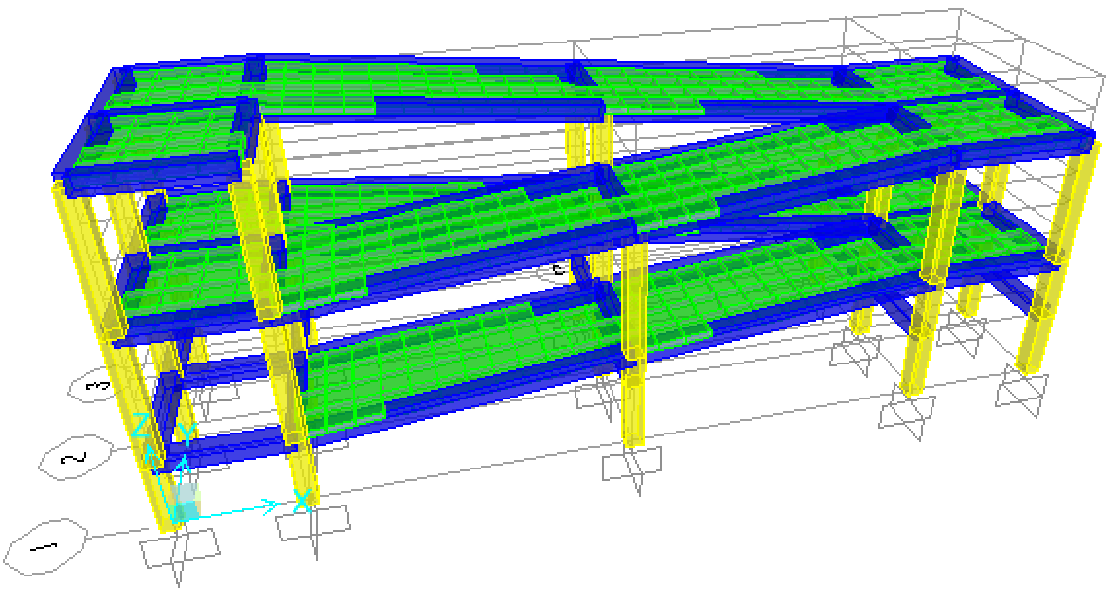

Figure 12-54: FEM Model of Ramp

12.2.14 Guard Post

Guard Post of 1.5mx15.m plan area with 2m clear inner height is proposed. CGI covering with four side slope is supported on metal rafters which are supported on vertical posts at four corners forming square metal welded frame. Post are framed with horizontal member at eaves board levels.

Curtain wall is proposed of suitable prefabricated board and two numbers fixed glazed windows and are hinged type metal door is proposed. This Guard post is usual light structure and construction detailing as follows are to be sufficient for stability and safety of proposed structure system.

Post: 75x75x5mm box

Bracing: 75x75x5mm

Rafter/Purlin: 50x50x3mm

Foundation: 600mm thick stone masonry back fill and 200mm thick M20 grade concrete with single top layer nominal reinforcement properly attach prefab board far walling.

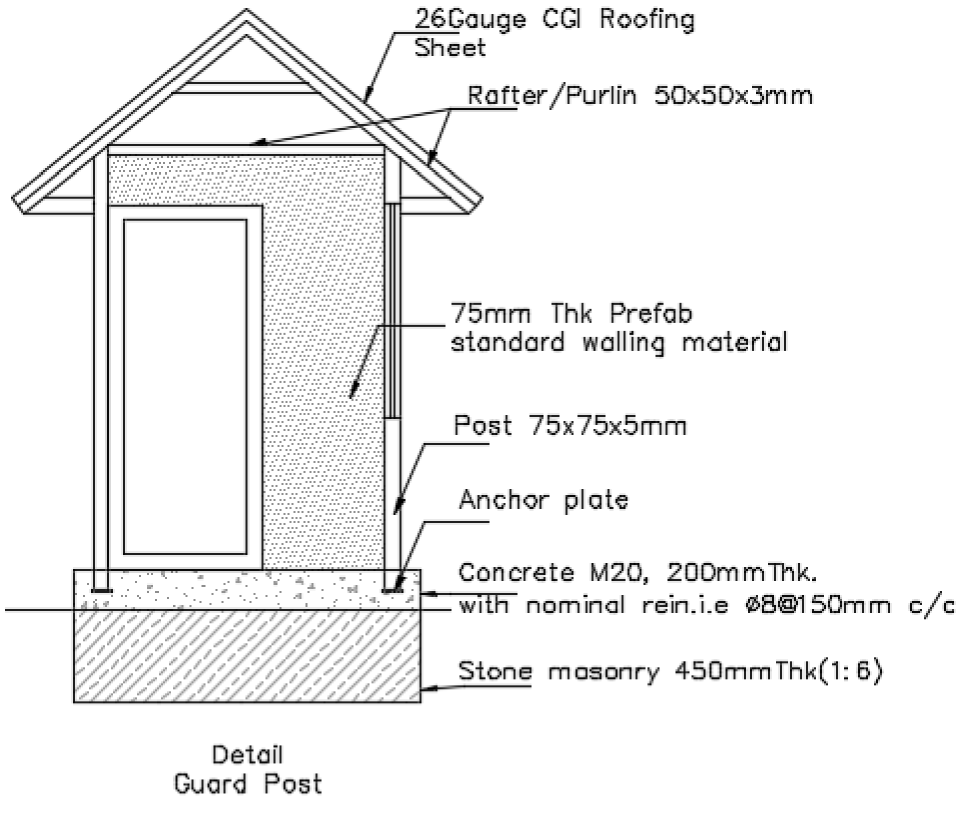

Figure 12-55: Detail of Guard Post

12.3 Project Roads

12.3.1 Acronyms used in this Subchapter

DOLIDAR	Department of Local Infrastructure Development and Agricultural Roads
DoR	Department of Road
GoN	Government of Nepal
IRC	Indian Roads Congress
ToR	Terms of Reference
VDC	Village Development Committee

12.3.2 Introduction

The Tamakoshi-V Hydropower Project is located approximately 170 km north east of Kathmandu, the capital of Nepal and approximately 40 km away from the district head-quarter of Dolkha District- Charikot Bazaar. The district head-quarter is connected with the capital by Arniko Highway upto Lamasangu which is approximately 90 km and by Lamasnagu – Jiri road which is approximately 40 km up to Charikot. A gravel road of about 33 km is available from Charikot to Singate Bazaar is recently upgraded for the construction of Upper Tamakoshi Project. The newly constructed road connecting Singate Bazaar and Lamabagar for the construction of Upper Tamakoshi HEP passes from the both powerhouse and the headwork sites of this project. This road is recently built.

12.3.3 Road Design Criteria

12.3.3.1 Road Type

Access Road of Hydropower Project falls under District Road and the design standard to be followed is that of Nepal Road Standard. The road type of project access road is Graveled road with hill side drain.

12.3.3.2 Connection to Access Road for Charikot-Lamabagar Road

The actual design level of the Charikot - Lamabagar road at the connecting point is unknown and it is tentatively considered by the judgment of present alignment.

12.3.3.3 Software

The road has been designed with the help of SW Road software considering the basic design criteria mentioned below.

12.3.3.4 Road Design Standards

Following road standards have been followed for design of access road:

- Nepal road standard 2070
- Nepal Rural Road Standards 2055 1st revision

- IRC SP-42-2014

The basic design parameters for the Access Road has been described in following sub-sections.

<u>Pavement/Sub-base for graveled road</u>

200mm thick layer of leveled compacted sub grade is prepared and 150 mm thick layer of gravel (river gravel) sub base is placed carefully over a well compacted and leveled sub grade. With final layer of 25mm base is prepared. Gravel material is spread with greater thickness at center and less towards the edge so as to obtain the desire camber.

<u>Carriageway</u>

The Carriageway has been designed by adopting following details:

- Width: 3.75 m
- Camber Slope: 3%
- Camber Side: Both Sides
- Layer: Sub-Base above well prepared Sub-grade.
- Material: Gravel

<u>Shoulder</u>

Shoulder has been designed by adopting following details:

- Width: 0.75 m
- Placement: Both Sides
- Camber Slope: 5.0 %
- Camber Side: Both Sides
- Material: Well prepared Sub-grade.
- Layer Thickness: No thickness is necessary due to the use of sub-grade as shoulder material.

<u>Horizontal Alignment</u>

The horizontal alignment has been fixed by introducing circular curves as the road has to deviate or bend from its original position to reach the destination. The radius of horizontal curve depends upon the horizontal angle of the respective Intersecting Points. The minimum radius of horizontal curve used is 15.0 m for the smooth and safe movement of vehicle in case of access road. However, in some cases 10m and 12.5 m radius has been used at some bends in order to decrease the excessive quantity of excavation in exceptional cases. The details of the horizontal alignment are presented in DDR Part E - Drawing of the report.

<u>Vertical Alignment</u>

The vertical alignment of the road is provided for smooth longitudinal profile consistent with category of road and terrain. The vertical alignment of access road is governed by Design Speed and corresponding Sight Distance considered in the design. Following details has been adopted in the design of vertical alignment:

- Design Speed: 30kmph

- Stopping Sight Distance: 30m
- Minimum length of vertical curve: 20m
- Maximum Gradient: 10%
- Minimum Gradient: 1% (0% in Crossing Structures)
- Minimum Change in Grade: 1.5%
- Average Gradient: 7%
- Exceptional gradient: 12%
- Maximum length of road in excess of average gradient: 200 m
- Maximum length of recovery at the grade specified: 100m/ grade: 4%-6%

The details of vertical alignment are presented in DDR Part E - Drawing of the report.

Extra – Widening

Extra-Widening of the access road is necessary to compensate additional space to the road width occupied by the vehicles on the curve due to the tracking of rear wheels. The Extra – Widening (EW) has been varied depending upon the radius of horizontal alignment as tabulated below:

Extra-widening has been placed such that One-third of its total width will be provided at the inner side while the rest at the outer side of the horizontal curve as far as practicable.

Table 12-1: Extra Widening with respect to Radius of Horizontal Alignment

Radius (m)		EW (m)
From	To	
0.0	12.5	2.0
12.6	20.0	1.5
21	40	1.2
41	60	0.8
61	80	0.5
81	100	0.3
101	1000	0

Elevation

Super-Elevation (SE) is necessary to reduce the effect of centrifugal force in the vehicle while moving along the curves. As in the case of extra – widening, super - Elevation of the access road has also been varied depending upon the radius of horizontal alignment as tabulated below:

Table 12-2: Super Elevation with respect to radius of Horizontal Alignment

Radius (m)		SE (%)
From	To	
0.0	20.0	7.0
20.0	35.0	6.0
35.0	55.0	5.0
55.0	80.0	4.0
80.0	1000.0	3.0

Pass-by

Pass-by is also one of the important components of the road and is allocated to provide extra space in the roadway for the purpose of short term parking in case of repair or get off and ride operation. As per standard, Pass-by should be provided in every 300 meters interval of the road length with the details to be included as below:

- Length interval: 300m. (Approximately)
- Full width length: 9m.
- Transition length: 6m.
- Total Width: 3.5m.

Pass-by will be constructed at the most economical location as determined by the terrain and ground condition, rather than at precise intervals.

Earthwork

Earthwork is a very common item in road works and is necessary whenever the road structure is above or below the natural ground level. The earthwork which includes cutting and filling of the natural ground disturbs the natural slope will be given utmost care in design as to maintain the stability of the ground and protect the road structure from possible slides and erosions. The general topography and type of soil plays a governing role in determining the cut-fill slope for earthwork. The details of earthwork in excavation and filling along with slope stability analysis for critical section are presented in DDR Part E - Drawing of the report. According to the design standard, the cut-fill slope has been kept, as far as practicable, as following:

In Cutting

Table 12-3: Cut Fill Slope

Type of Soil	Cut-Slope (H : V)
Hard Rock	Almost Vertical
Soft rock	1 : 6.0
Hard Soil	1 : 3.0
Ordinary Soil (OS)	1 : 0.33

In Filling

Filling work is done using suitable filling material with adequate compaction to prevent it from settlement and other stability related problems. As per standard, Filling will be done in the ratio of 1 vertical: 1.50 horizontal as far as possible for embankment formation with the lower limit not below 1 vertical: 1 horizontal. However, the slope will be adjusted depending upon site conditions to prevent ponding and other drainage related problems. For high embankments, where the fill slope goes below 1.5 horizontal for 1 vertical, Bio-Engineering is recommended for stability.

In both cutting and filling works, if the vertical height exceeds 5m, it is recommended to introduce horizontal berm of about 1m-1.5m for stability consideration. However, the requirement of berms also depends upon natural ground slope and other geological conditions. While designing, the earthwork will be such that the cut and fill quantity is in balanced condition in mild sloped topography, as far as possible.

Retaining Walls

Like Earthwork, Retaining Wall is also common in road construction works especially in hilly and mountainous areas. The proper care has been given in the selection of type and size of retaining walls while designing the access road. Skin walls or Breast wall are used to retain the soil at hill side above the road while gabion walls below the road at valley side has been designed to withstand the load of road structure above along with the lateral earth pressure.

Breast walls at hill side are proposed mostly at places where the topography consists of soft or loose soils and may seem unnecessary in rocky area. Breast walls are proposed at hill side where the height of cut is more than 10 m and slope is more than 600 (1V:0.55H) in order to stabilize the soil. Masonry retaining walls for heights greater than 4m are proposed in valley side. Gabion retaining walls are also proposed for heights upto 5m as per site conditions. Reinforced concrete may also be necessary in case of greater heights (above 7m) depending upon the slope of ground. The details of the retaining structures proposed at various chainages are included in DDR Part E - Drawing of the report. However, additional breast walls may be required during construction and shall be decided by the site engineer as per site condition.

Drainage Facilities

Since the access road of hydropower project mostly lies in hilly areas, the design has been done adopting the concept of Hill Road. As such, the water behavior under the catchment area has been carefully considered in design. The design has provisions for good drainage system so that the flow of water within the road structure is properly drained out to the nearest natural water course. Drainage system consists of suitable Side drainage and Cross drainage structures at required places.

Side Drain

Depending upon the catchment area and the gradient of the road, the side drainage has been placed wherever required. The side drain detail is basically governed by the catchment area to be served by it. In most of the cases, the side drain is located at the hill side of the road in order to collect the water flowing down the hill along with surface runoff and drain it out through the proper outlet. Due to limitation in the formation width, the side drains are constructed to such a shape that at emergency vehicles could utilize this space for crossing at low speed or for parking. The Side drain is provided at both sides where the Box-cutting for the road is encountered. The surface water is collected and disposed off at the nearest stream, valley or water course using cross drainage structures. The usual types of drain are kerb and channel type (tick drain), trapezoidal drain, rectangular drain, etc. In narrow roads, kerb and channel drains are preferred in hilly region as the drain can be used in case of emergency for parking or overtaking. Kerb and channel type drain are used in all the 180 degree bends. Besides kerb and channel drain, trapezoidal drains can also be used as side drain. The side drain has been designed as per the catchment area and contributing length of road.

Cross Drain

Wherever the access road has to cross the existing water ways like rivers, springs, streams, gullies etc., it becomes necessary to introduce the cross drainage structures as required. Bridges, Culverts and Causeways are the most common examples to be used in Cross drainage systems. A good drainage system incorporates all the cross drainage facilities depending upon the catchment area of the Crossing structures.

For the length of above 6m, bridge is recommended while the culverts are recommended for the lengths up to 6m. Bridges may be major or minor ones and is designed separately in case of need considering all the hydrological and geological parameters in detail. Similarly, culverts may be of various types like Pipe culvert (Single pipe or double pipe), Slab culvert or Box culvert. The type and size of culvert is governed by the catchment area and the flow characteristics of the natural drainage. Causeways, depending upon the type of material used, may be Dry stone causeway or Concrete causeway. Dry stone causeway is used where the discharge of kholshi is very low whereas Concrete causeway is used where the discharge is comparatively high.

In bridges and culverts, the natural drainage is allowed to pass below the roadway without any interruption while in case of causeways; the flow gets disturbed as it has to pass through the roadway as a result of gradient restrictions. Cross drainage may also be required at places where the road alignment passes through valley curve so as to properly drain out the flow coming from the side drains at both ends of the curve.

Appropriate side drainage has been included in the design. As per NRS 2070, outlets from the side drains should be provided at no more than 500 m interval and as per site necessity. So, Catch pits and pipe culverts have been provided at various chainage.

12.3.4 Protection of the Environment

12.3.4.1 General

Road construction in hilly area may lead to destruction of vegetation, trees, cultivable lands, etc. The natural slope of the ground level is disturbed during road construction. This might result in potential adverse effect on the environment. In order to reduce and avoid such adverse environmental consequences of the project, various mitigation measures need to be adopted. These mitigation measures will not only help to maximize the benefits of project but also minimize undesirable impacts on the project. Following measures shall be adopted to minimize the environmental impact of the project.

- Check dams and retaining wall will be constructed at various chainage in order to prevent soil erosion. Check dams are usually provided in big size kholshis having high gradient to reduce the bed erosion. However, there is not such case in this project.
- Road alignment has been fixed by minimizing the tree cutting.
- Tree cutting at landslide prone area will be avoided which has been also considered during the design as far as possible.
- Bioengineering shall be applied at landslide prone area
- Excavated materials will not be placed at steep sloppy area
- Plantation shall be done where ever possible
- Excavation by blasting shall be avoided.
- Excavated muck shall be levelled properly to control soil run off by monsoon rain.

12.3.4.2 Recommended species for bioengineering in road sector

Grasses

- Babio (Eulaliopsisbinita)
- Khar (Cymbopogonmicrotheca)
- Napier (Pennisetumpaniceum)
- Amliso (Thysanolaena maxima)
- Kans (SaccharumPontaneum)

Shrubs / Small Trees

- Chutro (Berberisaristata)
- Ainselu (Rubuselipticus)
- Areri (Acacia pennata)
- Bains (Salix tetrasperma)

Trees

- Chilaune (Schimawallichi)
- Jamun (Syzygiumcumini)

- Kutmiro (Litseamonopetala)
- Okhar (Juglansregia)
- Ratosiris (Albiziajulibrissin)

12.3.5 Road Furniture

12.3.5.1 General

Road Furniture means the additional items in the road which may seem necessary to be included to make the road serviceable and operable in the long run. The various items which are included in this category are listed as below:

12.3.5.2 Road Signs

There are different types of signs which are to be located at particular places as per requirement. Road signs are placed especially for the purpose of disseminating important and useful notices and warnings regarding the road characteristics for its users. These signs mostly contain symbols or texts in some cases to represent the road information. The design standard of road signs which has been separately issued by the road department shall be followed in the design of access road. Signs indicating No Entry, No Parking, Sharp Curve Ahead, Bridge Ahead, Hump Ahead, Turnings are common examples of road signs.

12.3.5.3 Distance Signs

Distance signs may be necessary to get the information about the distance travelled in particular stretch of road. This information is obtained by the construction of Kilometer Posts at regular intervals. The design of kilometer posts will be done in accordance with the standard set up by the Road Department.

12.3.5.4 Safety Components

For the sake of road users, Safety components are necessary at places where there is more probability of road accidents to occur. These places mainly include sharp bends, steep slopes at valley side, etc. which requires safety measures to be carefully considered in design. For this, Delineators, Iron railings and other safety measuring devices shall be used, where necessary, by following the standard of the Road Department. Besides these, Speed breakers, Barriers shall be designed in the restricted areas depending upon site conditions.

12.3.6 Drawing Preparation

12.3.6.1 General

All the drawings has been prepared using SW road software and AutoCAD. The details provided in each drawing are explained below.

12.3.6.2 General Layout

The overall layout of the project area including access road, structures, camp area, bench marks, etc. has been prepared in 1:25000 scale. The general layout of the access roads has been prepared in 1:2000. The overall layout of the access road along with road width and right of way has been shown in DDR Part E - Drawing of the report.

12.3.6.3 Plan and Profile

The plan of the project access road has been prepared in 1:2000 scale and profile of the project access road has been prepared in 1:1000 scale. The profile consists of information about vertical alignment including the design level and existing level at various chainage along with details of Intersection points, radius used in horizontal alignment and Soil type. The plan consists of information about the horizontal alignment of the road along with drainage facilities, retaining structures adopted.

12.3.6.4 Cross Section

The cross section of the project access road has been prepared in 1:200 scale. The cross section drawings have been prepared at interval of 10m. Cross section consists of details of the pavement, drainage facilities, retaining structures and earthwork at various chainage.

13 CONSTRUCTION PLANNING AND QUANTITIES

13.1 Introduction

13.1.1 General

This part of the Tamakoshi V Hydropower Project report covers the construction planning aspect of the project. The following areas are covered in this report:

- Project geographic location, site accessibility, project start-up requirements, manpower availability and effective working hours
- Required construction materials, storage and major equipment
- Estimated quantities of major civil works
- Construction approach for the different components of the Project
- Detailed and summary project schedules
- Method of construction supervision

13.2 Project Area, EMAP and Site Access

13.2.1 Project Area

The Tamakoshi V project lies in the Dolkha District, Janakpur Zone of Nepal. The Headworks site is located between Purano Jagat and Gongor village. The underground Powerhouse is located approximately 7 Km upstream from Singati Bazar.

This project was conceptualized as a tandem arrangement with the Upper Tamakoshi HEP. The project was identified best project by the Project Development Department (PDD) of Nepal Electricity Authority during the project identification study conducted by the department.

The feasibility study of the project was conducted by the PDD of NEA. The installed capacity of the project as per present design is 99.8 MW.

13.2.2 Environment Survey Report (EMAP)

The project proponent Nepal Electricity Authority (NEA) has obtained survey license from the then Ministry of Water Resources to carry out the feasibility study including EIA of the proposed project. The EIA of the proposed project was prepared and got approval from the Ministry of Population and Environment (MoPE). The present Detailed Engineering Design for Tamakoshi-V has an installed capacity is 99.8 MW with 4 units, 3 units of 31.6 MW and 1 unit of 5.0 MW.

The Field Investigation Report, Part A - Environment Survey Report (EMAP) from August 13, 2018 was prepared based upon the EIA and other documents.

The EMAP is an environmental operation manual for use by management and staff employed on the project, and will serve as an advisory document to regulatory authorities such as Ministry of Water Resources and Energy, Ministry of Forest and Environment, Gaurishankar Conservation Area Project.

The EMAP has five primary objectives, namely to:

- define environmental management principles and guidelines for the design, construction and operation of the project;
- establish the roles and responsibilities of all parties involved in project environmental management;
- describe mitigation measures that shall be implemented to avoid or mitigate adverse environmental impacts
- formulate environment management framework to ensure the implementation of mitigation measures and monitoring programs; and
- establish a supervision, monitoring, auditing and reporting framework.

The proposed project acquires 25.53 ha land for the placement of project structure and facilities. Out of this 14.24 ha land will be permanently acquired and 11.29 ha will be taken on lease for the temporary uses. Included are 10.44 ha cultivated low land (Khet), 4.95 ha forest area, 3.00 ha barren and 5.62 ha river and flood plain.

13.2.3 Road Access

The Tamakoshi V project is located about 90 km North-East of Kathmandu and approximately 40km away from the district headquarter of Dolakha District-Charikot Bazaar. The district headquarter is connected with the Capital by Araniko highway up to Lamasangu which is approximately 90 km and by Lamasangu-Jiri road which is approximately 40 km up to Charikot. A gravel road of about 33 km is available from Charikot to Singati Bazaar which has been upgraded for the construction of Upper Tamakoshi Project. The new road constructed, connecting Singati Bazaar and Lamabagar for construction of Upper Tamakoshi HEP passes from the both powerhouse and the headwork sites of this project.

The Araniko Highway has comparatively mild gradients (maximum 8 percent), is a Class 70 road (allowing for maximum single axle loads of 26.4 tonnes and a maximum point load of 62.5 tonnes). The design speed is 32 km/h.

Landslides during the rainy season on the Singati-Gongar road can be experienced during the rainy seasons, blocking the vehicular traffic ranging from a few hours to several days and can even extend up to a few weeks. This factor should be considered while planning activities during the rainy season.

Distances between project site and major landmarks are as follows:

Table 13-1: Major Landmarks

Major Landmark	Distance (km)	Importance
Kathmandu	170	Capital City
Charikot Bazaar	40	District Capital
Gaurishankar Conservation Area	Project Area is within Con-	Conservation
Singati Bazaar	3	Nearest Market

13.2.4 Access by Railway

The most appropriate and easy access by railway for the transportation of materials and equipment is from Kolkata, India to Birgunj, Nepal. Total railway distance between Kolkata and Birgunj is 763 km. Recently, a 53 km railway line was started from Jaynagar, India to Janakpur, Nepal, which is primarily used for passenger transport. The distance from Janakpur railway station to Charikot, Dolakha is184 km.

13.2.5 Access by Air

There is no airport close to the project site. Kathmandu's Tribhuvan is the closest international airport at about 180 km; however, helipads will be available which can be used in case of extraordinary situations.

13.2.6 Project Access and Construction Roads

Main access to the project site is along the Singati-Lamabagar public access road. However, a network of access roads will have to be built to manage the construction activities. Later some of these roads will be used permanently for the O & M activities.

13.3 Main Works Start-up Requirements

Upon completion of the Detailed Design Report, several preliminary works must be completed before the start of the main works on the tunnels and Power Station. These activities include:

- Project approval and funding arrangements
- Consultant selection process
- Development of tender documents and drafting of the contract documents including technical specifications
- Pre-qualification, tendering and contractors' selection process
- Relocation and re-settlement process complete

Each of the above activities requires a time frame of months to complete. Most of these activities need to run in parallel before construction of the main works can begin.

The monsoon period in the Project area spans from June to November each year. In view of this, an unimpeded working season of 9 months from October to May was considered for surface work. It was also considered that work will continue during the monsoon season although the progress rate of surface work might drop to as low as 30% of normal. However, underground work will continue unimpeded throughout the year.

To shorten the duration of the main works, preliminary works such as land acquisition, camp facilities, Client and Consultant's housing, resettlement and relocation, plant areas, stores and approach roads should be completed before the awards of the main contracts for construction of the main works.

13.4 Manpower

13.4.1 Unskilled and Skilled

13.4.1.1 Unskilled

Unskilled labour including drivers, guards, clerical staff, etc. will be all indigenous and mostly available from towns and settlements close to the project site.

13.4.1.2 Skilled

Skilled labour will be required for the operation of construction equipment, rock drilling and blasting, welding, heavy equipment repair, concrete form work construction, installation of E&M equipment, batching plant operation, crushing and screening plant operation, surveying, quality control, testing of construction materials, and other construction-related laboratory and field testing. This skilled labour will be a mix of expatriate and local technicians. Such manpower must be brought from different parts of the country provided they are given proper salary incentives and living facilities.

13.4.2 Effective Working Time

Non-productive periods

To determine the net effective working time available for construction, the following non- productive time was deducted from the total available time:

Saturdays

Saturday is a weekly holiday in Nepal. It was, therefore, assumed that, under normal circumstances, only six days would be worked each week.

Public Holidays

There are 16 official public holidays in Nepal, as shown in the below table. Additionally, there are some optional holidays, corresponding to religious or civil occasions.

Table 13-2: Official Public Holidays

S.N.	Date	Day	Occasion
1	15-Jan	Mon	Maghe Sankranti
7	4-Mar	Mon	Maha Shivaratri
8	8-Mar	Thu	International Women's Day
10	14-Apr	Sat	Nepali New Year
11	20-Apr	Wed	Holi
12	30-Apr	Mon	Buddha Jayanti
13	1-May	Tue	Labour Day
14	19-Sep	Wed	Constitution Day
15	16-Oct	Tue	Fulpati
16	17-Oct	Wed	Maha Ashtami
17	18-Oct	Thu	Maha Navami
18	19-Oct	Fri	Vijaya Dashami
19	20-Oct	Sat	Ekadashi
20	7-Nov	Wed	Laxmi Puja
21	8-Nov	Thu	Govardhan Puja
22	9-Nov	Fri	Bhai Tika

Effective Working Periods

Based on the above considerations, the effective working time was determined as 25 days per month.

For the purposes of development of the cost estimate and construction schedule, the required production rates were calculated based on the effective working time.

Daily shifts

It is expected that critical path activities will have round the clock operations, but this is less likely for non-critical activities for the following reasons:

- Additional expenses in labour camp and catering costs.
- Additional supervisory expenses.

- No spare capacity of manpower and equipment would be available for peak production or emergency periods.
- Adequate maintenance of equipment, haul roads and jobsite installations would be difficult.
- No margins would be available to perform tasks unforeseen or omitted from the schedule.
- Round the Clock Operations

Operations for activities anticipated to have round the clock operations are detailed as follows:

- All above-ground earthmoving (open excavations, processing and placing of fill, etc.) will proceed in two shifts of 8 hours each (leaving at least 4 hours for routine maintenance and fuelling).
- All underground excavations (tunnels, shafts, chambers, etc.) will proceed in three shifts of 8 hours each, which will be devoted to the main construction activity i.e. drilling, blasting, ventilation and mucking as well as for ancillary functions, such as the installation of supports, lighting, water and compressed air pipes, maintenance, etc.
- Conventional concrete production and placing will be in two shifts above ground and two shifts underground, with the batching plants operating 12 hours per shift.

Because of the above considerations, the net working periods shown in the following table were used as a basis for the scheduling computations.

Table 13-3: Net Working Periods

No.	Activities	Hours Per Sift		Shifts Per Day		Net Working Hours		
		Gross	Net	Gross	Net	Day	Month	Year
1	Above-ground earthmoving (excavations, quarries, earth and rock-processing, hauling and placing of fill, etc.)	9	8	2	2	16	400	4,800
2	Underground excavations (tunnels, shafts, caverns, etc.	9	8	3	3	24	600	7200
3	Above-ground concrete	9	8	2	2	16	400	4,800
4	Underground concrete	9	8	3	3	24	600	7,200

Note: The scheduling periods are based on 25 working days per month or 300 days per year.

13.5 Construction Materials and Equipment

13.5.1 Material Requirement, Procurement and Storage

13.5.1.1 Material Requirements

The table below summarizes the requirements of key construction materials for the Tamakoshi V Hydropower Project:

Table 13-4: Requirements of Key Quantities of Construction Materials

No.	Type of Material	Unit	Requirement
1	Stone Aggregate (includes sand)	m^3	188,000
2	Cement	tons	87,300
3	H-beams	tons	470
4	Reinforcing Steel (includes rock bolts)	tons	70,000
5	Explosives	tons	960
7	Plywood formwork	m^2	1510
8	Steel formwork	m^2	128,000

13.5.1.2 Procurement and Storage of Materials

Stone Aggregate

The volume of underground rock excavated in this project will be about 490,000 m^3. Another source of stone aggregate is the Tamakoshi River.

Cement

The following sources have been identified for supply of the cementitious materials for the Tamakoshi V project:

Table 13-5: Cement Manufacturers in Nepal

Manufacturer	Location	Production(tons/day)/ Distance (km)
Shivam Cement Industries	Nawalparasi	3000 tons/day Around 402.4km
CG Cement Industries	Dumbikas,Nawalparasi	1500 tons/day Around 474km
Jagadamba Cement Industries	Bairahawa	About 900 tons/day Around 447km
Udayapur Cement Industries	Jaljale, Udayapur	800 tons/day Around 337km
Hetauda Cement Industries	Lamsure, Hetauda	About 710 tons/day Around 332km

The cement required for this project will be acquired from Nepali manufacturers as well as imported from India.

Table 13-6: Cement Manufactures in India

Manufacturer	Location	Production(tons/day) Distance(km)
JSW Cement	Kolkata, India	6576 tons/day 883.33km
Ambuja Cement	Salat, Uttar Pradesh	81233 tons/day 961.1km
Birla Corporation	Durgapur, West Bengal	6301.370 tons/day 821km

The cementitious material (Portland cement) will be supplied in 50 kg bags and bulk and will be stored in weatherproof warehouses near the jobsite A storage capacity of around (equivalent to peak requirement of two three weeks). The use of the access roads to the site can be interrupted by inclement weather for periods of 8 to10 days. Cement silos with storage capacity of 7 days production will have to be provided close to the batching plants.

Sand

High quality and large sand deposits are available near the village of Lamabagar on the Tamakoshi River for use in the Tamakoshi V project.

Reinforcing Steel

The quantity of reinforcing steel required for the project will be about 70,000 tonnes, with a monthly average consumption of about 3,190 tonnes (22 months construction). Several factories in the country produce small quantities of reinforcing steel, in the form of both plain and deformed bars. There is no factory producing reinforcing steel bars near the proposed site. The nearest locations from where the reinforcing steel bars of the desired specification are available are in India.

The storage of reinforcing steel enough for one-month consumption (3,190 tons) will be required. Also, a bar bending, bar cutting and thread cutting facility will be required.

Explosives

The Nepalese Army will be capable of supplying all the explosives and blasting consumables required for the project. A total of approximately 960 tonnes of explosives will be needed, with an average monthly consumption of 50 tonnes.

Adequate storage depots, which are secure, will be needed at the jobsite, with a total capacity of about 50 tons (enough for one month of average operation).

Formwork, Steel Plates, plywood and Steel Formwork

About a 128,000 m² of steel formwork is required for Tamakoshi V Hydropower Project. Steel sheets of various thicknesses are produced at steel mills which can be used to fabricate steel formwork. These can also be used for fabricating steel liners and other miscellaneous items required in connection with the construction activities. Alternatively, steel items can be imported from abroad. The most likely source of steel plate could be nearby countries such as China and India. 1510 m² of plywood will be needed. This is based on reusing the plywood 20 times.

Petroleum Products

Petroleum products can be imported from India and will be stored at mechanical and storage yards proposed at interconnection area and Jagat. A capacity of around one-month worth of operation, to cope with extraordinary conditions like landslides on the project site roads, should be foreseen.

Water and Stone for Concrete Work

For the purposes of construction planning it is noted that most of the materials will be produced as a result of the required excavations or will be obtained from sources near the project site.

The Tamakoshi River water has been used in the making of concrete at Upper Tamakoshi HPP. The results of water samples taken from upstream of the dam site indicate that the water is suitable for mixing and curing of concrete. Water for concrete production and other needs can be obtained from this source. This water is pure and was used for the Upper Tamakoshi HPP.

Water for concrete production and other needs can be obtained by building a small storage reservoir.

13.5.2 General Construction Equipment

Most of the construction plant and equipment required for the execution of the works will have to be imported from international sources like excavators, dumpers, crawlers, cutters, drilling jumbos, drilling and grouting equipment, batching and concrete plant, vibrators, mixers, graders, rollers, dump trucks etc.; the same applies to most of the spare parts and construction plant and equipment-related consumable supplies. Due to the haulage conditions through hilly/mountainous terrain, the size of equipment will generally be smaller than would normally be used for a project of this scale, with most units weighing less than 40 tonnes when prepared for transportation. Large items of equipment may be brought to the site in knock-down condition for assembling at the site.

13.5.2.1 Batching Plants

The contractor will require work areas for various functions including space for stockpiling aggregates, aggregate manufacturing, and concrete batching and mixing plants. The demand for space will vary with time; areas will have to be developed as the construction activities progress. The concrete batching plants will require space for the 60 m³/h mixer, bins, stockpiles, operation room and silos. This facility need not be on level ground and it can be installed on a series of cut benches with delivery access to bins, batch and mix plant.

Two batching plants will be required for the project, one at the Headworks and the second at the Powerhouse areas. The space available for the batching plants is 1.4 ha and 0.7 ha respectively at the Headworks and the Powerhouse areas. See General Layout Drawing 31-00053-DD-4922-Q 1100 for the planned locations of the two batching plants.

Table 13-7: Batching plant peak performance required for the project

Batching Plant Usage Area	Peak Requirement m³/month	Capacity of each batching plant m³/h
Headrace Tunnel (lining + shotcrete)	7512 (conventional concrete + shotcrete)	60m³/h

13.5.2.2 Aggregate Processing Plant

The total required aggregate required for this project is about 188,000 m³. The aggregates for the Tamakoshi V project will be processed from material excavated from tunnelling operations. It is intended that two crushing plants will be located near the two planned batching plants. The rocks excavated from the caverns and tunnels will be transported to the aggregate processing plant using dump trucks, where they will be processed into the required aggregate sizes and stockpiled. Aggregate processing consists of crushing, screening, washing and stockpiling the aggregate to obtain proper cleanliness and gradation. The crushing plant capacity should be established according to the required production rate and amount of preproduction achievable. In consideration of the prescribed concrete production rate, the aggregate processing plant should have a production rate of 40 m³/h if no pre-production is carried out.

The processed aggregate will be stockpiled at the crushing plant in separate groups according to the grading. From the stockpiles, the aggregate will be transported to the batching plant using trucks.

The concrete production system comprises the batching plants, aggregate stockpiles, cement silos, storage of other ingredients of concrete (silica fume, admixtures, etc.), water supply and drainage system, and power supply system.

Four cement silos each with a capacity of 50 + 20 + 50 + 20 tons or more is foreseen. Compressed air should be used to deliver the cement for which mobile air compressors could be deployed.

For transport of conventional concrete 6 m^3 cement trucks are foreseen.

Proper curing of the fresh conventional concrete is an important step to ensure concrete quality. Depending on the ambient temperature at site, the surface of the fresh concrete should be cured through fog (or water mist) to create a local environment with lower temperature and higher moisture content, and/or covered with fiber blanket cover. If possible, concrete can also be cured with flowing water.

13.5.3 Camp

The Tamakoshi V project will require a substantial workforce from inside and outside of Nepal. Singati and the surrounding area is rural and does not have adequate facilities to accommodate such a large workforce. Therefore, adequate residential and office facilities must be established at convenient locations at the project site.

A permanent residential and office area has been envisaged close to the project site to accommodate the residential requirements of the Employer's and Engineer's Camp. For full description of buildings and building layout please refer to the drawings 31-00053-DD-5310-Q 1620 to 31-00053-DD-5310- Q 1633.

The Employer's and Engineer's Camp contains the following main components:

Table 13-8: Employer's and Engineer's Camp

Description	Total Area (m^2)
Camp Buildings	7,490
Camp Landscaping	35,000
Parking Area	868
Asphalt Roads Inside Camp and access to Helipad (900 x 8.5 m)	7,650
Helipad (30x30 m)	900
Gravel Access Roads (230 m x 8.5 m)	1955
Main Water supply pipe (diameter 90 mm) 2,330 m	-
Reservoir Tank (50m^3)	-
Two Sewage Treatment Plants, 780 m pipe	-
Street Lighting (880 m and 30 poles	-
Security Fencing	-
Temporary Power Supply Singati to Camp (11 kv line)	-

13.5.4 Bridges

There are three existing bridges which cross the Tamakoshi River along its reach corresponding to the length of the project site; the Bhorle Bailey Bridge, the Jamune Bridge and the Jagat Bridge. Two additional bridges are proposed to access Adits 2 and 3. The construction of these two temporary bridges will be the responsibility of Contractor 1.

13.5.5 Construction of Access Roads

The public road between Singati and the Upper Tamakoshi HPP was purpose built for this project. This road will be used during the construction of the Tamakoshi V project. Additional construction roads, however, will be required directly on site.

Table 13-9: Additional Construction Roads to be Constructed

No.	Description
1	Access road to Adit 2 (Temporary road, contractor's responsibility)
2	Access road to Adit 3 (Temporary road, contractor's responsibility)
3	Access road to Adit 4 (Project Road)
4	Access road to Surge Tank Ventilation Tunnel (Project Road)
5	Access Road to (MAT) Project Road (Part of Power Station area. To be developed as part of outdoor works)
6	Access to Tailrace (Part of Power Station area. To be developed as part of outdoor works)
7	Access to Cable Tunnel (Part of Power Station area. To be developed as part of outdoor works)
8	Permanent Camp Roads (Part of Power Station area. To be developed as part of outdoor works)

13.5.6 Care and Handling of Water

Appropriate measures will be required to maintain dry all open-cut and underground work areas by use of drainage and dewatering systems to remove seepage and rainfall water.

These open-cut and underground dewatering facilities will require installation, operation and maintenance of drainage collection systems and sumps, electric power transmission lines, pumping stations with extensive discharge pipelines, and sedimentation ponds.

13.5.7 Spoil Areas

Seven spoil areas were planned for this project. The volume of underground rock excavated in this project is about 490,000 m^3. This is sufficient for the cofferdam construction and for the aggregate requirements for the project.

13.5.8 Temporary Cofferdam

Features and major work items for the temporary cofferdam are as follows:

Fill quantity for Cofferdam: 5000 m³

Required crest level of cofferdam: 986.50 m asl

Construction of the cofferdam will be done during the dry season. The cofferdam will be built to a crest elevation 986.50 m asl, sufficient to divert the river and to allow construction of the Outlet Structure. The cofferdam design will be within the Contract 1 EPC contractor’s responsibility; tentatively it has been assumed to be a random embankment with an internal grout curtain for improved water tightness.

Placement of stockpiled material will be carried out by 2 m³ hydraulic excavators in combination with 20/25 MT rear dumpers. The equipment required for spreading and compaction work would be crawler dozers and 10/20 t vibratory rollers, water sprinklers, etc.

13.6 Planning and Construction Methodology

13.6.1 Introduction

Tamakoshi V Hydropower Project involves the construction of various components. These components are presented in the introduction of this report; construction relevant details are further provided in the annexes to Part C of the report (refer to Tunnel Works Structures, Surface Works and Permanent Camp, respectively).

The location, arrangement and structural details of the above components are shown in the Album of Drawings, Part E of the Detailed Design Report.

13.6.2 Major Quantities for Civil Work

The quantities of the major items of work and materials involved in the construction of Tamakoshi V Hydropower Project are presented in the introduction of this report; construction relevant details are further provided in the annexes to Part C of the report (refer to Tunnel Works Structures, Surface Works and Permanent Camp, respectively).

13.6.3 Project Key Activities

The quantities of the major items of work involved in the construction of Tamakoshi V HEP are summarized in three tables below corresponding to Contract 1, Contract 2 and Contract 3. Refer to the Tamakoshi V Time Schedule in, Part E, Album of Draws, of the Detailed Design Report and the Tender Document for further details.

Table 13-10: Major Activities Contract 1

Activity ID	Activity Name
1	**Mobilisation and Site Installations** Mobilisation of key personnel and main construction equipment. Construction of site installations, buildings, temporary access roads and site installation services.
2	**Access Roads and Portals for Adits** Completion of access roads and excavation and support of the portals for Adit 1, Adit 2, Adit 3 and Adit 4
3	**Excavation and Rock Support Adits 1 to 4** Excavation and installation, testing and maintenance of all necessary rock support measures in Adit 1, Adit 2, Adit 3 and Adit 4 (including all branches of these Adits). Included are all dewatering measures, ventilation and all other tunnel excavation support measures.
4	**Headworks Tunnels and Headworks Structures** Completion of structures at the Headworks, including excavation, support and concreting.
5	Excavation and Rock Support of Headrace Tunnel (HRT), from fix point HRT-00 to HRT-A2 (From Adit 1 to Adit 2) Excavation and installation, testing and maintenance of all necessary rock support measures of Headrace Tunnel (HRT), from fix point HRT-00 to HRT-A2, including intake. Included are all dewatering measures, ventilation and all other tunnel excavation support measures.
6	**Excavation and Rock Support of Headrace Tunnel (HRT), from fix point HRT-A2 towards HRT-00 and HRT-A3 (From Adit 2 to Adit 1 and Adit 3)** Excavation and installation, testing and maintenance of all necessary rock support measures of Headrace Tunnel (HRT), from fix point HRT-A2 to HRT-00 and HRT-A3. Included are all dewatering measures, ventilation and all other tunnel excavation support measures.
7	**Excavation and Rock Support of Headrace Tunnel (HRT), from fix point HRT-A3 towards HRT-A2 and HRT-A4 (From Adit 3 to Adit 4)** Excavation and installation, testing and maintenance of all necessary rock support measures of Headrace Tunnel (HRT), from fix point HRT-A3 to HRT-A2 and HRT-A4. Included are all dewatering measures, ventilation and all other tunnel excavation support measures.
8	**Concreting of Headrace Tunnel (HRT), from fix point HRT-00 to HRT-A2 (From Adit 1 to Adit 2)** Concreting and completion of Headrace Tunnel (HRT), from fix point HRT-00 to HRT-A2. Included are all auxiliary works necessary to complete this section of the tunnel.
9	**Concreting of Headrace Tunnel (HRT), from fix point HRT-A2 to HRT-A3 (From Adit 2 to Adit 3)** Concreting and completion of Headrace Tunnel (HRT), from fix point HRT-A2 to HRT-A3. Included are all auxiliary works necessary to complete this section of the tunnel.
10	**Concreting of Headrace Tunnel (HRT), from fix point HRT-A3 to HRT-A4 (From Adit 3 to Adit 4)**

Activity ID	Activity Name
	Concreting and completion of Headrace Tunnel (HRT), from fix point HRT-A3 to HRT-A4. Included are all auxiliary works necessary to complete this section of the tunnel.
11	**Excavation and Rock Support of Headrace Tunnel (HRT), from fix point HRT-A4 to 5 m beyond centre line of vertical connecting shaft of the Surge Tank (From Adit 4 to Surge Tank+5 m, (start of steel lined 4.2 m dia. section))** Excavation and installation, testing and maintenance of all necessary rock support measures of Headrace Tunnel (HRT), from fix point HRT-A4 to Surge Tank+5 m. Included are all dewatering measures, ventilation and all other tunnel excavation support measures.
12	**Concreting of Headrace Tunnel (HRT), from fix point HRT-A4 to 5 m beyond centre line of vertical connecting shaft of the Surge Tank (From Adit 4 to Surge Tank+5 m)** Concreting and completion of Headrace Tunnel (HRT), from fix point HRT-A4 to Surge Tank+5 m (end of concrete lined section). Included are all auxiliary works necessary to complete this section of the tunnel.
13	**Excavation and Rock Support of Surge Tank, including Connecting Shaft to HRT and Surge Tank Ventilation Gallery** Excavation and installation, testing and maintenance of all necessary rock support measures of Surge Tank. Included are all dewatering measures, ventilation and all other excavation support measures.
14	**Concreting and completion of Surge Tank, including Connecting Shaft to HRT and Surge Tank Ventilation Gallery** Concreting and completion of Surge Tank, including Connecting Shaft to HRT and Surge Tank Ventilation Gallery
15	**Excavation and Rock Support of (Upstream) Valve Chamber, including tunnel from Surge Tank from 5 m beyond centre line of Surge Tank Connecting Shaft to start of upstream bend of Pressure Shaft (dia. 4.2 m)** Excavation and installation, testing and maintenance of all necessary rock support measures of Valve Chamber. Included are all dewatering measures, ventilation and all other excavation support measures.
16	**Concreting and completion of Valve Chamber, including tunnel from Surge Tank from 5 m beyond centre line of Surge Tank Connecting Shaft to start of upstream bend of Pressure Shaft (dia. 4.2 m)** Concreting and completion of Valve Chamber.
17	**Excavation and Rock Support of Pressure Shaft (dia. 4.2 m) from start of upstream bend, vertical section and to end of downstream bend** Excavation and installation, testing and maintenance of all necessary rock support measures of Pressure Shaft. Included are all dewatering measures, ventilation and all other excavation support measures and grouting behind steel lining.
18	**Excavation and Rock Support of Pressure Tunnel (dia. 4.2 m) from end of downstream bend to first upstream bifurcation** Excavation and installation, testing and maintenance of all necessary rock support measures of Pressure Tunnel. Included are all dewatering measures, ventilation and all other excavation support measures and grouting behind steel lining.
19	**Excavation and Rock Support of Upstream Manifolds from first upstream bifurcation to upstream side of Powerhouse**

Activity ID	Activity Name
	Excavation and installation, testing and maintenance of all necessary rock support measures of Upstream Manifolds. Included are all dewatering measures, ventilation and all other excavation support measures and grouting behind steel lining.
20	**Excavation and Rock Support of the Main Access Tunnel (MAT) and the Cable and Ventilation Tunnel (CVT)** Excavation and installation, testing and maintenance of all necessary rock support measures of the Main Access Tunnel (MAT) and the Cable and Ventilation Tunnel (CVT), including all branches. Included are all dewatering measures, ventilation and all other excavation support measures.
21	**Excavation and Rock Support for Powerhouse Cavern** Excavation and Rock Support for the Powerhouse Cavern, including the transformer cavern (Trafo-Cavern). Included are all dewatering measures, ventilation and all other tunnel excavation support measures.
22	**Powerhouse Cavern Service Tunnels** Excavation and support of all tunnels servicing the Powerhouse Cavern and Trafo-Cavern, including bus duct galleries, escape tunnel, access tunnel between the Powerhouse Cavern and Trafo-Cavern and all other service tunnels not elsewhere listed.
23	**Erection (Service) Bay in the Powerhouse Cavern** Concreting of the Erection Bay and installation of the Construction Crane
24	**Unit 1 Block** Concreting and embedded parts for the Unit 1 Block in the Powerhouse
25	**Unit 2 Block** Concreting and embedded parts for the Unit 2 Block in the Powerhouse
26	**Unit 3 Block** Concreting and embedded parts for the Unit 3 Block in the Powerhouse
27	**Unit 4 Block** Concreting and embedded parts for the Unit 4 Block in the Powerhouse
28	**Excavation and Rock Support of Downstream Manifolds from downstream side of Powerhouse to end of last downstream bifurcation** Excavation and installation, testing and maintenance of all necessary rock support measures of Downstream Manifolds. Included are all dewatering measures, ventilation and all other excavation support measures and grouting behind the steel lining.
29	**Excavation and Rock Support for the Tailrace Tunnel, including Adit to Tailrace Tunnel and Adit's portal, from TRT Adit towards Powerhouse and Outlet** Excavation and installation, testing and maintenance of all necessary rock support measures and concreting in Tailrace Tunnel. Included are all dewatering measures, ventilation and all other tunnel excavation support measures. Including completion of portal.
30	**Concreting and completion of Tailrace Tunnel** Concreting and completion of the Tailrace Tunnel, including concreting of the Down-

Activity ID	Activity Name
	stream Manifolds section. Included are all auxiliary works necessary to complete this tunnel section.
31	**Excavation, Rock Support, concreting and completion of Outlet Structure, including outlet portal and outlet cofferdam** Excavation, concreting and completion of Outlet (Tailrace) Structure. Included are the diversion of the river, including the cofferdam and all temporary access roads.
32	**Terminal and Ventilation Building** Excavation, support, concreting and completion of the Terminal and Ventilation Building, including all fittings and finishes
33	**Outdoor Buildings, including Operation Building, Switchyards Structures, Permanent Project Roads and Operator's Camp** Excavation, concreting, finishing and completion of Operation Building, Switchyards Structures, Permanent Project Roads and Operator's Camp
34	**Permanent Service Tunnels** The Permanent Service Tunnels will be fully completed, including the construction of a permanent concrete invert and all necessary fittings and accessories.
35	**Remaining Civil Works** Remaining civil works, including architectural works, landscaping, permanent drainage, fencing, auxiliary buildings and surface finishes, not covered by any of the other items and sub-items in this document.
36	**Hydromechanical Equipment** Supply and installation of all hydromechanical equipment, including all embedded parts, support systems (power and control), installation and testing.

Table 13-11: Major Activities Contract 2

Activity ID	Activity Name
1	**Mobilisation and Site Installations** Mobilisation of key personnel and main construction equipment. Construction of site installation buildings, access roads and site installation services.
2	Turbine preassembly Unit 1, including spiral case, stay ring, draft tube and governor
3	Turbine preassembly Unit 2, including spiral case, stay ring, draft tube and governor
4	Turbine preassembly Unit 3, including spiral case, stay ring, draft tube and governor
5	Turbine final assembly Unit 1, including cooling water system, compressed air system and oil handling system
6	Turbine final assembly Unit 2, including cooling water system, compressed air system and oil handling system

Activity ID	Activity Name
7	Turbine final assembly Unit 3, including cooling water system, compressed air system and oil handling system
8	Generator preassembly Unit 1
9	Generator preassembly Unit 2
10	Generator preassembly Unit 3
11	Generator final assembly Unit 1
12	Generator final assembly Unit 2
13	Generator final assembly Unit 3
14	Construction Crane in the Powerhouse Cavern (temporary crane under the roof)
15	Drainage and Dewatering System
16	Main Inlet Valves (MIVs) for Units 1, 2 and 3
17	Draft Tube Gates, Units 1, 2 and 3
18	Turbine and generator Unit 4, including turbine, generator and supporting systems
19	Cargo and Passenger Elevator
20	Main Transformers, including all supporting systems
21	220 kV GIS and Take Off Yards
22	Diesel Generating Unit
23	All remaining mechanical and electrical systems (Remaining Systems) in the project not cover by any of the above items.
24	220 kV Transmission Lines

Table 13-12: Major Activities Contract 3

Activity ID	Activity Name
1	**Construction of Access Roads to Camp** Construction of the access roads from the public road to the Employer's/Employer's Representative's camp, and the access roads inside the camp, including sub-base, base course and surfacing course, drainage and dewatering installations, road signs and markings.
2	**Construction of Buildings in Camp** Construction of the buildings in Employer's/Employer's Representative's camp, including foundations, wall, roofing, windows and doors, and all fittings and finishes.

3	**Completion and Commissioning of Services in Camp** Construction of the Services for the Employer's/Employer's Representative's camp, including water supply, wastewater system and supply of electricity. The electrical network within the camp, the supply of electricity to all buildings and all electrical fittings required within the buildings are included in this item.
4	**Supply and Installation of Furniture in Buildings** Supply and installation of all furniture required in the buildings of Employer's/Employer's Representative's camp, including household furniture, general office equipment and domestic appliances.
5	**Completion of Landscaping at Camp** Landscaping at the Camp, including the planting of grass and trees where required.

The following particular construction aspects were considered for construction planning:

- Contract 1: A 10 m long rock plug will be left in place during the construction period in the Connecting Tunnel. This plug will ensure that no accidental water inrush can occur from the UTK tailrace tunnel. The plug will be removed, and the tunnel section lined only at the very end of HRT construction, shortly before the HRT initial filling and wet testing is scheduled.

- Contract 1: A 10 m long rock plug will be left in place during the construction period in the outlet tunnel section between Outlet Structure Shaft and Tailbay. This plug will ensure that no accidental water inrush can occur from the Tamakoshi River and Tailbay. The plug will be removed, and the tunnel section lined only at the very end of Outlet Structure construction, shortly before the TRT initial filling and wet testing is scheduled.

- Contract 1: The riverside part of the Outlet Structure and Tailbay are foreseen to be constructed during the dry season of the year. The protection level of the cofferdam was selected to suit this construction concept. The riverside part of the Outlet Structure is scheduled to be constructed during two subsequent dry seasons. The cofferdam may be overtopped during the imbedded wet season; a partial reconstruction of the cofferdam at the beginning of the second dry season was considered in the Construction Time Schedule.

- Contract 1: The bridges of the access roads to the portals of Adits 2 and 3 are a priori considered to be temporary structures. They are within the scope of design/construction responsibility of the Contract 1 EPC contractor and will have to suit the requirements of the contractor's construction operations. It has been left to the contract negotiations for this contract whether the bridges shall be maintained as permanent bridges after completion of the Tamakoshi V HEP construction.

- Contract 1: It is proposed to construct the most upstream section of the Headrace Tunnel over a length of tentatively 1,000 m lined only with shotcrete and concrete invert segment. The final decision on this shall be taken after the respective tunnel section has been excavated, and the in-situ rock conditions examined. The decision shall also include the appraisal whether the shotcrete lined tunnel section shall be shortened, extended and/or relocated. In conjunction with the final decision on the shotcrete lined section of the HRT it shall also be decided whether a rock trap shall be constructed upstream from the pressure shaft in the close vicinity of the Adit 4 pressure door.

- Contracts 2 & 3: Under the aspect of power supply security it has been foreseen that electric power to the Permanent Camp will be supplied during the construction period from a diesel generator unit (DGU) to be provided under Contract 3. In the initial period the DGU shall be operated by the contractor of Contract 3; later operation shall be handed over to Contract 2 until this contractor has constructed the 11 kV line from the Power Station to the camp, and Tamakoshi V is operational to supply the camp wit power. Alternatively, however, power supply to the camp during the construction period could also be arranged via an 11 kV line from Singati to the camp; part of this line would have to be dismantled at the end of the Tamakoshi V construction. It has been left to the contract negotiations for the Contracts 2 and 3 which concept for the power supply to the camp shall finally be followed.

13.6.4 Construction Methodology

The tunnels, adits and caverns along with their corresponding quantities of excavation and the required materials for rock support and concrete linings etc. are presented in Annex A to Part C of this report. The geometry and layouts of the above structures are presented in the Album of Drawings, Part E of the Detailed Design Report.

13.6.4.1 Tunnels

The Head Works, the Tailrace, the High Pressure Tunnel & the U/S Manifolds, the D/S Manifolds, the Bus Duct Tunnels the Service Tunnels and the Adits, will require large volumes of underground excavation which is a time consuming activity and must be started as early as possible. Much of this work can start and proceed concurrently. However, access to the various work faces will require careful planning. The Consultant has proposed two new temporary bridges across the Tamakoshi to provide access to Adits 2 and 3. Construction of the bridges, together with the corresponding access roads, have highest priority.

Excavation

Excavation will be done using the drill and blast technique.

Heading will be done using two boom drill jumbos. Benching will be carried out by drilling a pattern of vertical holes using track-drills. Cycle time includes drilling, loading of explosive, blasting, defuming, mucking, shotcreting, rock bolting and ribbing etc. Blasting will be done using conventional explosives. As the heading is driven, both initial and final rock support will be installed. Support measures will comprise, as necessary: rock bolts, shotcrete, wire mesh, steel sets and drainage holes. Advance rate depends on the type of rock. The advance rate is based on three shifts per day. This advance rate has been adopted after reviewing data derived from standard guides as well as ongoing projects of similar nature within Nepal.

The equipment planned for excavation, are drilling jumbos, crawler drills, 2.5 m³ bucket wheel loaders, 20/25 ton multi axle rear dump trucks, jackhammers, compressors shotcrete m/c, etc.

One set of equipment will be utilized for each face of the tunnel excavation.

The excavation and concreting of portals can be carried out simultaneously to the works in the tunnels, adits and caverns.

The start and completion dates of each structure are presented in the Construction Schedule, which can be found in the Album of Drawings, Part E of the Detailed Design Report.

Concrete Linings

Concrete linings will be required for the Connecting Tunnel walls, Headpond walls, Spillway Tunnel walls, Headrace Tunnel (including U/S Manifold), Surge Tank and Tailrace Tunnels (including D/S Manifold), Outlet Structure Shaft, Outlet Structure Tunnel. Equipment planned for concreting the lining are 38 m^3/h concrete pump, 6.0 m^3 transit mixer, two 60 m^3/h capacity batching and mixing plants, etc.

The start and completion dates of each structure are presented in the Construction Schedule and can be found in the Album of Drawings, Part E of the Detailed Design Report.

13.6.4.2 Caverns

Powerhouse Cavern

Excavation

An existing adit, which later will be widened to become the Cable and Ventilation Tunnel, will be used for the initial excavation of the Powerhouse Cavern. The top heading will be completed over the full length of the crown of the Powerhouse. This is then widened by slashing to the cavern's full width. Both initial and permanent crown rock support, comprising of systematic rock bolting and shotcrete, will be applied as the excavation proceeds. An additional bench will be excavated to allow for drilling for the prestressed rock anchors required to fix the temporary concrete crane beam. A 2x10/6 ton temporary crane for the Powerhouse will be installed. This crane will be then used during excavation and construction activities.

Excavation then proceeds using benching and with the help of internal ramps. Mucking-out will initially be through the Cable and Ventilation Tunnel and then, subsequently, as excavation progresses, through the Main Access Tunnels. The lowest parts of the cavern for the draft tubes will be excavated by mucking-out through the Tailrace Tunnel.

The equipment planned for excavation will similar to the equipment used for the tunnel works.

The start and completion dates of each structure are presented in the Construction Schedule and can be found in the Album of Drawings, Part E of the Detailed Design Report.

Concrete Works and Installation of M&E Equipment

Once the rock excavation is complete, concreting of the Powerhouse Cavern, and subsequent installation of the M&E installation, can proceed.

The first stage concrete forming the outer load-bearing structure of the unit blocks is then constructed up to the main crane beam. Once the first stage concrete for Powerhouse is completed, the 80/15 ton Powerhouse crane can be installed. The draft tubes sections are then transported to the Powerhouse and are welded together. These sections can be brought in through the Tailrace Tunnels on low bed trailers and positioned in place with chain hoists. The space is then concreted in stages, filling the void between the steel and the rock face. The spiral case is then installed, welded and concreted. Then follows erection of the generator stators followed by installation of the rotors. Installation of the various services and controls proceeds at the same time. As the construction proceeds upwards, construction of the Valve Floor, Turbine Floor, Generator Floor, Machine Hall Floor, Administration Floor and Crane Runway floor. will be completed along with installation of the corresponding M&E equipment.

Equipment proposed for concreting are 38 m³/h concrete pump, 6 m³ transit mixer, concrete vibrators, two 60 m³/h batching plants for conventional concrete and shotcrete along with an aggregate processing plant would be installed near the Powerhouse to serve the concreting works.

The start and completion dates of concreting of the Powerhouse structure, M&E equipment installation, including second stage concrete and testing and commissioning of M&E equipment are presented in the Construction Schedule and can be found in the Album of Drawings, Part E of the Detailed Design Report.

Transformer Cavern

Excavation

Underground excavation will be done in a similar way to that in the Powerhouse Cavern using the Cable and Ventilation Tunnel near the crown of the Transformer Cavern and then excavating downwards by heading and benching. Mucking-out will initially be through the Cable and Ventilation Tunnel and then, subsequently, as excavation progresses, through the Main Access Tunnels. The concrete structure will have to be essentially complete prior to installation of the equipment. Permanent rails will be provided to move the transformers in place. A 5-ton mobile crane will be installed in the Transformer Cavern. This crane will be then used during excavation and construction activities.

Concrete Works and Installation of M&E Equipment

The concrete structure will have to be essentially complete prior to installation of the equipment. Permanent rails will be provided to move the transformers in place.

The start and completion dates of the excavation works, concreting works and the installation of the permanent equipment are presented in the Construction Schedule, which can be found in the Album of Drawings, Part E of the Detailed Design Report.

13.6.4.3 Surge Tank & Pressure Shaft

Excavation

The excavation of the 76 m high and 16 m diameter Surge Tank and the113 m high and 5.0 m diameter vertical Pressure Shaft will use the raise bore method for excavation. where a small-diameter hole, called a pilot hole, is first drilled. The construction process of the pilot hole by raise boring machines is the same as that by conventional drilling machines except the back reaming process introduced in the raise boring method. When the pilot hole breaks through into the lower level (Headrace Tunnel and High Pressure Tunnel, respectively), the drill bit is removed and replaced with a large-diameter reaming head. The reamer is rotated and pulled back toward the drilling unit, and through this way a large-diameter shaft is formed. The maximum diameter of back reaming is usually less than 3.5 m. The remaining diameter of the shaft is excavated conventionally. Mucking-out will proceed through Adit 4, in the case of the Surge Tank and the Access to U/S Manifold in the case of the Pressure Shaft.

Concrete Works

The Surge Tank will have a concrete liner (50 cm thickness). Slip forms are proposed to speed productivity.

The start and completion dates of the excavation works, concreting works are presented in the Construction Schedule, which can be found in the Album of Drawings, Part E of the Detailed Design Report.

13.6.4.4 Installation of the Steel Liners of the Pressure Shaft, the High Pressure Tunnel, the U/S Manifold

The pressure tunnel subdivides into four manifold branches. i.e. the High Pressure Tunnel delivers water to four turbines. It is foreseen to transport the Cans through the Access to the High Pressure Tunnel (a branch of the MAT). The first Can will be positioned at the top of the lower bend of the Pressure Shaft (El 988.505). After alignment and back filling with concrete this Can will act as a "fixed point Can" After two or three Can placements D/S of the "fixed point Can", Cans can be positioned above the "fixed point Can". These Cans will be transported from the Tailrace Tunnel and/or the Access to the Valve Chamber (Adit 4 during construction) and lowered to the first Can ("fixed point Can"). In parallel, the Manifold Can placements can proceed, working U/S toward the Access to U/S Manifold. Except for the transition pieces of the manifold all Cans will involve only circumferential welds. Transportation from the manufacturing yard will be on a modified flat-bed trailer

The times presented in the Construction Schedule are based on the transport, placement, welding, backfilling and grouting of the Cans. Time was also considered in the schedule for concreting behind the liner, contact grouting, surface preparation and painting. However, it also assumes the use of concrete pumps, transit mixers, concrete vibrators, etc.

The start and completion dates of the installation and welding of the steel liners are presented in the Construction Schedule, which can be found in the Album of Drawings, Part E of the Detailed Design Report.

13.6.5 Construction Power

Electric power for construction was assumed to be provided by the contractors independently. Alternatively, the power could also be acquired from the nearby small hydro plant at Siprin Khola; however, with respect to this alternative the reliability of supply needs to be verified, and the contractor would have to make respective arrangements for the off-take of power.

The primary consumer for construction power will be the contractor for the civil works. His power provision has been estimated in the course of construction planning to be arranged through separate decentralized generators:

- 6 generators of 550 kVA capacity each will be placed at the portals of the adits along the HRT, the MAT portal and the portal of the adit to the TRT. These generators will provide power for ventilation, pumping, lighting and operation of the construction equipment at the work fronts.
- 2 generators of 330 kVA capacity will be operated at the two batching plants to provide power for aggregate processing and concrete production.

The power demand of the contractor for the electro-mechanical works is expected of secondary order when compared to the above capacities. The following power demand has been estimated on the basis of experience made at other projects:

- A total of about 300 kVA capacity will be required for the installation of the E&M equipment in the Power Station area. This demand will almost exclusively be associated with the assembly activities for the generating units.
- Increased power supply will only be required during times where corrosion protection work is ongoing in the power waterways. This demand is estimated at 750 kVA and results from simultaneous operations of air conditioning, cooling, de-dusting, compressors for sand blasting, winches, etc. The corrosion protection works will likely require special high-current plugs and connectors at the work site.

It has been assumed that this contractor will make his own arrangements to cope with his power demand; however, it may also be possible that he can cover his demand from the generator capacities of the civil works contractor as well.

The power demand related to the contractors' camps cannot be estimated accurately, as this depends to a wide extent on the contractors' camp planning. Based on a requirement of 16 A supply per 100 m² of floor area of the building at 220 V level and a concurrency factor of 0.75 a demand of about 200 kVA can be expected. Since the concurrency factor is set at rather high level, the above value should suffice to cover also common consumers like street lighting, etc.

14 PROJECT COSTS

14.1 Introduction

The estimate of project costs includes the costs of engineering, construction, equipment, environmental mitigation and security. A hydropower project like Tamakoshi V requires the construction of large civil structures and underground openings and the manufacture, delivery and erection of heavy mechanical and electrical equipment.

This chapter describes the following:

- Basic market rates for different materials, construction equipment and consumables.
- Market rates for labor.
- The detailed cost estimates.
- Cash flow for the project.

14.2 Project Cost Summary

The estimated project base cost is summarized in equivalent USD as follows:

Table 14-1: Project Base Cost is Summarized in Equivalent US Dollars

Item No.	Description	Foreign USD	Local NPR	Total USD
1	Civil Works	39,291,329	4,032,598,206	75,258,134
2	Hydromechanical Equipment	8,340,800	-	8,340,800
3	Mechanical Equipment	10,826,400	-	10,826,400
4	Electrical	23,831,600	-	23,831,600
5	Camp Works	-	4,173,896	4,173,896
6	Transmission Line	1,900,800	-	1,900,800
7	Owner's Costs	5,577,807	292,922,468	8,190,387
	Total Base Cost			**132,522,017**

14.3 Price Level and Rate of Exchange

The estimate is based on 1st Quarter 2019 price levels and was consolidated in US dollars. The exchange rate used was USD 1.00 = Nepalese Rupee (NPR) 112.12.

14.4 Scope of the Project Base Cost

The costs of all the works required to complete the project fit for its purpose was included in the estimate except for the following expenditures which were excluded from the Project Base Cost:

- Only local prices were used for workmanship. Escalation for labors was not applied since the base unit rate for “unskilled labor” was taken from a recent project.
- Duties and taxes on imported installed equipment and parts.
- Interest during construction (IDC) and other financing charges.

14.4.1 Project Cost

Project Cost Components

The project main costs are those directly associated with the main project components and include the cost of all structures, power generation and transmission facilities. The project main costs were subdivided as follows:

(A) The Civil Works for Tunnels & Access Roads, including:

- Tunnel Works
- Surface Works & Access Roads
- Auxiliary Prices for Civil Works (common expenses)
  - Technical Personnel at Site
  - Service Staff
  - Equipment & Facilities at Adits
  - Project Monitoring & Surveying
  - Vehicles for General Usage

(B) Permanent Camp

(C) Permanent Equipment, including:

- Electrical equipment,
- Mechanical equipment,
- Hydro-mechanical equipment,

- Transmission line equipment.

(D) Owner's Costs/Ancillary Costs (calculated as a percent of the sub-totals of the above items)

- Engineering Management (4%)
- Resettlement & Environment (1%)
- Owner's Expenses (2%)

Two additional costs were added to the sub-totals of the above amount;

- Miscellaneous Cost and
- Contingency Costs.

Miscellaneous costs refer to costs of items that are too small to list separately, for example elastomer-supports, movement joint seals, water stops etc.

Contingency costs arise during construction to cover physical contingencies including changes to the layout (but excluding fundamental changes) and unexpected ground conditions.

These Miscellaneous Costs and Contingency Costs were calculated as a percentage of a determined amount. The following table summarizes the percentage applied.

Table 14-2: Percentages used to Calculate Miscellaneous Cost and Contingency Costs

Item No.	Description	Percent calculated for Miscellaneous Costs	Percent calculated for Contingencies
1	Tunnel Works	3	5.0
2	Surface Works & Access Roads	3	4.5
3	Auxiliary Prices for Civil Works	3	3.5
4	Project Management Prices for Civil Works	3	2.5
5	Permanent Camps & Roads	3	5.0
6	Hydromechanical Equipment	3	1.0
7	Mechanical Equipment	3	1.0
8	Electrical Equipment	3	1.0
9	Transmission Lines	3	5.0

For the Owner's costs no Miscellaneous Cost and Contingency Costs were calculated

14.4.2 Currency Breakdown

The costs are broken down into local and foreign currency components. This breakdown takes account of the source of origin of the labor, materials, equipment or the services involved.

The resulting percentages were derived through detailed cost analysis and are shown in the table above.

14.4.3 Basic Assumptions and Limitations

The cost estimate was developed based on the following assumptions:

- The total project completion time from the start of the main works to completion of construction of the project will be 4 years (48 months). Please refer to the Construction Schedule.
- The economic conditions, both in Nepal and worldwide, will remain essentially stable.
- The project layout and work quantities as defined in this Detailed Design Report will not radically change (reasonable variations are covered by contingency allowances).
- Specifications and general contract conditions will be in accordance with international standards and will not be unduly restrictive. These will include an equitable distribution of risk, mobilization fees, contract price adjustment formulae and adequate provisions to maintain positive cash flows for the contractor.
- The project will be offered under international competitive tendering conditions, and an adequate number of firms will be invited to tender to ensure fair competition.
- All expatriate supervisory personnel and the equipment that are required for execution of specialized works such as underground excavation, M&E installation, etc., will be allowed to be imported.
- All necessary permits for the right-of-way, access to the site will be guaranteed and secured by the Owner.
- Resettlement and other preliminary activities will be completed by the Owner without affecting the scheduled key milestone dates.

14.4.4 Construction Procurement

It was assumed that the project work will be procured in three contracts. The title of these contracts are for identification purposes and do not reflect the scope of work for each contract.

Contract 1: Civil Works and Hydromechanical Equipment

- All access roads for the entire Project are included in this contract except for:
  - the access roads for the Transmission Lines and
  - permanent roads from the public road (Singati-Lamabagar) to the Permanent Camp and

  - permanent roads within the Permanent Camp

- All civil works for the entire project except for:

  - civil work required for the Transmission Lines and

  - civil work required for the Permanent Camp

- All hydromechanical, mechanical and electrical equipment outside of the Powerhouse and Transformer Caverns for the entire project. Excluded is the power take-off cables to the Terminal and Ventilation Building.

- The welded connection between the U/S manifolds and the extension pipes of the main inlet valves. This welded connection is the U/S limit between Contract 1 and Contract 2. The drawing grid axis “D” is the D/S limit.

- All other activities and equipment, required outside the Powerhouse and Transformer Caverns and the Permanent Camps and Roads, shall be included in the Contract 1.

Contract 2: Mechanical, Electrical Equipment and Transmission Lines

- Access roads for the Transmission Lines

- All civil works for the Transmission Lines

- All hydromechanical, mechanical and electrical equipment inside of the Powerhouse and Transformer Caverns for the entire project. Included is the power take-off cables to the Terminal and Ventilation Building.

- The welded connection between the U/S manifolds and the extension pipes of the main inlet valves is not part of Contract 2 and serves as the U/S limit of the works. The drawing grid axis “D” is the corresponding D/S limit.

Contract 3: Permanent Camps and Roads

Civil works, permanent roads, mechanical equipment, electrical equipment and all other activities and equipment required within the Permanent Camps

The procurement procedures of the donor agencies will necessarily have to be followed. This may entail revisiting the number of contract packages and time frame for award of the contracts.

14.4.5 Development of Cost Estimate for Project Civil Works

Direct civil construction costs for each of the main project structures were determined by assessing the quantity of each of the basic resources – manpower, equipment, materials and consumables – required to construct the respective structure according to prevailing efficient and safe working methods. These quantities, multiplied by the respective unit costs of basic resources at site and summed over construction task and project structure, result in the total direct construction cost. The basic resources are detailed below:

“**Wages**” are the costs of local/foreign labor required on the project.

“**Equipment**” covers depreciation, or the decline in value through general wear and tear and the financing charges to recover investment cost of construction plant and equipment. Machines for short usage will be rented

“**Permanent Materials**” are the costs of materials that remain in the project (cement, steel, wire mesh, rock bolts, etc.).

“**Job Materials**” are the costs of materials that are consumed through construction of the project (explosives, drill tools, formwork, equipment spare parts, tires, lubricants and fuel, etc.).

A major part of the above resources is available in Nepal which are then assigned a local cost, but some are sourced from abroad and are thus assigned a foreign cost. All cost estimates are made in dollars (USD) for both the foreign currency component and the local currency component. The conversion rate is stated under Subchapter 14.3 above.

14.4.5.1 Civil Works’ Quantities

Quantities of civil works have been measured from the detailed engineering design level drawings. Where possible, the quantities have been taken off using AutoCAD techniques; otherwise the quantities have been established by manual computations and previous experience.

The principal structural dimensions were defined in the corresponding drawings.

To consider miscellaneous quantities that cannot be measured accurately, allowances of 5 to 15% was added to the quantity estimates.

For underground work a “payment line” was applied.

14.4.5.2 Selection of Construction Methods

The cost estimate was developed based upon the utilization of conventional and proven construction methods that can be realistically applied in Nepal for construction of the project.

14.4.5.3 Open Cut Excavation

The cost estimate assumes that all open cut excavation will be executed by loaders with 3 m³ buckets and loaded on to 20-ton dump trucks. Drilling and blasting operations were assumed to be performed using a hydraulically controlled mining and tunnelling face drilling rig with two booms and conventional explosives.

Production rates for open cut excavation were assumed based on experience on similar projects.

14.4.5.4 Underground Excavation

The excavation for the tunnels and caverns was assumed to be performed using conventional drilling and blasting techniques and associated equipment.

Underground advance rates were based on calculations considering drilling, explosive, dust dispersions and mucking times for each rock type shown in the drawings and from experience on similar work. The construction schedule allows enough time for crossing unforeseeable faults and difficult ground conditions.

14.4.5.5 Manufacture of Processed Materials

The cost estimate assumed that all processed materials for both coarse and fine concrete aggregates and filters would be obtained mostly from river excavations and some from tunnel excavation.

For estimating purposes, it was assumed that a crushing and screening plant 40 m^3/h would be required to provide aggregates for both the conventional concrete and shotcrete operations.

14.4.5.6 Conventional Concrete

The cost estimate assumed that the conventional concrete would be manufactured using two batching plants each with a capacity of 60 m^3 per hour each.

14.4.6 Construction Schedule

A construction schedule was developed considering the logical sequence and interdependence of all main construction activities. This schedule is included in Part E, Drawings Album. This schedule comprises the total project break down into its main features. Scheduled activity sequences and durations were determined based on production rates that can be reasonably achieved by use of the plant and equipment that was selected for the purposes of developing the cost estimate.

14.4.7 Basic Cost Data

14.4.7.1 Labor

The wage rates prevalent in town and other areas of the country were obtained for both unskilled and skilled labor. Employment conditions governing such matters as work hours, overtime, leave and holidays were patterned on those developed by Upper Tamakoshi Project with minor modifications reflecting the passage of time.

14.4.7.2 Construction Plant and Equipment Ownership and Hourly Operating Costs

Unit hourly use rates for depreciation, operation and maintenance of the construction plant and equipment were derived based on rates derived from machine and equipment catalogs, site conditions and production rates of the machines. Appropriate adjustments were made to consider cost of shipping, parts warehousing, and other local factors such as the cost of labor and fuel.

It has also been assumed that the resale or salvage value of the equipment may compensate for expenses incurred during mobilization. Insurance and financing charges were included in the unit rate cost calculations.

Specialized processing plants to be used for manufacture of aggregates, batching and mixing conventional concrete as well as the drill jumbos for underground works, were priced based on site and rock conditions and manufacturers' quotations or purchase cost of similar machinery used on other projects. Adequate transportation and erection charges were added to the purchase price and the ownership expenses were expressed as cost of depreciation per unit of quantity of work, and not based on expected operating lives. Only labor, spare parts and consumables were computed on an hourly basis.

14.4.7.3 Fuel and Lubricants

High speed diesel, regular and premium gasoline and various lubricants have been priced based on Nepal rates prevailing in the 3rd quarter of 2018. Gasoline, lubricants and regular oil changes were estimated as 20% of the diesel consumption. The policy of price regulatory authority of said sector and time fluctuations along with transportation to site and storage costs, etc. have also been considered in the cost estimate.

14.4.7.4 Material and Supplies

It was assumed that most of the required materials will be available from sources in Nepal. The only significant items that will have to be imported are drill bits, rods and other drilling consumables. The items were considered as 10% of the drilling expenses. Rates for these items, therefore, are already include the custom duties and taxes.

The cost of the materials consumed for each item of work was considered for the calculation of the unit rates. A data bank comprising prices of various materials and supplies to be used for estimating the cost of construction works was established. The cost of these materials and supplies also covers the delivery of these supplies to the site. These materials include local as well as foreign currency purchase. The supply of cement, reinforcing steel, structural steel, etc. is included in the local currency purchases. Small tools and supplies such as safety clothing, hand tools, small power tools, hardware, ladders and welding plant were considered at different rates. The specific rate corresponded to a particular activity.

Unit prices for the major materials include transportation to the site, handling charges and wastage allowances.

14.4.7.5 Duties and Taxes

Costs relating to duties and taxes on imported construction equipment and installed plant were excluded from the project base cost.

14.4.8 Civil Work Cost Components

The estimate for the civil work was developed considering separately the contractor's direct cost and the contractor's overhead and general expenses.

The contractor's direct costs are directly related to the work activity and vary in relation to the quantity and production rate whereas the contractor's overheads and general expenses are project and time related.

14.4.8.1 Civil Contractor’s Direct Costs

The contractor’s direct costs for the civil work items were computed in several ways. The cost of all the work items that have a significant impact on the total cost was evaluated in detail.

The direct unit rates for the major items of civil work such as those related to placement of conventional concrete, underground excavation, and open cut excavation, were derived in detail taking into consideration the labor crew, materials, construction equipment composition, and production rates. The direct unit costs were broken down into their components of labor, materials, and construction equipment (see annexes to Part D1 of this report).

Historical Costs

Unit costs for minor civil work items were derived from properly adjusted historical records.

For comparison purposes, historical in-house data from previous projects, formulae and experience curves were also used to check estimated costs for the major civil works.

Lump Sums

Work which combine several minor cost items; have been shown under single headings as lump sums. In most cases, those items have been calculated analytically or obtained from historical data.

Quality Control

For special production items such as concrete manufacture, rock bolts etc., 1.5% was added to the unit rates to cover quality control.

Special Items

The Powerhouse and Transformer Caverns, Headworks, Headrace Tunnel and Tailrace Tunnel and the other underground excavations were analyzed in detail considering the latest equipment and most economical and efficient methods of construction in use for execution of such work.

Miscellaneous Items

The estimate includes an allowance for unquantifiable and unforeseen miscellaneous items, such as excavation over-break, water stops, lean concrete, bedding mortar, anchor bolts, special finishes, miscellaneous steel work, minor equipment, contact grouting, steel ribs, etc. (see Annex A to Part D1 of this report)

All such unquantifiable unforeseen miscellaneous items have been labeled and accounted globally as “Miscellaneous”. Depending on the type of work, the allowances for these items was 3% of the corresponding section subtotal.

14.4.8.2 Civil Contractor’s Indirect Cost

To account for “indirect costs” which are those costs not directly related to the actual scope of works, but which are necessary for the performance of the overall project. They cannot be allocated to individual works. These costs are detailed and presented in Annex E to Part D1 of this report.

14.4.8.3 Civil Contractor’s Overheads (On-Costs)

The contractor’s overheads (or “on-costs”) comprise various fees and charges and a margin for his profit and risk. These “on-costs” are estimated as percentages of the sum of the contractor’s “direct costs” and his “indirect costs”. The percentages are as shown in the table below:

These costs are covered in Part D2, Economic and Financial Analysis

14.4.8.4 Unit Rate Analysis for Civil Work

The unit rates used for most of the main civil works items have been computed based on resources deployment, in accordance with the following procedure:

- Each item was subdivided into a set of elementary operations.
- Construction methods, equipment/plant and progress rates were defined for each of these operations.
- The type and quantity of all input resources needed (labor, equipment and materials) were established and priced.
- The direct cost of each item was obtained by summing the amounts required for each operation and then dividing by the total work quantity involved.
- Unit rates consist of only the bare costs required to complete that item of work; the contractor’s indirect costs and on-costs are added to the direct costs.

Annex B to Part D1 of this report contains a summary of the rate analysis for Tunnel Works, and Annex C to Part D1 of this report contains a summary of the rate analysis for Surface Works. Also included are:

- Base Rates for the rate analysis (apply to Tunnel and Surface Works)
- Hourly rates for heavy and light equipment (apply to Tunnel and Surface Works)
- Hourly rates for unskilled and skilled labor in the Dolakha District (apply to Tunnel and Surface Works)
- Unit Process description of each activity

14.4.9 Development of Cost Estimate for Permanent Equipment

14.4.9.1 Turbines, Generators and Transformers

It is assumed that a major portion of the mechanical equipment will be imported from foreign sources. The costs illustrated in Annex A to Part D1 of this report for these components have been estimated from the consultant’s own data, from manufacturers’ quotations and from recent international bid prices.

The prices include profit, overheads and the cost of designing, purchasing raw materials, manufacturing, painting, testing, packing the products, delivering to the port of export, ocean freight, insurance, landing cost, inland transportation in Nepal and installation. A contingency amount of 1% of the total costs of all mechanical equipment has been added to cover the additional costs arising from increases in equipment dimensions due to unforeseen physical conditions at site. Unforeseen site conditions have the greatest impact on civil works, but they can also impact, to a lesser extent, on the dimensions and costs of equipment.

14.4.9.2 Hydraulic Steel Structures

It is assumed that a significant portion of the hydraulic steel structures will be imported from foreign sources. The costs for these structures have been estimated from the consultant’s own data, from manufacturer’s quotations and from recent international bid prices.

The prices include profit, overheads, and the cost of designing, purchasing raw materials, manufacturing, painting, testing, packing the products, delivering to the port of export, ocean freight, insurance, landing cost, inland transportation in Nepal and installation. A contingency amount of 1% of the total cost of all hydraulic steel structures has been added to cover the additional cost arising from increases in equipment dimensions due to unforeseen physical conditions at site. The cost estimate of the various items of hydraulic steel structures is presented in Annex A to Part D1 of this report.

14.4.9.3 Permanent Miscellaneous Electrical and Mechanical Equipment

To determine the cost of electrical equipment and the control system, suitable power plant equipment configurations covering all key components from low to high voltage as well as the control and protection systems were elaborated.

The corresponding costs were determined in a bottom–up approach summarizing the individual components and common systems. Cost information was taken from company internal database or determined by extrapolation from comparable Pakistan reference projects. The cost estimate of the various items of electrical and mechanical equipment is included in Annex A to Part D1 of this report**.**

14.4.9.4 Transmission Lines

The transmission lines cost in Annex A to Part D1 of this report was estimated based on contract prices of existing projects and prices obtained from manufacturers. The construction costs have been estimated considering the difficult terrain and access involved. Cost of towers and foundations were estimated based on conductor loadings.

14.4.10 Development of Cost Estimate for Permanent Camps and Roads

Costs have been estimated based on a detailed design level of design for the several permanent structures and roads detailed in Part A3 Chapter 10 Permanent Camps and Roads.

14.4.11 Environment and Resettlement

The cost for mitigation of the environmental impact and resettlement was estimated as 1% of the sub-amount of the project (without miscellaneous and contingencies). The costs are presented in Annex A to Part D1 of this report**.**

14.4.12 Project Management for Civil Works

Based on experience from previous projects, allowances for different project site civil management components are presented in Annex D to Part D1 of this report. The costs include the Technical Personnel foreseen for the Tamakoshi V project as well as the following:

- Service Staff
- Vehicles
- Diesel
- Facilities
- Stationeries/Computers/Printer/Plotter
- Clinic
- Personal Protection Equipment for technical staff
- Kitchen/Canteen
- Exp.for Main Mng. & Main Office
- Official Fees
- Insurance
- Guarantee payments

Table 14-3: Area of Works, Engineering and Project Management Expense

Area of Works	Engineering & Project Management Expense (%)
Civil Works	11.12 calculated
M & E Equipment	included in equipment costs
Hydraulic Steel Structures	included in equipment costs

Transmission Lines	included in equipment costs
Environment & Resettlement	included in environment & resettlement costs

14.5 Currency Breakdown

The breakdown of the costs for different structures and types of work into local and foreign currency is presented in Annex A to Part D1 of this report.

The detailed cost estimates of various components are shown in Annex A to Part D1 of this report.

14.6 Cash Flows

The capital cost estimate for the project base cost was prepared based on the construction schedule that is presented in Part E, Drawing Album. It is anticipated that all power generating units will be in commercial operation within 4 years. The resulting cash flow is shown in Annex G to Part D1 of this report.

15 FINANCIAL ANALYSIS OF TAMAKOSHI V HEP

15.1 Demand-Supply Analysis

15.1.1 Introduction

The historical Gross Domestic Product (GDP) Rate of Nepal from 2015-17 and projections for 2018 and 2019 is indicated in the figure below:

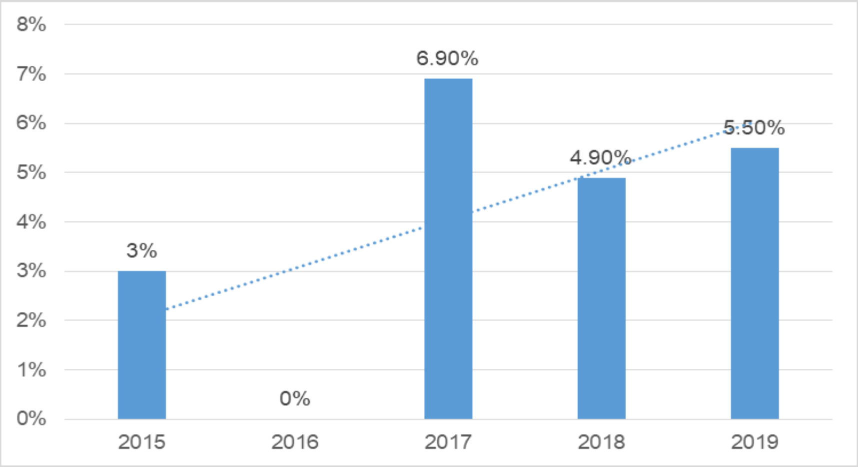

Figure 15-1: GDP Growth Rate of Nepal

Source-https://www.adb.org/countries/nepal/economy

There is an increasing gap between demand and supply of electricity and more than a third of the population does not have access to electricity. The per capital electricity consumption of Nepal in 2015 was 138.08 kWh. The trend of per capita consumption of electricity of Nepal from 1970 to 2014 is indicated in the figure below:

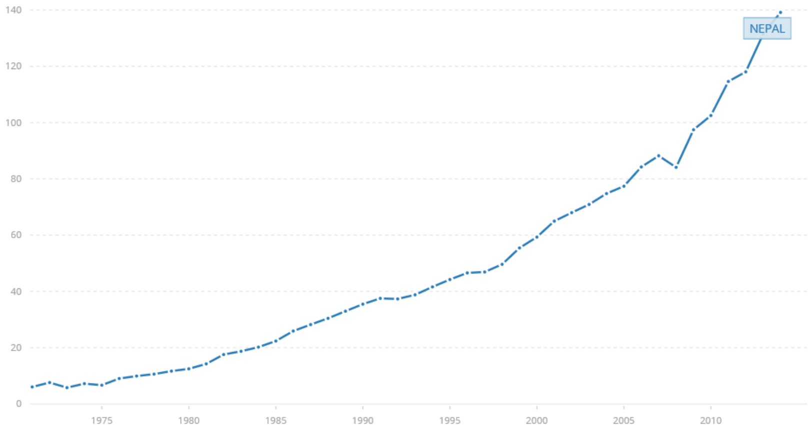

Figure 15-2: Per Capita Consumption of Electricity of Nepal

https://data.worldbank.org

The energy sources in Nepal are categorized as given in the table below

Table 15-1: Energy Sources in Nepal

Source of Energy	Description
Traditional	All types of biomass used for producing energy conventionally
Commercial	Comprises coal, hydropower (except micro-hydro) and petroleum products
Alternative	Biogas, micro-hydro, solar and wind energy

As per the Electricity Demand Forecast Report by Water and Energy Commission Secretariat (WECS), January 2017, total energy consumption in FY 2014/15 was 500 million Giga Joule (GJ). The share of different sources of energy in Nepal is indicated in the following figure:

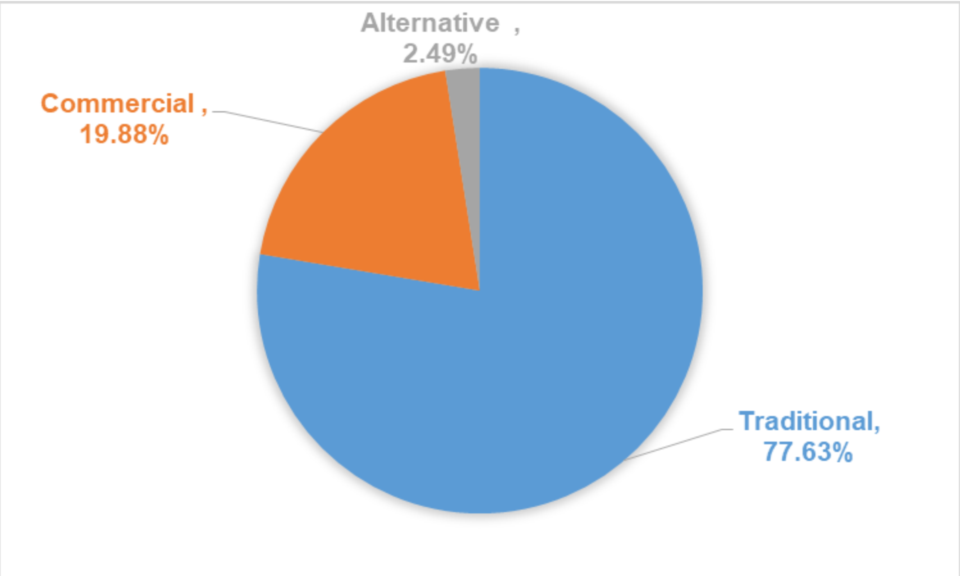

Figure 15-3: Energy Source of Nepal

15.1.2 Energy Demand

The electricity demand depends mainly on the household consumption, industrial activities, and other commercial & economic activities of the country. The total population of Nepal will increase from 28.06 million at the base year (2015) to 39.72 million at the end of 2040 with the growth rate of 1.4%.The urban household will reach 5.7 million in 2040 from 2.7 million in 2015 in other hand rural household will reach 3.9 million in 2040 from 3.3 million in 2015. As per the Electricity Demand Forecast Report by Water and Energy Commission Secretariat (WECS), January 2017, the Electricity Demand from 2015-2040 at different Economic Growth rates (@4.5%, @7.2% and @ 9.2%) is indicated in the figure below:

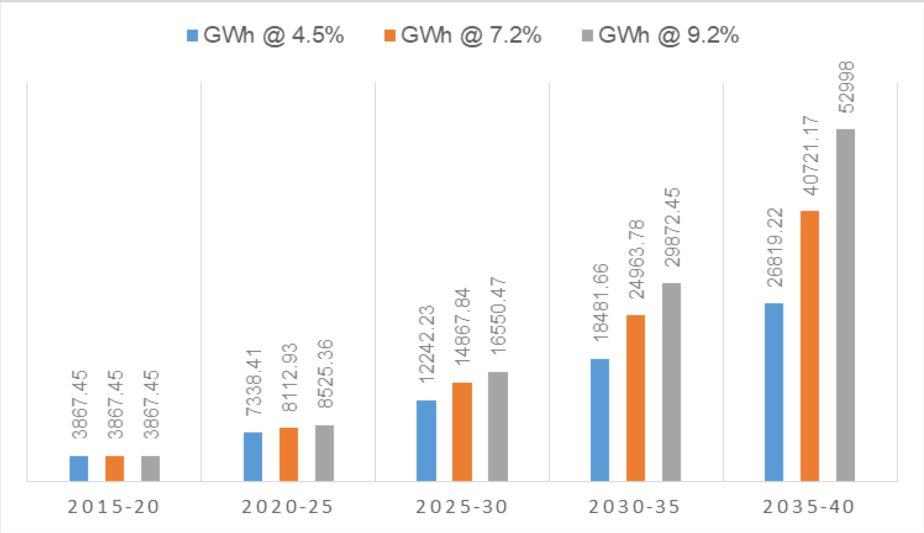

Figure 15-4: Electricity Demand Projection of Nepal

Source- Electricity Demand Forecast Report (WECS), January 2017

The installed capacity requirement was 1,721 MW in the year 2015 against actual installed capacity of 800 MW which resulted in load shedding. The per capital electricity demand will reach to 464 kWh in 2025 and 1536 kWh in 2040 considering 4.5% economic growth rate. The per capital electricity demand and the requirement of installed capacity is indicated in the figure below:

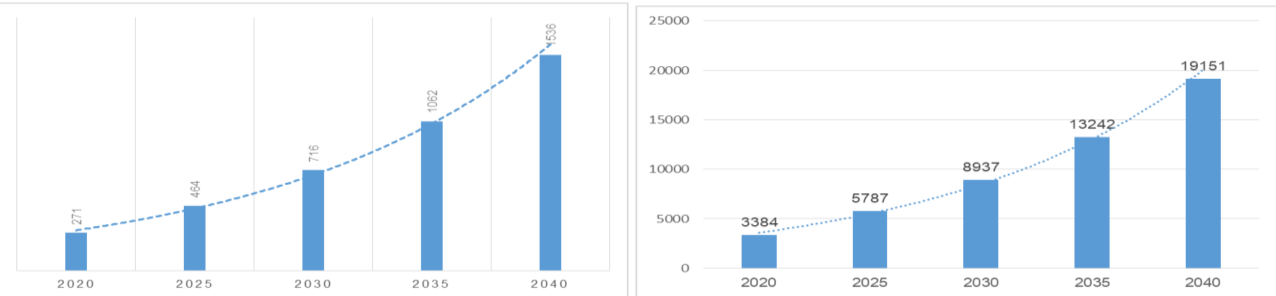

Figure 15-5: Per capital electricity demand and the requirement of installed capacity in Nepal

Source- Electricity Demand Forecast Report (WECS), January 2017

To meet the above per capital consumption, the installed capacity needs to be around 5790 MW in 2025 and 19000 MW in 2040.

As per the Annual Report of NEA 2018, the electricity demand will be 10000 GWh in 2020, 20000 GWh in 2025 and 35000 GWh in 2030. The peak demand shall be 2000 MW in 2020, 4000 MW in 2025 and around 7000 MW in 2030. The eneryg demand (GWh) and peak demand (MW) projection of Nepal from 2020 to 2040 is indicated in the figure below:

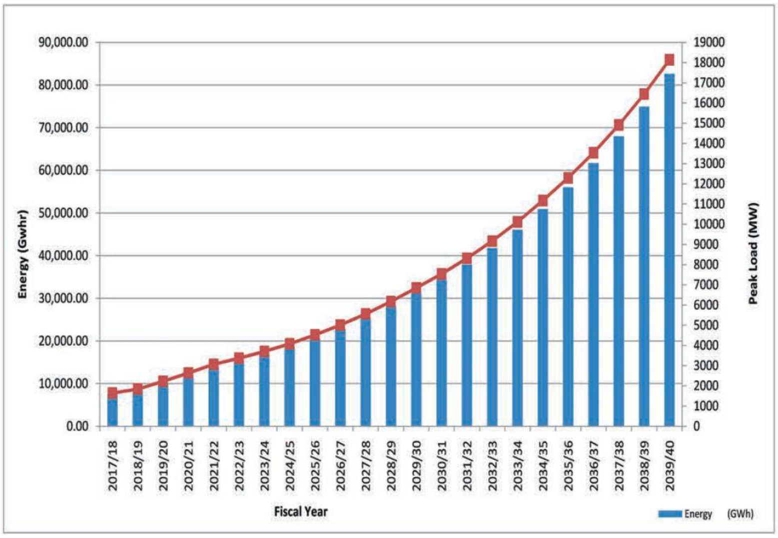

Figure 15-6: Energy demand (GWh) and peak demand (MW)

Source- Annual Report NEA 2018

15.1.3 Energy Supply

The historical total electricity supplied by NEA to different consumers from 2014-18 is indicated in the following figure:

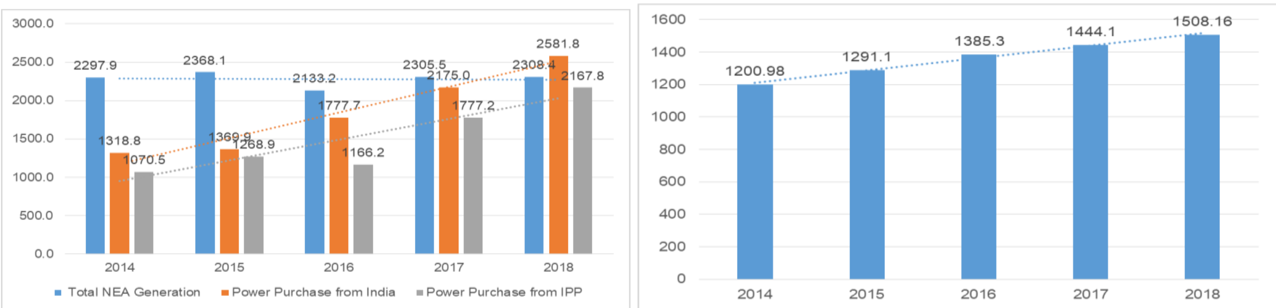

Figure 15-7: Electricity supplied (GWh) and Peak Demand (MW) from 2014-18

Source- Annual Report NEA 2018

There is increasing trend in power purchase from India and IPPs in Nepal. The contribution of IPPs in supply of electricity is 33% in 2018 compared to 23% in 2014. Similarly, there is increase in purchase of electricity from India which is 36% in 2018 compared to 28% in 2014. The various source of electricity for NEA for 2014 and 2018 is indicated in the figure below:

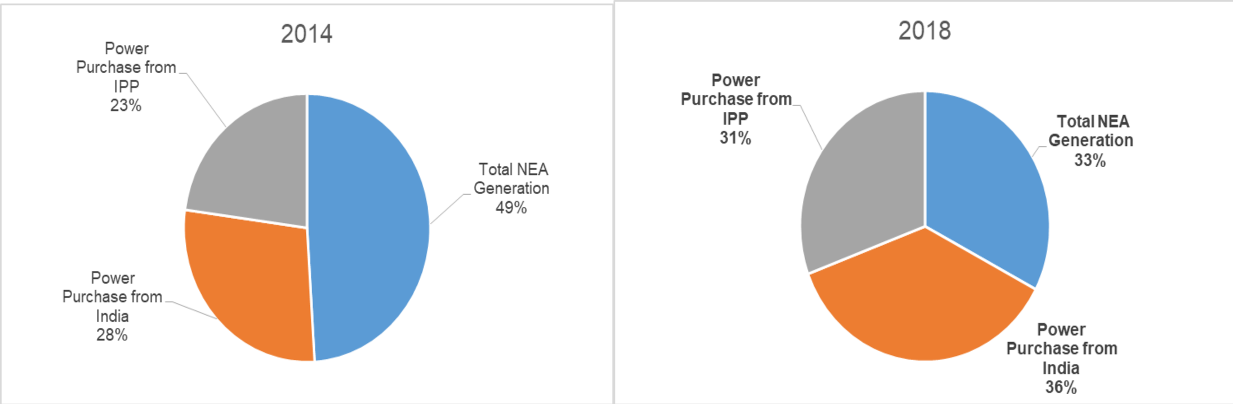

Figure 15-8: Source of electricity for NEA for 2014 and 2018

As of base year 2014/15, Nepal has a total installed capacity of 855.886 MW in which 53.41MW are thermal (multi-fuel) power plants and 100 KW solar plant. The installed capacity of Nepal from 2011 to 2015 is indicated in the next table:

Table 15-2: Installed capacity of Nepal from 2011 to 2015

Year	2011	2012	2013	2014	2015
Total Hydro (NEA)	477,530	477,930	477,930	477,930	477,930
Total Hydro (IPP)	187,581	230589	255,647	255,647	324,446*
Total Hydro (Nepal)	**665,111**	**708,519**	**733,577**	**733,577**	**802,376**
Total Thermal (NEA)	53,410	53410	53410	53410	53410
Total Solar (NEA)	100	100	100	100	100
Total Installed Capacity (Including Private & Others)	**718,621**	**762,029**	**787,087**	**787,087**	**855,886**
Total Installed Capacity	713985	757393	782451	782451	851250

** 2018- The installed capacity is 512,695.40 kW as per NAE Annual Report 2018.*
Source- Electricity Demand Forecast Report (WECS), January 2017

From the above table, it is observed that the total Hydro Project installed capacity is growing marginally from 2011 to 2015. The total IPP Hydro Project installed capacity is growing better than NEA from 2011 to 2015 which is indicated in the figure below:

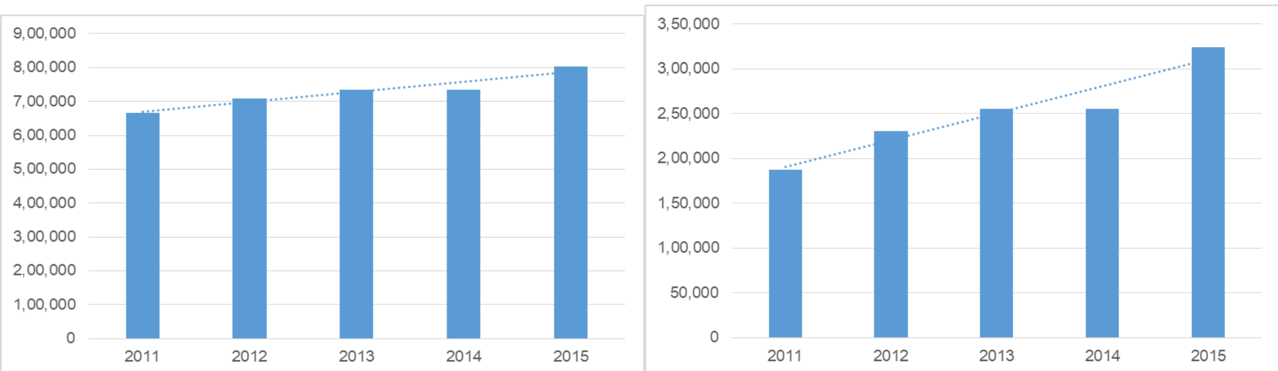

Figure 15-9: Hydro Project installed capacity from 2011 to 2015

The Hydropower potential in Nepal is indicated in the table below:

Table 15-3: Hydropower potential in Nepal

Major River Basins	Theoretical Potential in MW			Technical Potential		Economical Potential	
	Major river courses having catchments areas above 1000 km^2	Small river courses having catchments areas 300-1000 km^2	Total	Number of Project Sites	Technical Potential in MW	Number of Project Sites	Economical Potential in MW
Sapta Koshi	18750	3600	22350	53	11400	40	10860
Sapta Gandaki	17950	2700	20650	18	6660	12	5270
Karnali and Mahakali	32680	3500	36180	34	26570	9	25125
Southern River	3070	1040	4110	9	980	5	878
Country Total	**72450**	**10840**	**83290**	**114**	**45610**	**66**	**42133**

Source- Electricity Demand Forecast Report (WECS), January 2017

As per the Energy Demand Projection 2030: A MAED based Approach published by Investment Board of Nepal, the peak demand shall be 4400 MW in 2020, 7000 MW in 2025 and around 10000 MW in 2030. The Major projects (with installed capacities of over 100 MW) that are expected to come into operation within 2030 are indicated below:

15.1.4 Conclusion

Based on the above Demand Supply Analysis, it can be concluded that there is adequate demand of electricity in Nepal. The power generated from Tamakoshi V HPP with an installed capacity of 100 MW shall enhance energy security of Nepal and will be consumed particularly in Eastern and central Nepal. Commissioning of the project will help bring in industrial and commercial growth in Eastern and central Nepal which faces power shortage presently.

15.2 Tariff Policy and Sustainable Tariff

15.2.1 Introduction

The Sustainable Tariff which is charged to different consumers over a long period of time and the consumers are willing to pay for the use of electricity. As per present arrangement NEA charges different tariffs to different types of consumers based on voltage and current.

The domestic consumers are charged with two part Tariff namely, Service Charge (NRs/unit) and Energy Charge (NRs/unit). The Other Consumers like Industrial, Commercial, Non Commercial, Water Supply and others are charged with two part Tariff namely, Demand Charge (NRs per KVA per month) and Energy Charge (NRs/unit). The Time of the Day (ToD) Tariff rate charged to Other Consumers also includes pick time (17:00 to 23:00), off pick time (23:00-5:00) and Normal time (5:00-17:00).

The historical Consumer Wise Average Electrical Sales (GWh) is indicated in the figure below:

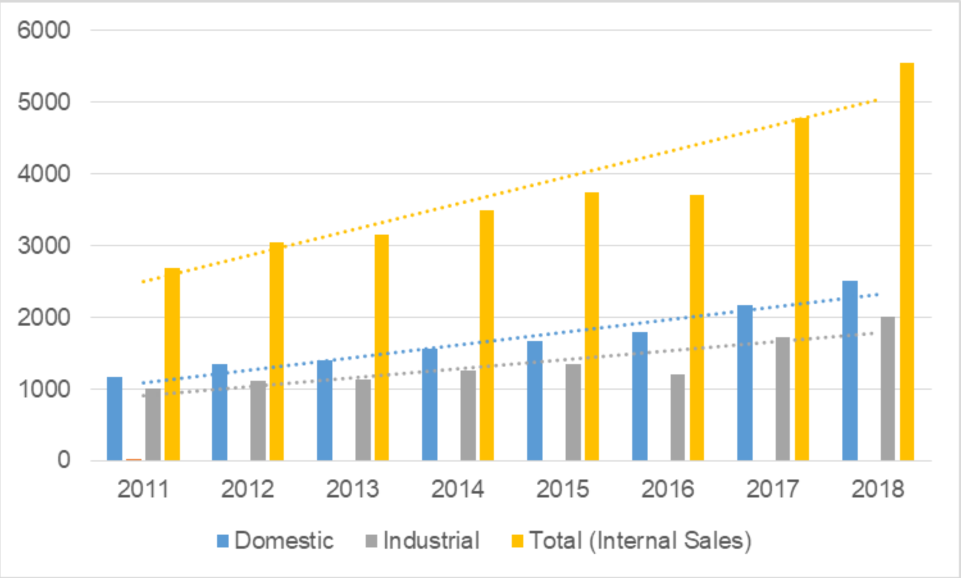

Figure 15-10: Historical Consumer Wise Average Electrical Sales (GWh)

Source-NEA Annual Report 2018

The historical Consumer Wise Average Retail tariff is indicated in the below figure:

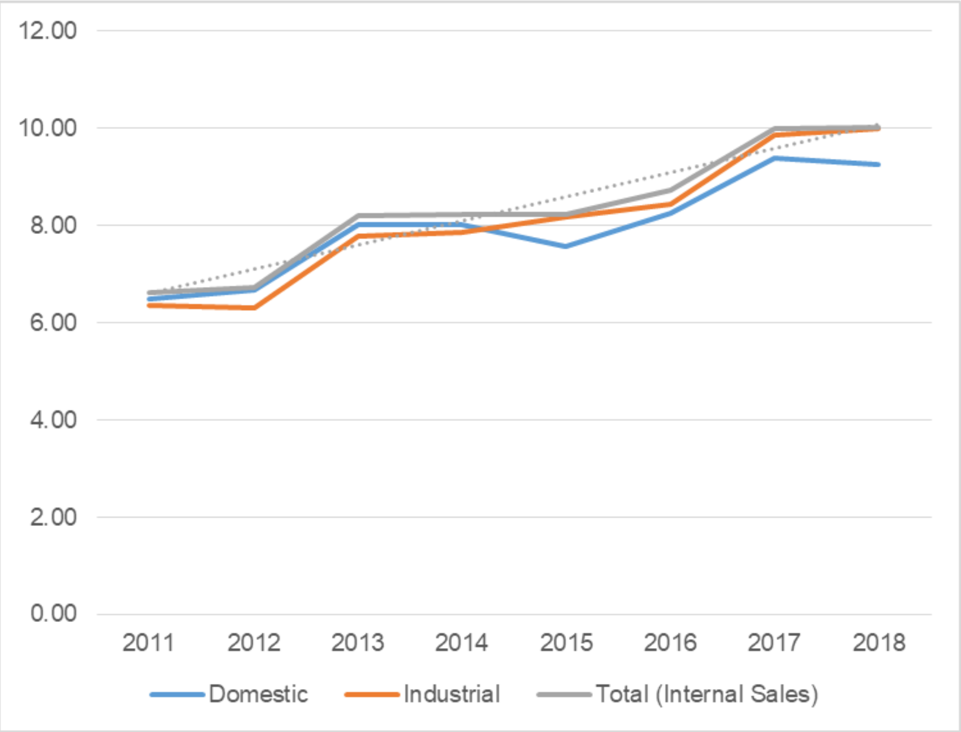

Figure 15-11: Historical Consumer Wise Average Retail tariff (NRs/kWh)

Source-NEA Annual Report 2018

The average Retail tariff is in the increasing trend from 2011 to 2018. The overall average Retail tariff of NEA internal sales and industrial consumers is very close to NRs 10/unit and for domestic consumers, the retail tariff is close to NRs 9.2/unit in 2018.

The average electricity sales growth % and Average Electricity Tariff growth % from 2011 to 2018 is given in the table here below:

Table 15-4: Average electricity sales growth % and Average Electricity Tariff growth %

Growth Rate	Average Electricity Sales growth %	Average Electricity Tariff growth %
Domestic Electricity Sales	12%	5.5%
Industrial Electricity Sales	11%	7.0%
Total (Internal Electricity Sales)	11%	6.4%

Source-NEA Annual Report 2018

The historical Inflation rate of Nepal is indicated in the following figure:

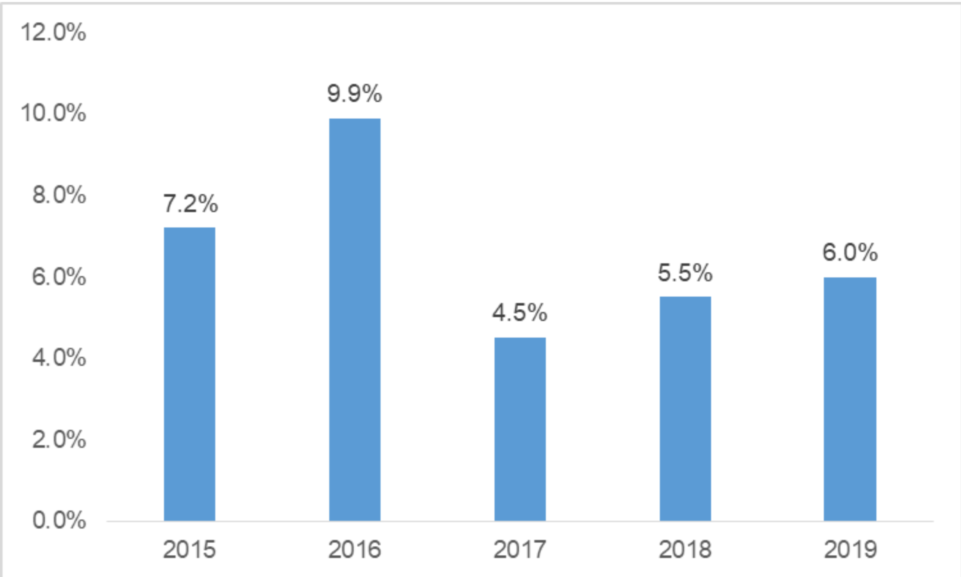

Figure 15-12: Inflation rate of Nepal

As per the NEA Annual Report 2018, the average purchase cost of electricity by NEA was NRs 7.1/kWh both from India and IPPs and the overall system loss was 22.45%.

15.2.2 Sustainable Electricity Price

The tariff structure for ROR Hydro Project as per the NEA Board decisions dated April 27, 2017 is indicated in the table below:

Table 15-5: Tariff structure for ROR Hydro Project

Season	Rate NRs./KWh	Min. Dry season Energy required
Dry (16 December- 13 April)	8.40	15%
Wet (14 April- 15 December)	4.80	

The elelctricity price will be escalated annually @3% till 8th year and it is assumed that from 9th year onwards the Electricity Price will remain same as that of 8th year. It is observed from the above figure that the Electricity price escaltion was within the annual inflation rate.

As per the above proposed NEA Electricity Price, the minimum energy sale in Dry season (4 Months) should be 15% of total energy available for sale. The net energy available for sale with outages and without considering Rolwaling from Tamakoshi V HPP is given in the below table:

Table 15-6: Net energy available for sale with outages and without considering Rolwaling

Sl. No.	Items Description	Unit	Values
1.	Dry Season Energy Sale to NEA	GWh	63.45
2.	Wet Season Energy Sale to NEA	GWh	359.55
3.	Spill Energy Available	GWh	89.33
4.	Total Energy Available for sale	GWh	512.33

From the above Table 2-3, the minimum energy sale in Dry season (4 Months) was calculated as 11%. Therefore, the total energy that would be sold to NEA is 423.00 GWh at the approved NEA tariff for the respective season. It was observed that around 89.33 GWh (17.4% of total energy) of spill energy is available to be sold to NEA. It is assumed that 100% of spill energy would be sold to NEA at wet season approved NEA tariff.

15.2.3 Estimated Electricity Price

The Estimated Electricity Price has the following components

- ✓ Return on Equity (RoE) @ 17%
- ✓ Debt Repayment
- ✓ Interest expense
- ✓ Interest expense on working Capital
- ✓ O&M expense
- ✓ Runner Replacement in 10 years
- ✓ Insurance payment
- ✓ Royalty to Government
- ✓ Tax
- ✓ Bonus and welfare fund

The NEA proposed Electricity Price vs Estimated Electricity Price for 30 years is indicated in the next figure:

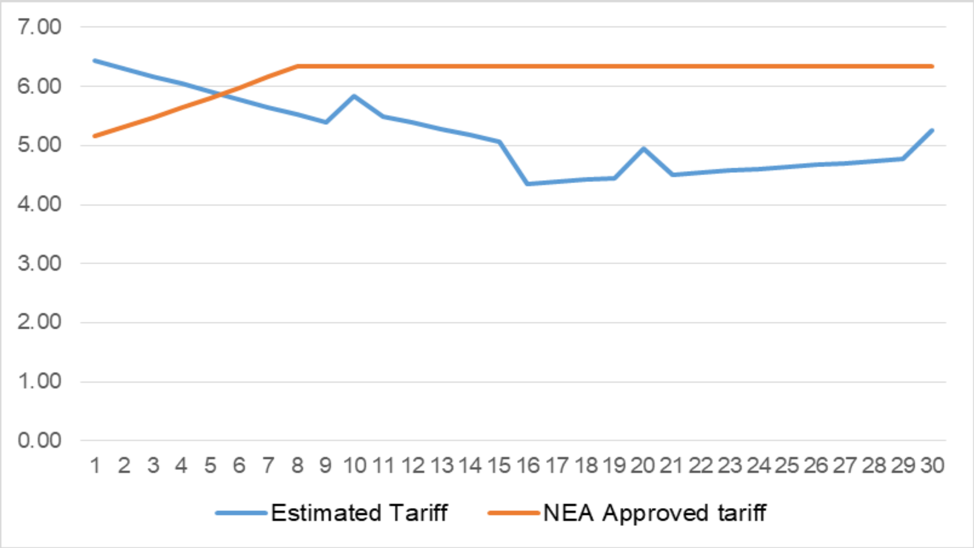

Figure 15-13: NEA proposed Electricity Price vs Estimated Electricity Price

The levelized Tariff in both the cases considering energy sale mentioned in the previous table is indicated in the table below:

Table 15-7: NEA Approved vs Estimated levelized Tariff

Description	Unit	1st year	8th year	Levelized for 30 years
Weighted Average NEA Tariff	NRs/kWh	5.16	6.35	5.95
Estimated Tariff	NRs/kWh	6.44	5.53	5.61

15.2.4 Conclusion

The estimated tariff required to recover the financing cost (Debt repayment, interest expense and RoE), the O&M expense, the royalties, taxes and duties is less than the present approved tariff by NEA except for initial 5 years of operation. The detail economic and financial viability is assessed under Scenario Analysis in the following Sections.

15.3 Economic Analysis

15.3.1 Introduction

The economic analysis had the objective of determining the Tamakoshi V project's impact on the national economy of Nepal. The cost and benefits analysis was considered at economic rates, which exclude taxes and duties, interest during construction, financing charges and includes, where appropriate, shadow pricing.

The economic analysis included the following aspect of the project

- Cost Benefit Analysis was calculated as gross direct benefits minus investment Costs, operating, maintenance and associated costs.
  - The gross direct benefits refer to savings in expenditure by the society as a whole.
  - Investment costs comprising the EPC Costs and administration cost during the construction period of the project.
  - Replacement costs of electrical and mechanical equipment during the economic life of the project.
  - Operation and maintenance cost comprising the amount of labor and materials that are required to make the project function over its life. These include labor, material and administrative expenses, but not all other financial costs, such as interest payments, taxes, depreciation and amortization of the borrowed capital. Both interest and taxes were considered transfer payments which had no real effect on the resources used by the project from a social point of view. Depreciation and amortization were excluded because they were embodied in the investment or project cost to which the benefits were compared.
  - Associated costs comprising all other costs of the side effects if they were not already considered in the assessment of the net direct benefits.

15.3.2 Cost Benefit Analysis

15.3.2.1 Gross direct benefits

The gross direct benefit included the following benefits

- The value assigned to hydropower production corresponds to the opportunity cost for producing the same amount of energy at the same plant factor in a combined cycle power plant with the same reliability of supply.
- The installed capacity of the CCPP was taken at 92 MW with Plant load factor 60% and Auxiliary power consumption of 3.5% to produce same amount of energy.
- For calculation of fixed Cost of Generation, the CAPEX for CCPP was taken at 85 Million USD with Debt to Equity Ratio as 70:30. The interest rate was assumed as 9% with repayment period of 15 years. The Return on Equity was taken 17%. The O&M expense for the CCPP was taken at 35000 USD/MW/Annum with annual escalation of 3% till the project life. For simplification of calculation, the working capital interest was not considered for fixed cost of generation.
- For calculation of variable cost of generation, the Net Heat Rate of the CCPP was considered as 8000 kJ/kWh at 60% PLF and the landed cost of gas was taken at USD 9.0/ GJ.
- For economic benefit analysis, equivalent amount of surplus energy has not been considered.

The gross direct economic benefit from alternative source of electricity and the cost of generation are indicated in the figure below:

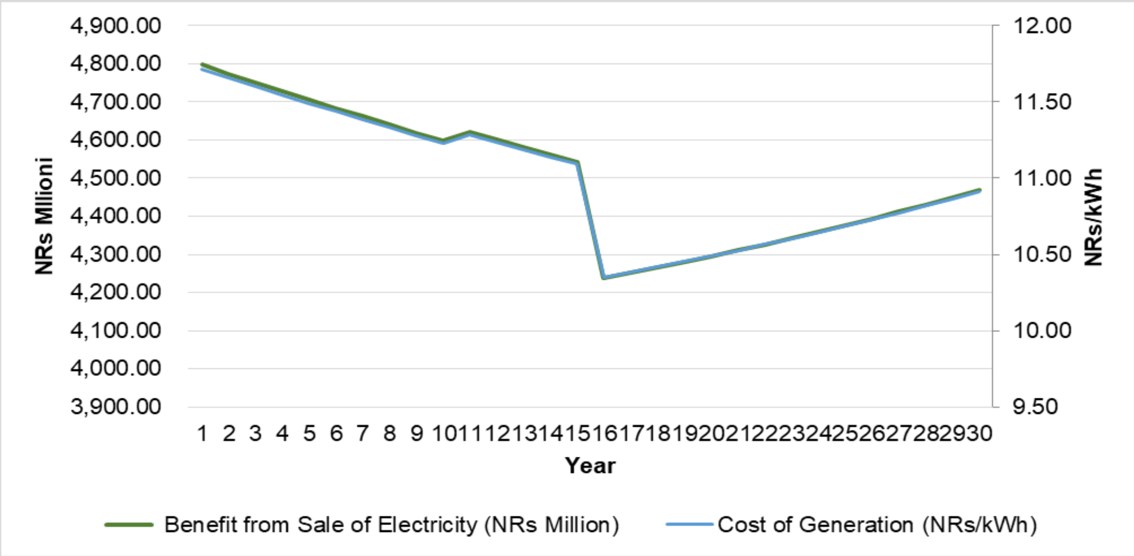

Figure 15-14: Economic benefit and the cost of generation from alternative energy source

15.3.2.2 Investment Costs

For the economic evaluation, market prices was not the best indication of the true value from a social point of view of the benefits and costs involved. Consequently, certain adjustments was made by application of shadow rates

- Imports
  - The Investment costs comprised of the EPC and other administration cost during the construction period of the project. The import duty and VAT was deducted from imported items and domestic items respectively.
  - All investment costs were taken at present price with no escalation during construction phase.
- Non -Traded Goods
  - Land, R&R and Environmental cost, as a non-traded item, constitutes a special case as it is not always and everywhere marketable in Nepal. Therefore, the value of Land, R&R and Environmental cost was considered at the present estimated cost.
- Unskilled labor
  - To the extent that unskilled labor was drawn from previously unemployed labor, the net loss of production to the rest of the economy was mainly given by foregone subsistence activities. Wage rates to be paid for unskilled labor in the project was however exceed the value of the subsistence activities to a large extent. Since the concept of costs in an economic appraisal is that of maximum alternative benefits foregone, the appropriate costs of unskilled labor was the value of subsistence activities. To arrive at the shadow price of unskilled labor, a conversion factor 0.8 was applied on the estimated cost of unskilled labor. The unskilled labour cost was taken at 3% of domestic components of Civil works, Hydraulic Steel Structures Electro-Mechanical and Transmission line cost. The unskilled labour cost after use of conversion factor worked out to be around NRs 60 Million. This had very marginal impact on the Economic Viability of this project.

- Foreign Exchange Rate
    - The present nominal exchange rate for the Rupee of NRs 112.12/ USD was considered.
- The total investment costs worked out to be NRs 14858.4 Million.

15.3.2.3 Replacement costs

The replacement of spares are part of annual operation and maintenance cost. However, runner and other high value spares replacement cost is taken separately for USD 2.5 Million in every 10 years.

15.3.2.4 Operation and maintenance cost

The Operation and Maintenance expenditure includes labour, material and administrative expenses including insurance cost, but not all other financial costs, such as interest payments, taxes, depreciation and amortization of the borrowed capital. This is taken at 1.0% of CAPEX on 1st year of operation and there after escalated at 3% annually.

15.3.3 Economic benefit Indicators

The Economic Benefit Indicators are given in the table below:

Table 15-8: Economic Benefit Indicators

Parameters	Values
Benefit to Cost Ratio	2.06
Economic IRR	21.07%
Economic Payback period	3.30 years
Economic NPV (NRs Million)	15,072.76

From the above table, it is observed the project is economically viable.

15.3.4 Sensitivity Analysis

A sensitivity analysis was performed to show how the profitability alters with different values assigned to

- **Increase in project cost by 10%**

Parameters	Values
Benefit to Cost Ratio	1.90
Economic IRR	19.53%
Economic Payback period	3.64 years
Economic NPV (NRs Million)	13,884.75

With increase in project cost, the economic parameters were lower than those of base case.

- **Increase the exchange rate from NRs 112.12/USD to NRs 115/USD**

Parameters	Values
Benefit to Cost Ratio	1.98
Economic IRR	20.32%
Economic Payback period	3.46 years
Economic NPV (NRs Million)	14,076.90

With increase in exchange rate, the investment cost increased. However, the higher exchange rate is not considered for the electricity cost of generation from gas based combined cycle power plant. Therefore, the economic parameters were lower than those of base case.

- **Increase the fuel cost from USD 9.0/GJ to USD 10.0/GJ**

Parameters	Values
Benefit to Cost Ratio	1.90
Economic IRR	19.65%
Economic Payback period	3.60 years
Economic NPV (NRs Million)	12,746.90

The higher fuel cost was not considered for the electricity cost of generation from gas based combined cycle power plant. Therefore, the economic parameters were lower than those of base case.

- **Increase in discount rate from 10.1% to 12%**

Parameters	Values
Benefit to Cost Ratio	1.79
Economic IRR	21.07%
Economic Payback period	3.30 years
Economic NPV (NRs Million)	10,491.55

With higher discount rate, the Economic NPV decreases.

15.3.5 Conclusion

The project is economically viable as the opportunity cost for producing the same amount of energy at in a combined cycle power plant is much more than that present approved by NEA.

15.4 Financial Analysis

15.4.1 Introduction

The financial analysis of a project was carried out to enable the both Debtors and Equity investors to take investment decisions. A financial model was prepared to analyse the financial viability of the project. The financial analysis included the revenue and expense projections to assess the cash available for Debt servicing and dividend payments. The detail analysis is discussed below.

15.4.2 Project Cost

15.4.2.1 Breakup of Project Cost

The broad breakup of Project Cost is indicated in the table below.

Table 15-9: Breakup of Project Cost

Sl No	Particulars	Foreign Component (USD Million)	Local component (Millions NRs)	Total Cost in (Millions NRs)
1.	Resettlement & Environment	0.8	41.8	131.2
2.	Engineering Management	3.2	167.4	524.7
3.	Owner's Expenses	1.6	83.7	262.4
4.	Permanent Camp & Buildings & Roads	0.0	468.0	468.0
5.	**EPC Cost**	**84.2**	**4032.6**	**13472.1**
a.	Civil Works	39.3	4032.6	8437.9
b.	Hydro mechanical Equipment	8.3	0.0	935.2
c.	Mechanical Equipment	10.8	0.0	1213.9
d.	Electrical Equipment	23.8	0.0	2672.0
e.	Transmission Lines	1.9	0.0	213.1
6.	Taxes & Duties	0.0	524.2	524.2
7.	Financing Charges	0.0	50.7	50.7
8.	Interest During Construction		2206.6	2206.6
	Total	**89.8**	**7575.0**	**17639.9**

15.4.2.2 Resettlement and Environment Costs

The Resettlement & Environment Cost includes all costs to be incurred towards resettlement of project affected people and towards environmental conservation and is assumed as 1% of EPC cost.

15.4.2.3 Engineering Management

The Engineering Management Expense is assumed as 4% of EPC cost.

15.4.2.4 Owner's Cost

The Owner’s Cost is estimated at 2% of EPC cost.

15.4.2.5 EPC Cost

The EPC Cost is estimated based on Rate Analysis and Internal Data Base. The EPC Cost includes contingency and 3% Miscellaneous Expenses. The Contingency is assumed at 5% on Tunnel & camp works, 4.5% on Surface works, 1% on Hydro-Mechanical, Mechanical and Electrical works, 5% on Transmission line towards any exigencies.

15.4.2.6 Taxes and Duties on EPC Cost

Table 15-10: Import Duty and VAT on Supply and Service

Items Description	Taxes and Duties
Electro Mechanical and Transmission line	
Import portion	0.0%
Domestic portion	13.0%
Civil works and other services	
Import portion	0.0%
Domestic portion	13.0%

15.4.2.7 Phasing of Expenditure

The project would be constructed within 48 months. The phasing of expenditure was assumed on half yearly basis considering the planned schedule of different activities.

15.4.2.8 Forex Variation during Construction Phase

The current exchange rate of NRs 112.12/ USD was considered for the viability assessment. The historical exchange rate as published by Nepal Rastra Bank is indicated in the figure below:

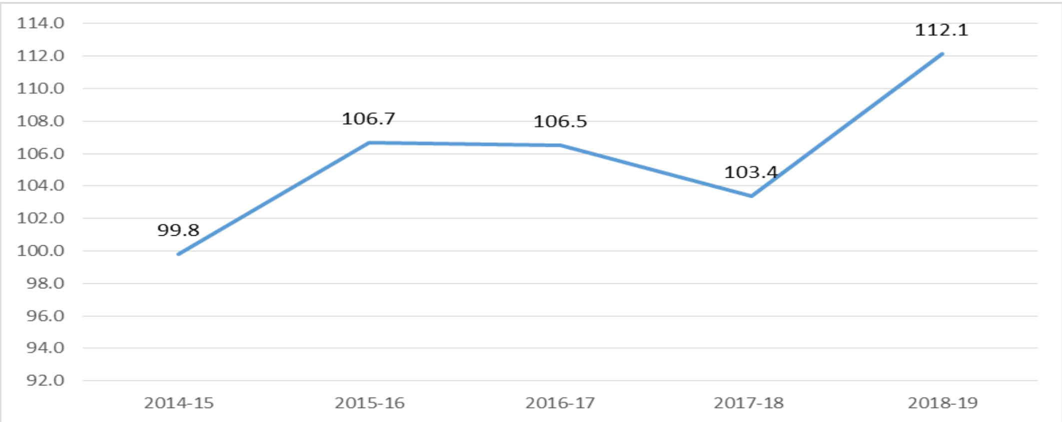

Figure 15-15: Historical Exchange Rate of Nepal

15.4.2.9 Financing Charges

All the financial institutions charge financing charges on loan capital. These are made to cover the financial commitment/servicing charges. These charges was taken as 0.5% of the loan capital. As per the general practice, the charges were assumed payable upon effectiveness of the loan, i.e. an upfront payment.

15.4.2.10 Interest during construction (IDC)

Interest during Construction was calculated on half yearly basis with an annual interest rate of 9.0%**.** It was assumed that IDC was funded 100% by lenders.

15.4.3 Inputs and Assumptions

15.4.3.1 Sale of Electricity

The amount of electrical energy available for sale in different scenarios is indicated in the table below:

Table 15-11: Energy available for Sale

Season	Energy Sale in GWh (with Rolwaling and Outages)
Wet (14 April- 15 December)	448.88
Dry (16 December- 13 April)	63.45
Total	512.33

The base case analysis was carried out with 8 months wet season and 4 months dry season with schedule outages and considering Rolwaling. As indicated in the table showing the tariff structure for ROR Hydro Projects (see Section 15.2.2), the minimum energy requirement as required by NEA is 15% during dry season. In this project, the dry season energy available is 11% and as a result, around 17% spill energy is available during wet season which is assumed to be sold at wet season tariff for financial viability.

The amount of energy available for sale and the amount of spill energy is indicated in the following table:

Table 15-12: Energy available for Sale and Tariff

Sl. No.	Items Description	Values (GWh)	Tariff (NRs/kWh)
1.	Dry Season Available Energy	63.45	8.4
2.	Wet Season Available Energy	359.55	4.8
3.	Surplus Energy	89.33	4.8
4.	Total Energy Available for sale	**512.33**	

15.4.3.2 Economic Life of the project

It is assumed that the economic life of the project would be 30 years after commissioning.

15.4.3.3 Royalty to Government of Nepal

Royalty would be paid to Government of Nepal at NRs 100/kW per annum for 1st 15 years from commercial operation and NRs 1000/KW per annum for balance life of the plant. Also, the royalty would be paid @2% of Revenue per annum for 1st 15 years from commercial operation and @10% of Revenue per annum for balance life of the plant.

15.4.3.4 Operation and Maintenance (O&M) Expenditure

The Operation and maintenance expenses for the first full year of operation after commissioning of the plant was considered as 1.0% of the capital cost and in the subsequent years that would increase by 3% over the previous year expense. The O& M expenditure included the employee expense, Repair & Maintenance expenditure, Spares & Consumables, Administrative and other related miscellaneous expenditure. The Nepal CPI and WPI trend is indicated below:

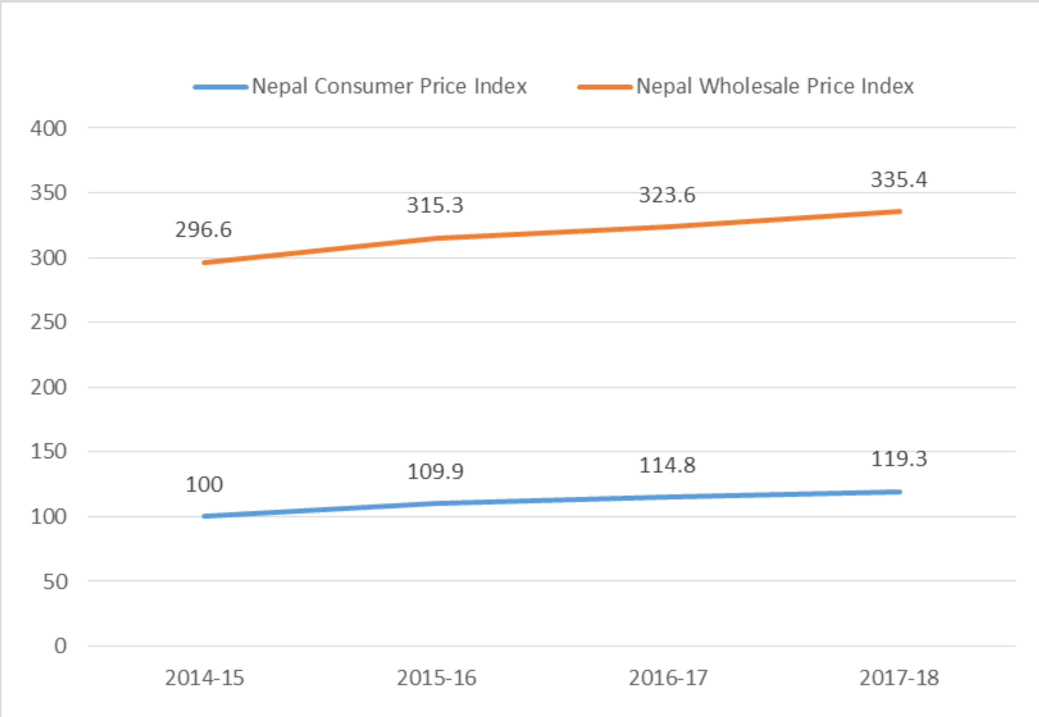

Figure 15-16: CPI and WPI Trend

15.4.3.5 Runner Replacement Cost

It is assumed that the old runners will be replaced with new ones in every 10 years for which USD 2.5 Million is considered.

15.4.3.6 Insurance Cost

The insurance premium for a Hydro plant varies w.r.to coverage in terms fire, force majeure events like earthquake, flash floods, very high siltation, damage of high value spares, loss Business etc. The Insurance expenditure for the first full year of operation after commissioning of the plant was considered as 0.5% of the capital cost and in the subsequent years that was increased by 3% over the previous year expense.

15.4.3.7 Financing Assumptions

The financing assumptions are indicated in the table below:

Table 15-13: Financing Assumptions

Sl. No.	Items Description	Values
1.	Equity	30%
2.	Debt	70%
3.	Annual interest Rate on Loan	9%
4.	Repayment period	15 years

15.4.3.8 Debt and Equity Drawl

The yearly Debt and Equity Drawl is indicated in the table below:

Table 15-14: Yearly Debt and Equity Drawl

Sl. No.	Debt	Equity	Total CAPEX
1.	2859.8	1414.6	4274.4
2.	2936.0	1337.3	4273.3
3.	4090.3	1772.9	5863.2
4.	2461.8	767.3	3229.1
Total	**12347.8**	**5292.1**	**17639.9**
5.	**70%**	**30%**	**100%**

15.4.3.9 Depreciation

The depreciation assumptions are indicated in the table below:

Table 15-15: Depreciation assumptions

Sl. No.	Items Description	Values
1.	Depreciation Rate for Books of Account	
2.	Plant and Machinery	3.3%
3.	Civil Works	3.3%
4.	Depreciation Rate for Income Tax	17%
5.	Plant and Machinery	15%
6.	Civil Works	5%

15.4.3.10 Working Capital Assumptions

The working capital assumptions are indicated in the table below:

Table 15-16: Working Capital Assumptions

Sl. No.	Items Description	Values
1.	Receivables (in terms of monthly billing)	2 months
2.	Operation and Maintenance expense	1 month
3.	Maintenance spares as % of O&M charges	15%
4.	Working Capital Borrowing	100%
5.	Working Capital Interest	12%

15.4.3.11 Tax and Duties

The Tax assumptions are indicated in the following table:

Table 15-17: Corporate Tax

Sl. No.	Items Description	Years from COD	Values
1.	Corporate Tax Rate	0-10	0%
2.		11-15	10%
3.		16-25	20%

15.4.3.12 Discount Rate

The Discount rate is calculated as weighted average cost of Capital i.e Debt%*interest rate*(1-tax Rate) +Equity%*Cost of Equity

Discount Rate= 70%*9%*(1-20%) +30%*17%= 10.1%

15.4.4 Financial Viability

15.4.4.1 Project Internal Rate of Return (IRR)

The Project IRR was obtained by equating the present value of investment costs (as cash out-flows) from Project and the present value of net incomes (as net cash in-flows) to Project which was Cash inflow from Revenue i.e. from Sale of electricity and cash outflows like Royalty, O&M expense, insurance expenses, Bonus & welfare expenses and tax expense.

15.4.4.2 Project Net Present Value (NPV)

The Project NPV was obtained by deducting the present value of investment costs (as cash out-flows) from the present value of net incomes (as net cash in-flows) to Project which was Cash inflow from Revenue i.e. from Sale of electricity and cash outflows like Royalty, O&M expense, insurance expenses, Bonus & welfare expenses and tax expense. The present value was calculated with a discount rate of 10.1% as indicated above.

15.4.4.3 Benefit to Cost Ratio (B/C)

The Benefit to Cost Ratio was obtained by dividing the present value of net incomes (as net cash in-flows) to Project which was Cash inflow from Revenue i.e. from Sale of electricity and cash outflows like Royalty, O&M expense, insurance expenses, Bonus & welfare expenses and tax expense with the present value of investment costs (as cash out-flows). The present value was calculated with a discount rate of 10.1% as indicated above.

15.4.4.4 Payback Period (years)

The payback period was calculated in number of years when the project investment cost is fully recovered.

The financial viability indicators are indicated in the table below:

Table 15-18: Financial Viability Indicators (Base Case)

Parameters	Unit	Values
Project IRR	%	10.81%
Project NPV	Million NRs	883.66
Benefit to Cost Ratio		1.06
Payback Period	Years	7.19

From the above financial indicator, it is evident that the project was financially viable in the base case with NEA approved tariff considering spill energy to be sold to NEA at wet season tariff.

15.4.5 Sensitivity Analysis

The sensitivity analysis was carried out to assess the project viability under different conditions.

15.4.5.1 Case-1 (Upper Tamakoshi Transmission Loss)

The sensitivity analysis was carried out keeping all other assumptions remain same except the following payment to Upper Tamakoshi Project for transmission losses.

The financial indicators for sensitivity analysis are given in the table below:

Table 15-19: Financial Viability Indicators for Case-1 sensitivity analysis

Parameters	Unit	Values
Project IRR	%	10.55%
Project NPV	Million NRs	542.08
Benefit to Cost Ratio		1.04
Payback Period	Years	7.35

From the above financial indicator, it is evident that the project was financially viable if the payment is made to Upper Tamakoshi Project for transmission loss.

15.4.5.2 Case-2 (Increase in Project Cost)

The sensitivity analysis was carried out keeping all other assumptions remain same except the following Project Cost increased by 5%

The financial indicators for sensitivity analysis are given in the following table:

Table 15-20: Financial Viability Indicators for Case-2 sensitivity analysis

Parameters	Unit	Values
Project IRR	%	10.21%
Project NPV	Million NRs	96.25
Benefit to Cost Ratio		1.01
Payback Period	Years	7.55

From the above Case-2 analysis, it is observed that the project was not viable if the project cost is increased by 5%.

15.4.5.3 Case-3 (Increase in project schedule)

The sensitivity analysis was carried out keeping all other assumptions remain same except the following Project Schedule increased by 6 months.

The financial indicators for sensitivity analysis are given in the table below:

Table 15-21: Financial Viability Indicators for Case-3 sensitivity analysis

Parameters	Unit	Values
Project IRR	%	10.45%
Project NPV	Million NRs	411.26
Benefit to Cost Ratio		1.03
Payback Period	Years	7.41

From the above Case-3 analysis, it is observed that the project was not viable if the project is delayed by 6 months from the 48 months implemetation schedule.

15.4.5.4 Case-4 (Increase in interest rate)

The sensitivity analysis was carried out keeping all other assumptions remain same except the following Increase in interest rate by 1% i.e. with interest rate of 10%

The financial indicators for sensitivity analysis are given in the following table:

Table 15-22: Financial Viability Indicators for Case-4 sensitivity analysis

Parameters	Unit	Values
Project IRR	%	10.65%
Project NPV	Million NRs	(59.76)
Benefit to Cost Ratio		0.99
Payback Period	Years	7.29

From the above Case-4 analysis, it is observed that the project was not viable if the interest rate of loan is increased by 1%.

15.4.5.5 Case-5 (Increase in outages by 1%)

The sensitivity analysis was carried out keeping all other assumptions remain same except the following Increase in outages by 1%.

The financial indicators for sensitivity analysis are given in folowing below:

Table 15-23: Financial Indicators for Case-5 sensitivity analysis

Parameters	Unit	Values
Project IRR	%	10.70%
Project NPV	Million NRs	742.96
Benefit to Cost Ratio		1.05
Payback Period	Years	7.25

From the above Case-5 analysis, it is observed that the project was viable if the outage is increased by 1%

15.4.6 Conclusion

It is concluded that with the estimated project cost, financing plan and NEA approved Tariff, the Project is financial viable in the base case considering spill energy is required to be sold to NEA at wet season tariff. The financial viability of the Project is very sensitive to project cost, schedule and interest rate.

Hence, it is recommended to negotiate on the above conditions with NEA and banks/FIs/Multilateral Agencies on the interest rate to make the project financially viable. It is also recommended to make proper implementation plan and to monitor the construction progress closely to avoid any time and cost overrun.

15.5 Project Promotion

15.5.1 Introduction

The Tamakoshi V HPP would take water discharged from Upper Tamakoshi HPP and produce electricity. Therefore, there is no dam and settlement chamber required for this project which saves investment cost and protects the environment. There will be minimal displacement of people. The installed capacity would be around 100 MW. The project can be completed in 48 months. The project is highly valuable for the country economical because it would contribute to the Nepal in term of reducing the energy deficit and it would develop the economy and the welfare of the Nepali citizens.

15.5.2 Financing Arrangement

Several financing arrangements could be considered

- **100% Public Ownership Development**
  - ***Financing by International Multilateral Institutions*** – As per the rules International Financial Institutions, International Competitive Bidding is required for procurement either Employer's or Contractor's Design. This is a very lengthy and complicated process which can take years to be implemented and not in line with the schedule of 4 years requirement expressed by the NEA.
  - ***100% by NEA***- International Competitive Bidding is required for procurement either Employer's or Contractor's Design. This is not recommended as the cost of equity is high and NEA needs to arrange the entire fund from their own resources. The financial viability indicators for this financing alternative is indicated in the table below:

Table 15-24: Financial viability indicators for 100% NEA financing plan

Parameters	Unit	Values
Project IRR	%	12.36%

Parameters	Unit	Values
Project NPV	Million NRs	(3,243.49)
Benefit to Cost Ratio		0.70
Payback Period	Years	6.31

The project is not viable if the project is 100% financed by NEA.

- *Partial Self-Financing by NEA (30%) and Loan* from Commercial Bank (*70%)* with competitive rate at 9% with Repayment period of 15 years. NEA needs to negotiate with concerned authority so that 100% of spill energy from the project is sold at Wet season tariff. The financial viability indicators were discussed in the indicators table for the base case.

• **Full Private Ownership Development**

- In that case, financing of the Equity would be brought by Private Sponsors (30%), the Loans would come from Multilateral Institutions (50%) and from Commercial Banks guaranteed by World Bank or Exim Banks (20%). ICB's Procurement is required under rules of the IFIs with either Employer's or Contractor's Design. It is also a very long and complicated process – Loan conditions are not concessional rendering the feasibility of this approach very uncertain (required RoE by Private Sponsors difficult to achieve).

• **Full Private/PPP Ownership Development**

- The project would be financed where the Equity would be brought by Public and Private Sponsors (30%), including Concessional Loans from Multilateral Institutions to finance the Public Investment Company (of Nepal),
- A Loan Agreement would be used for the rest of the Financing insured by an Export Credit Agency (70% of the Total Investment Cost) and with concessional conditions (20 years maturity, 5.0% interest rates)
- This type of credit may require only one EPC Contract with a Group of Contractor/Suppliers sourced in the Country awarding the loan.

Any interest rate and loan repayment better than the base case would make the project financially viable.

15.5.3 Promotional Strategy

The project proponent would promote on the advantages of the project as follows:

- The Tamakoshi V HPP has no dam and settlement chamber which saves investment cost and protects the environment.
- There would be minimal displacement of people.
- The installed capacity would be around 100 MW. The project can be completed in 48 months.
- The first year tariff would NRs 4.8/kWh and would escalate annually @3%.
- Nepal Electricity Authority (NEA) is a government entity, profitable and has high credit rating which assures payment towards procurement of electricity.

Considering many advantages of Tamakoshi V project, the NEA would be able to negotiate with banks for loan of 70% of investment cost with interest rate at 9% and repayment period of 15 years.

16 CONCLUSIONS AND RECOMMENDATIONS

16.1 Conclusions

16.1.1 General

The Tamakoshi V HEP has been designed to a detail sufficient for bidding. The design has been founded on a range of basic studies covering the fields of topography, hydrology, geology, construction materials and the environment. The costs of implementing this project have been estimated, and different financing scenarios have been analysed. Based on the results of those studies, the scheme remains technically feasible; its financial attractivity depends, however, much on the assumptions at which conditions the generated energy can be sold. In respect to the latter, the key issue was identified to be the portion of excess energy which is produced by Tamakoshi V HEP, and which is to a wide degree determined by the fact that Tamakoshi V is operated as a tandem project to UTK HEP.

An assessment of the impact of the scheme on the environment has been completed, and the adverse impacts were identified to be in the manageable order of magnitude. This is to a great deal accountable to the fact that most of the project components are located underground. A workable plan for minimizing, mitigating and/or compensating the identified adverse impacts was conceptualized. The impact assessment judged the scheme to be environmentally acceptable. However, the construction of the scheme will generate large quantities of excavated rock. As much as possible will be processed and used as concrete aggregate for the project structures. However, a significant part of this excavated material will remain unused and needs to be disposed off in various locations in the Tamakoshi valley.. Other disposal areas or further opportunities for the use of the material may have to be identified in the near future.

Construction of the scheme will provide employment and career opportunities for several hundred people. Two bailey bridges across the Tamakoshi River to the Adits 2 and 3 may enhance transport links in the area. This may improve the quality of living for the inhabitants of several local communities and should stimulate local trade.

16.1.2 Technical Characteristics of the Project

Due to the fact that the Tamakoshi V scheme has been conceptualized as a tandem project to UTK HEP which shall exploit the additional head downstream from the UTK HEP outlet, the principal characteristics was mostly maintained from the design of the Feasibility Study. This characteristics is generally marked by the valley-parallel arrangement of the project, comprising a long headrace tunnel leading through a pressure shaft to the underground powerhouse.

Few variations with respect to the arrangement of the Outlet Structure were investigated; these included also the associated necessary modifications of the Tailrace Tunnel alignment. The investigations showed that a shifting of the Outlet Structure some 500 m downstream along Tamakoshi River would significantly add to the project’s generating capacity while keeping the additional cost for this modification within feasible range.

The main features of the Tamakoshi V HEP as determined for the final characteristics have been listed in the introduction of this report with their respective salient features.

The energy which will be generated by the Tamakoshi V HEP has been estimated for four different scenarios, which principally take into account whether water will also be available from the partial diversion of the Rolwaling River into the Upper Tamakoshi reservoir or not, and whether planned outages are included in the simulation of the generation scenarios or not. The total annual energy amounts which will be produced by Tamakoshi V (referring to the high voltage terminals of the main transformers) were derived as follows:

- Rolwaling not considered, Outages not simulated 464.77 GWh/a
- Rolwaling not considered, Outages simulated 461.66 GWh/a
- Rolwaling considered, Outages not simulated 516.55 GWh/a
- Rolwaling considered, Outages simulated 512.33 GWh/a

From these data it is obvious that the consideration of planned outages has only a marginal effect on the generation capacity; the supply of addition water through the Rolwaling diversion is, to the contrary, of significant impact.

16.1.3 Project Costs

The total cost of the project have been estimated as listed in the table below:

Table 16-1: Project Base Cost is Summarized in Equivalent US Dollars

Item No.	Description	Foreign USD	Local NPR	Total USD
1	Civil Works	39,291,329	4,032,598,206	75,258,134
2	Hydromechanical Equipment	8,340,800	-	8,340,800
3	Mechanical Equipment	10,826,400	-	10,826,400
4	Electrical	23,831,600	-	23,831,600
5	Camp Works	-	467,977,213	4,173,896
6	Transmission Line	1,900,800	-	1,900,800
7	Owner's Costs	5,577,807	292,922,469	8,190,387
	Total Base Cost			**132,522,017**

16.1.4 Financing Scenarios

Different financing scenarios were investigated to check for the financial viability of the Tamakoshi V HEP. In these scenario the energy sales conditions were adopted in line with communicated NEA sales tariffs for dry and wet season energy, with all generated wet season energy being sold to NEA at its respective tariff. The interest rate was assumed at 9%, the repayment period as 15 years. The economic life of the project was considered with 30 years.

Based on these assumptions the following financial indicators were obtained which show the project to be financially viable.

Table 16-2: Financial Viability Indicators (Base Case)

Parameters	Unit	Values
Project IRR	%	10.81%
Project NPV	Million NRs	883.66
Benefit to Cost Ratio		1.06
Payback Period	Years	7.19

The investigated sensitivity scenarios showed that the project's financially viability varies depending on the assumptions adopted for the individual sensitivity cases. The investigated sensitivities included the payment of additional charges for transmission line losses to Upper Tamakoshi HPP, the increase in project costs, the prolongation of the construction time, the increase in the interest rate for project financing and the increase in plant outage.

16.2 Recommendations

16.2.1 General

Based on the findings of the present studies it is strongly recommended that the implementation of the Tamakoshi V HEP should be followed up with highest priority. The focus for the activities to be undertaken in the nearest future should be on the further improvement of the field data and the clarification of the financing scenarios being actually relevant for the project implementation.

Key issues to be the focal points of concentration should include the following.

16.2.2 Hydrology & Sedimentation

The filed observation campaign should continue to monitor the daily discharged in the Tamakoshi River at Lamabagar and the location of the Tamakoshi V Outlet Structure to establish a more reliable data base for river flow assessment. The campaign carried out during the present studies in 2017 and 2018 provide a sound basis for flow assessment; however, the observation period is yet short.

In a likewise manner, the observation of suspended sediments should continue. With respect to these data a campaign was carried out only over few months in the wet season of year 2018, and this observation period should significantly be extended to acquire a more reliable data base.

16.2.3 Geology

Considering the geological condition of the site the key recommendations for the Tamakoshi 'V' Hydroelectric project include the following:

The drillings carried out from the Test Adit in the location of the powerhouse cavern did not allow to conclude on the in-situ rock conditions, since only highly disturbed drill cores were obtained. The geological conditions in this location should be clarified at highest priority by a suitable additional investigation program, which may include additional drill holes, borehole scanning, and other investigations as deemed appropriate.

From the ERT completed in the tailrace tunnel alignment close to the outlet structure it is observed that one of the tomogram profiles revealed weak bedrock in the TRT alignment. Therefore, an investigation drilling is advisable to confirm the bedrock along the alignment during construction stage.

River water sample from Tamakoshi is recommended for chemical test at three different locations viz. near head pond area, downstream to confluence of Orang Khola and nearby powerhouse location to assess its suitability with cement and aggregates for construction purpose.

16.2.4 Preparation for Construction

All land acquisition for the permanent surface structures of the project, including access roads, transmission line, cable car / ropeway, headworks, all adit and tunnel portals and spoil areas should be completed.

For lands required for temporary facilities such as equipment and work yards for the contractor(s) and accommodation and office camps for the contractor(s), the Engineer and the Employer respective lease agreements should be concluded.

The financing scheme which shall finally be relevant for the implementation of the Tamakoshi V HEP shall be clarified. This shall include the clarification of the energy sales and tariff conditions, the conditions pertaining to any loans and other financing arrangements, debt/equity arrangements and related aspects.