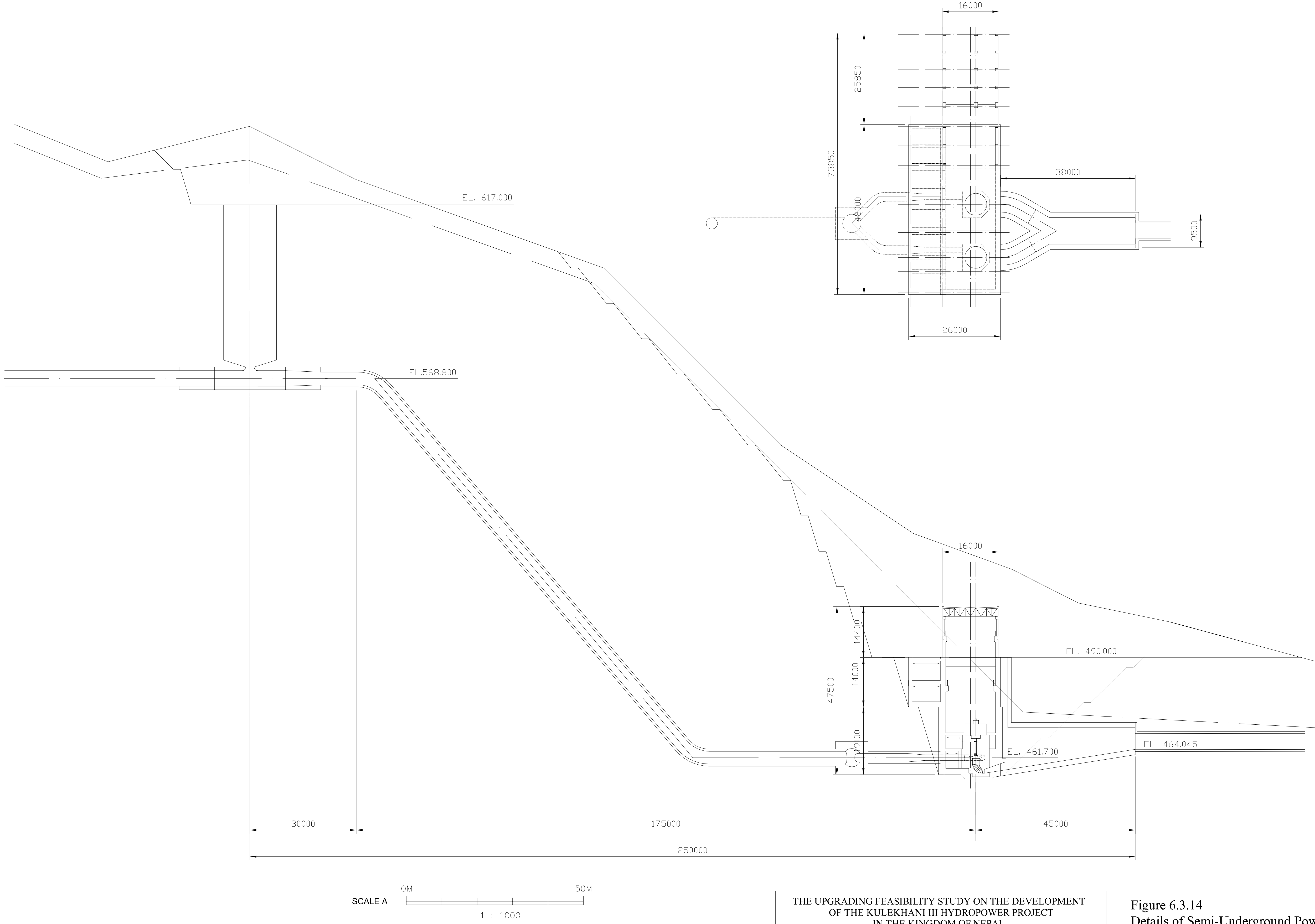

THE UPGRADING FEASIBILITY STUDY ON THE DEVELOPMENT OF THE KULEKHANI III HYDROPOWER PROJECT IN THE KINGDOM OF NEPAL

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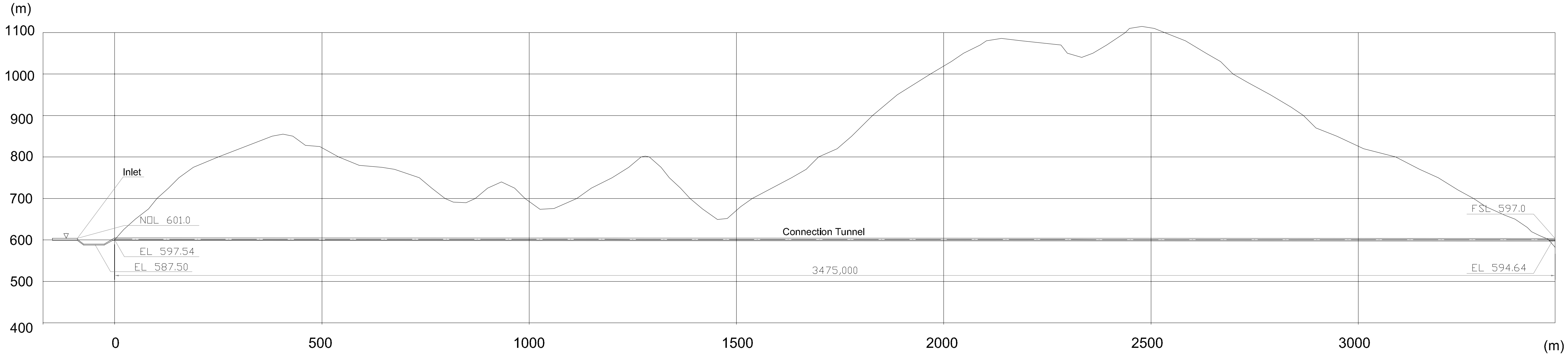
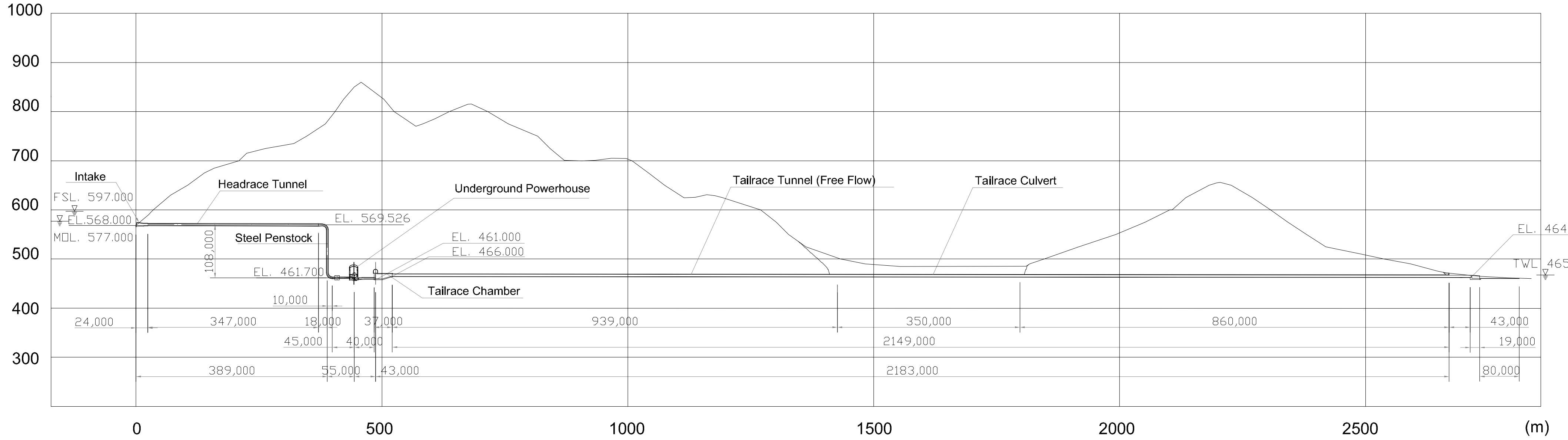
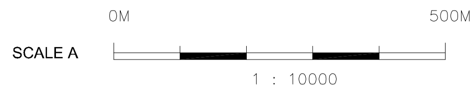
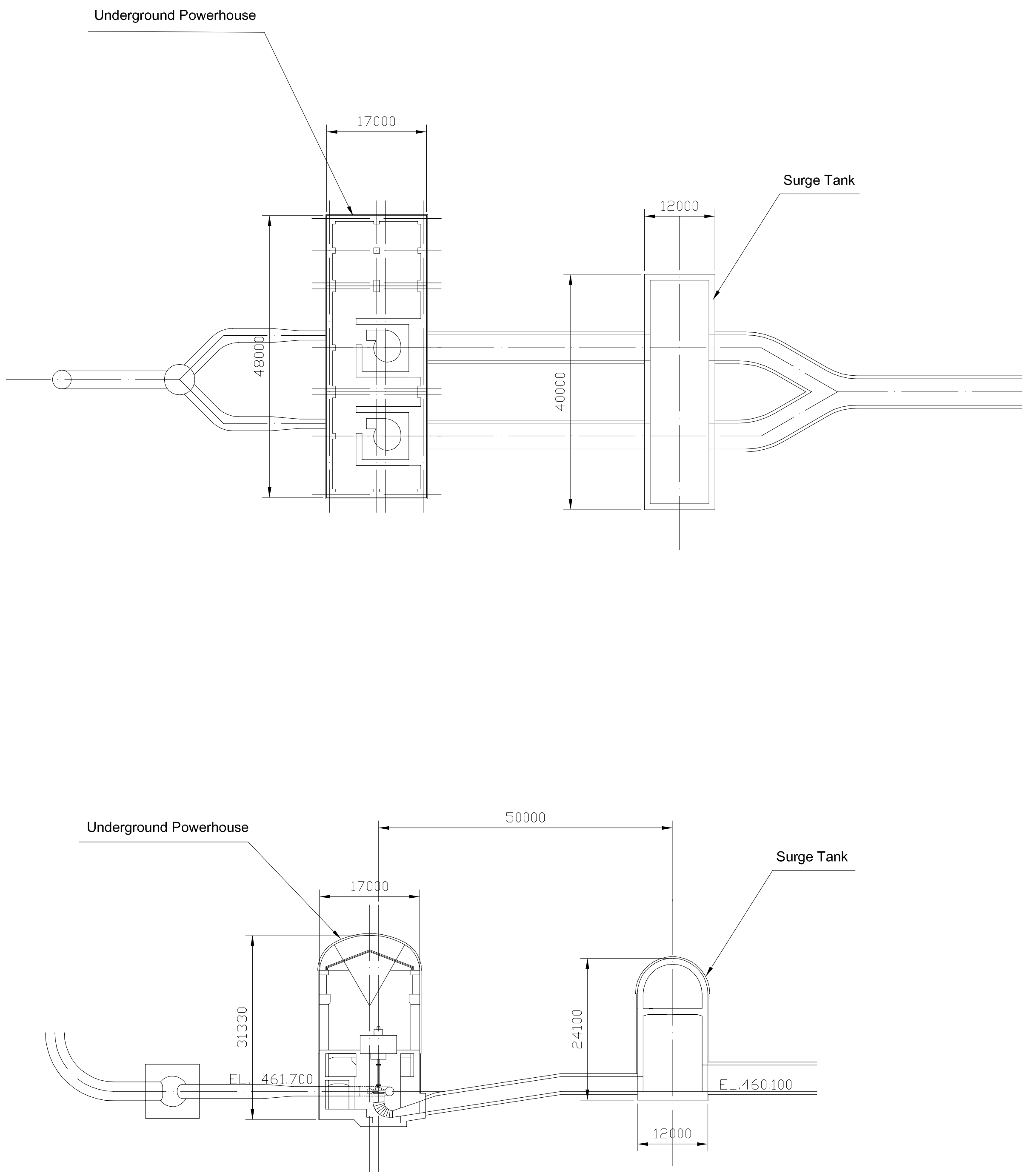
Figure 6.3.14
Details of Semi-Underground Powerhouse (Alternative A-1)



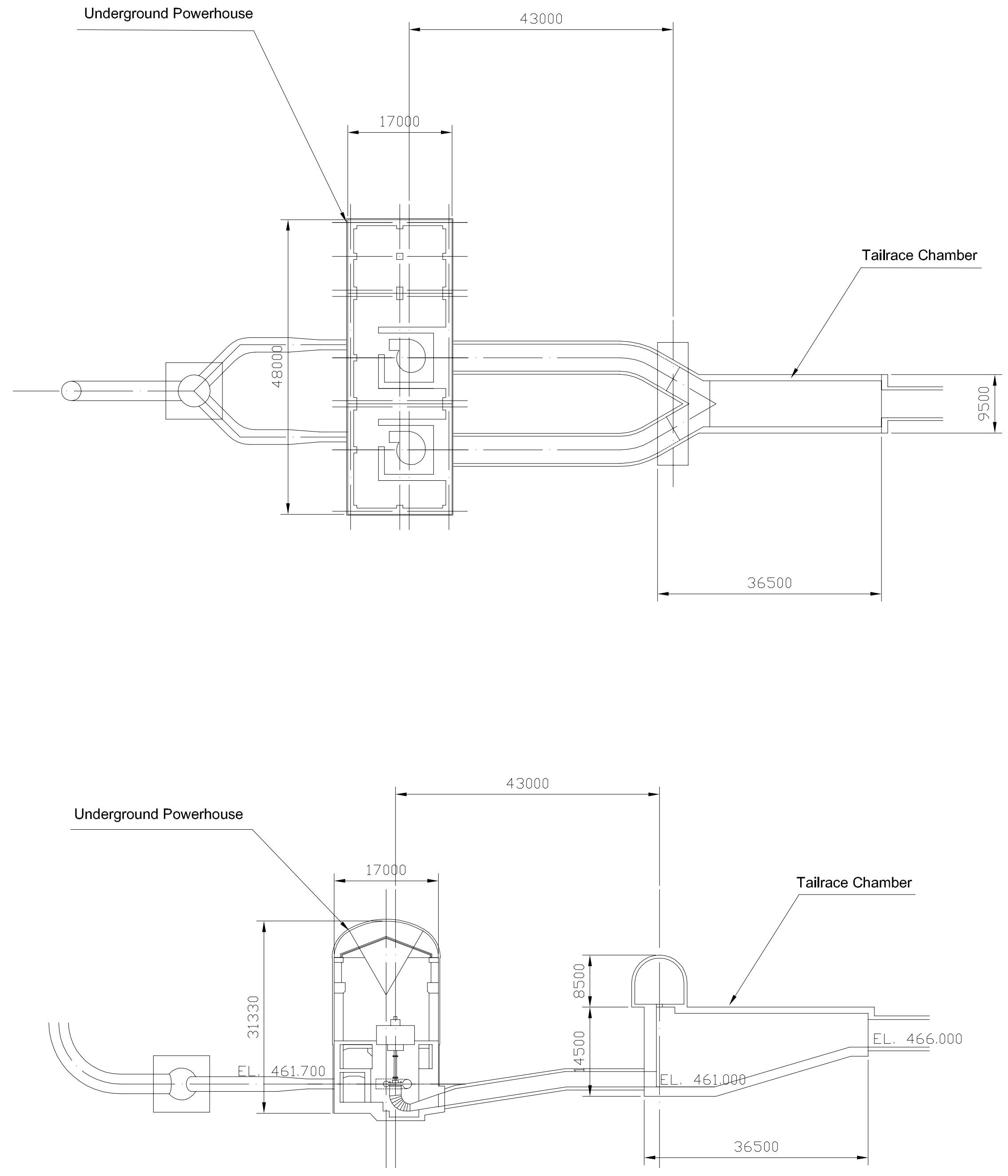




THE UPGRADING FEASIBILITY STUDY ON THE DEVELOPMENT OF THE KULEKHANI III HYDROPOWER PROJECT IN THE KINGDOM OF NEPAL	Figure 6.3.15 Profile of Alternative A-2 Underground Powerhouse + Tailrace Culvert
JAPAN INTERNATIONAL COOPERATION AGENCY	



Pressure Tunnel Plan
(Alternative A)



Free Flow Tunnel + Culvert Tailrace Plan
(Alternative A-2)

SCALE A 0M 50M
1 : 1000

THE UPGRADING FEASIBILITY STUDY ON THE DEVELOPMENT
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IN THE KINGDOM OF NEPAL

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Figure 6.3.16
Alternatives of Tailrace Tunnel

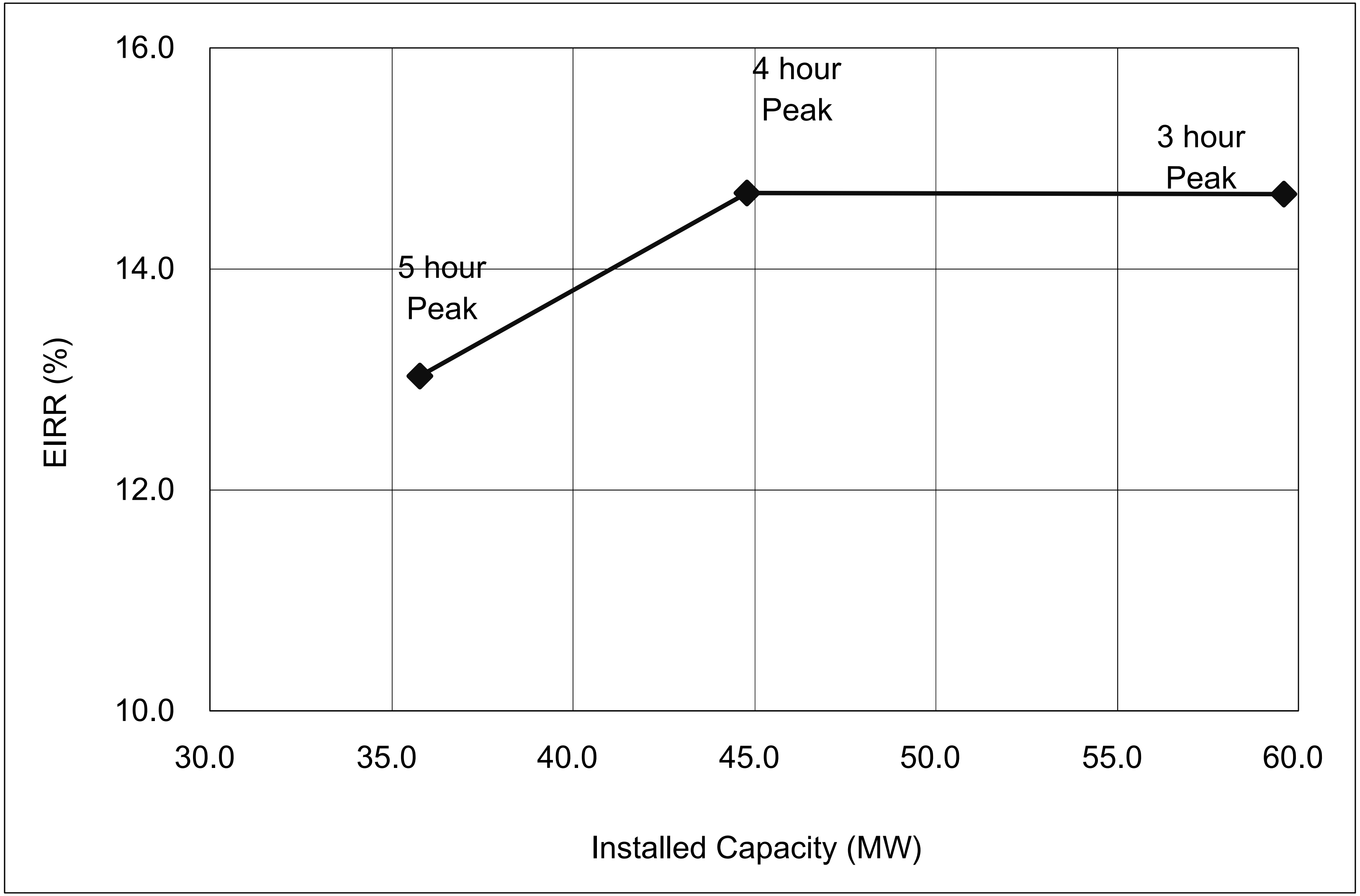


Figure 6.3.17 Comparative Study of Project Scale (EIRR)

Chapter 7

CHAPTER 7 OPTIMUM RESERVOIR OPERATION

7.1 Introduction

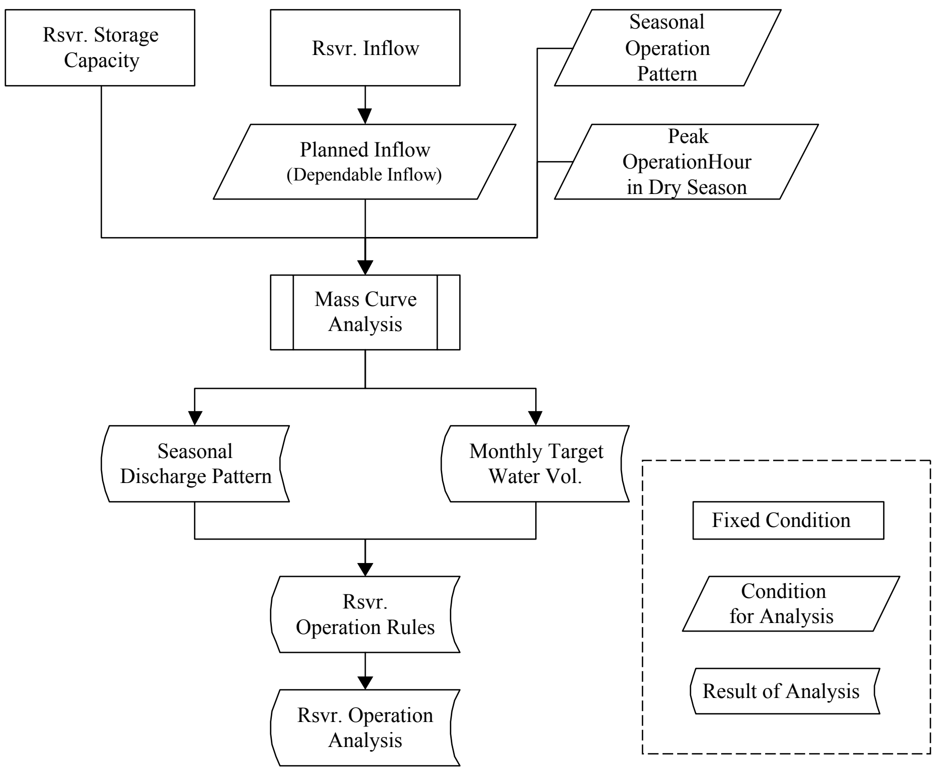
The operation of the Kulekhani hydropower station series (KL series), consisting of the Kulekhani I, II and III hydropower stations (KL-I, KL-II and KL-III), is controlled by the Kulekhani reservoir. The existing operation rules[1)] for KL-I were formulated with a basic discharge pattern that was derived from the runoff record at the Kulekhani G.S. in the third drought year of 1963, out of a total analysis period of 1963 to 1972.

Existing Operation Rules	Dry Season	Wet Season
Seasonal Operation Pattern	Dec. to May (6 months)	Jun. to Nov. (6 months)
Peak Operation	13.1 m^3/s x 4 hrs.	6.55 m^3/s x 4 hrs.
Off-peak Operation	4.8 m^3/s x 20 hrs.	1.21 m^3/s x 20 hrs.

It is necessary to review the operational rules by considering changes in demand patterns and the reduced storage capacity of the reservoir since then.

7.2 Reservoir Operation Analysis

The reservoir operation is analyzed following a workflow illustrated below:



Comparison studies have been carried out to optimize operation conditions, such as planned inflow, seasonal operation pattern and peak operation hour in the dry season. Prior to this analysis, seasonal discharge pattern and monthly target water volume were set by mass curve analysis as a part of the operation rules.

(1) Reservoir Inflow

The long-term sequence of the reservoir inflow, which was estimated through the runoff analysis, is utilized for this analysis after converting to data on a 5-day basis.

The operation rules are formulated on the basis of monthly effective inflow, defined as the reservoir inflow minus evaporation loss. An average evaporation loss is estimated to be 0.05 m^3/s, from 80% of the annual evaporation record at Chisapani Gadhi and an area of reservoir water surface of 1.2 km^2 at an average water level of EL.1,508.3 m for the period 1983 to 1995. The monthly effective inflow is obtained, from the inflow, by subtracting the average evaporation loss on dry days.

Table 7.1.1 shows the long-term sequence of the monthly effective inflow for 33 years from 1963 to 1995. It indicates an annual mean effective inflow of 4.32 m^3/s.

The following table shows standard deviations of the monthly effective inflow. This indicates a standard deviation of 2.65 for an annual mean value, with marked annual fluctuations in the wet season. The 80%, 70% and 60% dependable inflows, obtained by applying probabilistic analysis based on the standard normal distribution, are utilized as planned inflows for formulating the operation rules.

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Mean
Mean	1.66	1.44	1.33	1.29	1.65	4.45	11.09	10.16	8.86	4.5	3.21	2.22	4.32
Max.	2.59	2.25	2.24	2.48	3.45	24.4	31.7	24.33	23.96	11.95	26.58	11.33	13.94
Min.	0.69	0.67	0.52	0.38	0.70	0.72	2.16	4.40	2.89	1.86	1.24	1.00	1.44
Standard Deviation	0.42	0.38	0.40	0.51	0.63	4.45	7.35	4.67	4.74	2.38	4.24	1.68	2.65
Dependability of Planned Inflow													
80%	1.30	1.12	1.00	0.86	1.12	1.62	5.30	6.24	4.88	2.50	1.78	1.64	-
70%	1.44	1.25	1.12	1.03	1.32	2.46	7.26	7.73	6.40	3.26	2.05	1.75	-
60%	1.55	1.35	1.23	1.17	1.49	3.17	8.91	8.99	7.68	3.91	2.27	1.85	-

(2) Storage Capacity Curve of Kulekhani Reservoir

The storage capacity curve as of 1995[2)], which was estimated by the Area-Increment method, is adopted for this analysis.

The difference in gross storage between 1995 and 2001 is 1.14 x 10^6 m^3, equivalent to less than 2% of the gross storage, as tabulated below. Accordingly, the curve of

1995 is used as the latest one with reasonable accuracy. The M.O.L. was raised to EL.1,483.5 m from the original EL.1,476.0 m after installation of a sloping intake in 1997. For this analysis, the live storage is revised to be 55.51 x 10^6 m^3 by reducing the storage volume below the M.O.L.

	Gross Storage (10^6 m^3)	Dead Storage (10^6 m^3)	Live Storage (10^6 m^3)	F.S.L. (EL. m)	M.O.L. (EL. m)
Nov. '95	63.50	4.60	58.90	1,530.0	1,476.0
Nov. '01	62.36	6.79	55.51	1,530.0	1,483.5

(3) Seasonal Operation Pattern and Peak Operation Hour

1) Seasonal operation pattern

The long-term trends of the reservoir inflow seem to have a seasonal pattern of hydrological condition that divides the year into the dry season (December to May) and the wet season (June to November).

However, recent operation records of the existing power stations have indicated remarkable tendencies of demand concentrated in the December to March/April period. As presented in Figure 7.1.1, the power supply during this period heavily relies on the power output from the KL series having a seasonal regulating capacity.

Accordingly, the reservoir operation is analyzed by setting seasonal operation patterns that either divide the year into a dry season of 4 months and a wet season of 8 months or a dry season of 5 months and a wet season of 7 months.

2) Peak operation hour

According to the daily load curve projected for estimation of the peak demand in 2008, as presented in Figure 6.2.2, the daily load on KL-I and KL-II will reach more than 90% (80 MW) of their generating capacity (60 + 32 = 92 MW) over a period of 8 to 9 hours.

Therefore, comparison studies are carried out on the daily operation pattern of KL-I in the dry season, setting 8 hours as a standard peak operation hour and varying it between 4 to 12 hours. The peak operation hour in the wet season is set at 4 hours following the existing operation rules.

With regard to the seasonal operation pattern and the peak operation hour, conditions for this analysis are summarized as follows:

Seasonal Operation Pattern	Dry Season	Wet Season
4-month Dry Season Operation	Dec. to Mar. (4 months) <4 to 12 hours>	Apr. to Nov. (8 months) <4 hours>
5-month Dry Season Operation	Dec. to Apr. (5 months) <4 to 12 hours>	May to Nov. (7 months) <4 hours>
Existing Operation Rule	Dec. to May (6 months) <4 hours>	Jun. to Nov. (6 months) <4 hours>

<Peak operation hour>

(4) Benefit of Generation

The benefit of generation is evaluated in accordance with the Long Run Marginal Cost (LRMC) [3)]. Two alternatives are considered for calculating the benefit of generation as follows:

Case 1: Energy benefit (kWh) only applied to KL-I and KL- II

Case 2: Power and energy benefit (kW + kWh) applied to KL-III

The kWh value is divided into for a primary energy (peak energy in the dry season: December to May) and for a secondary energy (off-peak energy in the dry season plus peak and off-peak energy in the wet season: June to November). Following a definition of the peak energy in the LRMC, the peak energy over 8 hours per day is evaluated as the secondary energy.

Unit values of the benefit of generation are tabulated below. Considering imports from India of shortfall electricity and exports to India of surplus electricity, kWh value for secondary energy is based on the price of power exchange with India that was effective from FY1996.

		Case-1 (Energy Only)	Case-2 (Power + Energy)	Remarks
Applied to		KL-I and KL-II	KL-III	
kW Value (Power)	US$ /kW	-	121	-
kWh Value (Energy)				-
- Primary Energy	US ¢ /kWh	9.0	6.1	Peak Energy in the Dry Season: Dec. to May
- Secondary Energy	US ¢ /kWh	4.0	4.0	Except the above

7.3 Optimum Reservoir Operation

(1) Optimization Process

To optimize the reservoir operation, comparison studies are carried out under the several conditions following a screening process as tabulated below:

Analysis Conditions	1st Screening	2nd Screening
KL-I		
- Rsvr. Planned Inflow	60%, $\boxed{70\%}$, 80% Dependable Inflow	
- Dry Season Operation	$\boxed{4\text{ –month}}$, 5-month	
- Peak Operation Hour in Dry Season	(Fixed at 8-hour)	4, 6, $\boxed{8}$, 10, 12 hours
KL-III		
- Peak Operation Hour	(Fixed at 4-hour)	(Fixed at 4-hour)

Reservoir simulations are conducted to evaluate the operation of the KL series under the above conditions using the hydrological data over 33 years.

Methods of the mass curve analysis and the reservoir simulation are detailed in the Volume II, Supporting Report (1), Appendix D1 and D2 with results obtained from those analyses.

(2) Results of Analysis

The result of each screening process is obtained from the reservoir simulations, as summarized below:

1) Results of 1st screening

In the 1st screening process, the planned inflow and the seasonal operation pattern is optimized as shown in Figure 7.2.1 and summarized below:

Seasonal Operation Pattern		4-month Dry Season Operation			5-month Dry Season Operation		
Planned Inflow (Dependability)	%	60	70	80	60	70	80
Seasonal Power Dis. - Dry	m^3/s	6.80	6.70	6.58	5.68	5.58	5.43
- Wet	m^3/s	2.27	1.37	0.42	2.27	1.37	0.35
Annual Spill Out (KL-I & II)							
- Dry	m^3/s	0.10	0.09	0.08	0.12	0.11	0.12
- Wet	m^3/s	0.86	0.86	0.96	0.94	0.95	1.05
Installed Capacity of KL-III	MW	44.6	44.8	44.6	35.0	38.4	38.0
Annual Energy Production (KL Series)							
- Primary Energy	GWh	117.6	122.5	114.3	131.1	137.2	132.4
- Total Energy	GWh	315.5	315.2	310.1	315.5	314.5	310.1
Annual Benefit (KL Series)	10^6US$	23.3	23.5	22.9	22.8	23.4	23.0

The seasonal power discharge and the monthly target water volume are determined by utilizing the mass curves drawn from the monthly planned inflows with the respective dependability and the seasonal operation pattern.

Each condition for the operation is optimized by the following reasons:

<u>Reservoir planned inflow</u>

- The annual spill out from KL-I and KL-II is maximized in the case of the 80% dependable inflow and minimized in the case of 70%. Under the higher dependability of the planned inflow, the monthly power discharge is reduced more severely to retain reservoir water volume for utilization in the dry season. This results in increasing of the annual spill out of excess water.
- It is thought that effective utilization of the reservoir is achieved by well-balanced operation rules planned by the 70% dependable inflow, which will represent annual fluctuations of the reservoir inflow.

<u>Dry season operation of KL-I</u>

- In the case of the 4-month dry season operation, the primary energy production by the KL series is decreased in comparison with the case of the 5-month period. This results from the definition of the primary energy in the LRMC, which is evaluated as peak energy during 6 months from December to May, irrespective of the dry season operation of KL-I for 4 or 5 months.
- Due to the concentrated utilization of the reservoir water volume in the shorter period, the larger installed capacity of KL-III is gained in the case of the 4-month dry season operation of KL-I compared with the case of the 5-month.
- The annual benefit of the KL series is maximized in the case of the 4-month dry season operation since the increment of the power benefit of KL-III exceeds the decrement of the primary energy benefit of the KL series.

In comparison of the annual benefit of the KL series, the power operation is optimized under the conditions of the planned inflow with 70% dependability and the 4-month dry season operation.

2) Results of 2nd screening

In the 2nd screening process, the peak operation hour of KL-I in the dry season is optimized as shown in Figure 7.2.2 and summarized below:

Peak Operation Hour of KL-I *)	hr.	4	6	8	10	12
Annual Spill Out from KL-II *)	m^3/s	0.03	0.04	0.05	0.06	0.18
Installed Capacity of KL-III	MW	44.8	44.8	44.8	44.6	43.6
Annual Energy Production						
- KL-I & II (Primary Energy)	GWh	57.3	78.5	99.5	99.1	98.9
(Total Energy)	GWh	269.8	268.7	266.8	264.6	262.7
- KL-III (Primary Energy)	GWh	23.0	23.0	23.1	23.0	22.5
(Total Energy)	GWh	48.2	48.3	48.4	48.4	48.1
Annual Benefit (KL Series)	10^6US$	21.5	22.5	23.5	23.4	23.1

*) Values in the dry season

Each condition for the operation is optimized by the following reasons:

<u>Peak operation hour of KL-I in dry season</u>

- For KL-I and KL-II, the higher ratios of the primary energy to the total energy are obtained under the longer peak operation in the dry season. This results in increasing the annual benefit, following extensions of the peak operation hour reflecting the difference in the unit values of benefit between the primary and the secondary energies.
- The annual benefit of the KL series is maximized in the case of the 8-hour peak operation of KL-I in the dry season since a peak energy over 8 hours per day is evaluated as a secondary energy, following the definition in the LRMC.
- The annual benefit of KL-III is not affected so much by the peak operation hour of KL-I in the dry season. It is gradually reduced by increased spill out from KL-II that is caused by insufficient power discharges for off-peak generation of KL-I due to the longer duration of peak generation.

In comparison of the annual benefit of the KL series, the power operation is optimized under the condition of the 8-hour peak operation of KL-I in the dry season.

(3) Conclusions

Based on the results of the series of the screening processes, the conditions for the optimum reservoir operation are presented as follows:

- Planned inflow : 70% dependable inflow
- Seasonal operation pattern : 4-month dry season operation
- Peak operation hour in dry season : 8-hour

On the above conditions, the KL series will enable to produce power generation as tabulated below:

Unit: GWh /yr.

	Primary	Secondary	Total
KL-I & II	99.5	167.3	266.8
KL-III	23.1	25.3	48.4
KL Series	122.6	192.6	315.2

Under the above optimum conditions, the annual energy production of KL-III is studied in detail in chapter 6, taking further detail conditions into account.

The main features of KL-III are as shown below:

- Full Supply Level (F.S.L.) : 597.0 m
- Effective Storage Volume : 475,000 m^3
- Firm Plant Discharge : 7.18 m^3/s
- Maximum Plant Discharge : 43.1 m^3/s
- Installed Capacity : 44.8 MW

REFERENCES:

1) Kulekhani Hydroelectric Project, Operation and Maintenance Manual for Civil Structures, Vol.1, 1982, Nippon Koei Co., Ltd.

2) Report on Sedimentation Study of Kulekhani Reservoir, 1995, NEA

3) Power System Master Plan for Nepal – Long-Run Marginal Cost, Final Report, Aug.1998, Norconsult

TABLES

Chapter 7

Table 7.1.1 Monthly Effective Inflow into Kulekhani Reservoir

Unit: m^3/s

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Mean
1963	1.42	1.28	1.38	1.41	2.04	1.40	8.44	9.77	7.31	4.43	2.69	2.16	3.64
1964	1.80	1.70	1.41	1.10	1.56	2.77	9.18	15.56	19.39	5.66	2.27	1.85	5.35
1965	1.61	1.58	1.49	2.18	1.82	4.70	21.91	22.60	7.26	3.97	3.58	2.16	6.24
1966	1.91	1.74	1.38	0.89	1.38	1.42	12.48	24.33	12.85	4.18	2.73	2.10	5.62
1967	1.31	1.13	1.04	1.24	0.78	5.55	14.05	8.81	7.19	4.21	2.89	2.07	4.19
1968	2.06	1.79	2.02	1.47	1.35	3.19	7.22	8.04	3.26	8.62	2.81	1.80	3.64
1969	1.44	1.07	1.04	1.12	1.18	1.10	3.94	8.80	5.30	2.36	1.34	1.00	2.47
1970	0.99	0.87	0.75	0.72	0.77	4.34	24.10	11.48	6.68	4.13	2.70	1.80	4.94
1971	1.40	1.35	1.34	2.48	2.67	24.40	6.47	9.82	5.40	4.78	2.76	2.29	5.43
1972	2.12	2.21	1.99	1.74	1.70	5.57	31.70	7.84	10.48	3.10	2.28	1.94	6.06
1973	1.69	1.52	2.24	1.16	1.60	12.21	9.37	10.18	11.92	11.95	6.50	3.43	6.15
1974	1.96	1.63	1.38	1.42	1.56	2.09	9.14	18.20	23.96	4.40	2.42	1.84	5.83
1975	1.57	1.45	1.07	0.91	1.33	2.67	17.15	13.94	14.71	5.95	2.91	2.05	5.48
1976	1.79	1.54	1.23	1.33	1.80	9.78	7.70	6.05	4.99	2.86	2.16	1.77	3.58
1977	1.57	1.49	1.30	1.74	2.04	1.96	4.00	4.40	2.89	2.23	1.97	2.01	2.30
1978	2.07	1.92	1.95	2.14	2.70	5.78	18.50	10.64	6.30	6.61	2.73	2.49	5.32
1979	2.32	2.25	1.95	1.71	1.55	4.10	13.01	9.11	3.59	2.24	1.85	2.11	3.82
1980	1.67	1.53	1.41	1.21	1.13	9.44	8.77	6.56	6.95	2.18	1.75	1.59	3.68
1981	1.48	1.35	1.16	1.77	1.12	1.29	2.77	4.41	11.40	2.76	1.40	1.25	2.68
1982	1.12	1.09	1.02	0.82	0.80	2.80	2.16	7.38	8.71	1.86	1.24	1.10	2.51
1983	0.69	0.91	0.67	0.89	1.86	1.47	10.23	6.20	9.56	5.48	3.61	1.90	3.62
1984	1.41	1.27	0.91	0.62	0.70	2.33	7.80	6.73	12.70	5.67	2.36	2.12	3.72
1985	1.35	0.85	1.33	0.38	2.96	1.80	6.84	7.38	17.52	9.92	3.21	2.88	4.70
1986	2.13	1.76	1.25	1.91	3.45	6.29	7.28	13.11	11.34	6.92	2.86	2.40	5.06
1987	1.84	1.62	1.69	1.10	1.00	0.72	16.10	12.30	6.66	7.79	3.17	1.96	4.66
1988	1.75	1.36	1.73	0.99	1.67	4.59	7.15	12.49	9.03	3.34	2.06	2.34	4.04
1989	2.59	1.44	1.01	0.69	2.10	2.33	12.66	6.68	6.32	4.22	2.47	1.42	3.66
1990	1.22	1.32	1.47	1.20	2.16	2.16	12.80	12.34	10.14	4.16	2.31	1.92	4.43
1991	1.90	1.24	1.21	1.48	1.06	2.56	4.95	9.69	7.53	2.24	1.63	1.62	3.09
1992	1.38	0.93	0.52	0.47	1.76	1.21	3.67	4.68	3.29	1.92	1.27	1.12	1.85
1993	0.93	0.67	0.82	1.23	1.67	4.34	30.98	12.24	3.99	3.08	1.92	1.34	5.27
1994	2.35	1.99	1.50	2.10	1.98	2.69	3.01	5.00	8.01	2.26	1.61	2.01	2.88
1995	1.84	1.77	1.28	1.05	1.18	7.87	10.37	8.48	5.91	3.10	26.58	11.33	6.73
Mean	1.66	1.44	1.33	1.29	1.65	4.45	11.09	10.16	8.86	4.50	3.21	2.22	4.32

FIGURES

Chapter 7

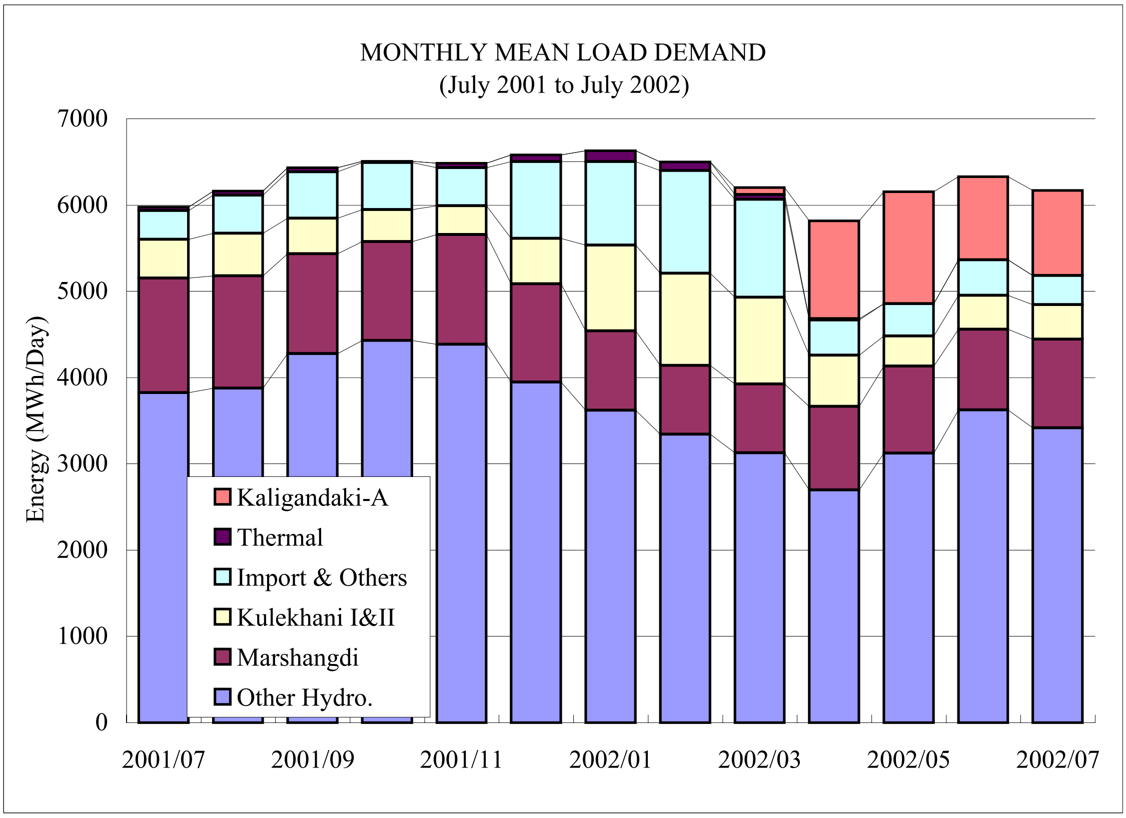


Figure 7.1.1 Monthly Mean Load Demand

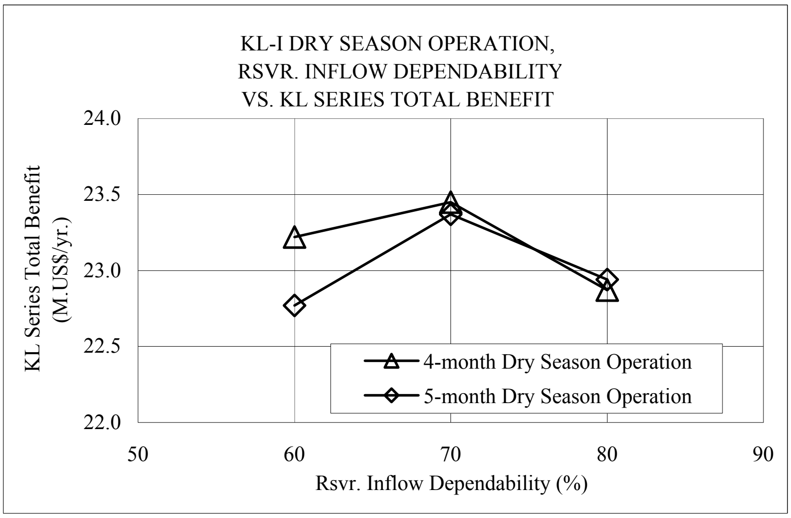


Figure 7.2.1 Results of 1st Screening

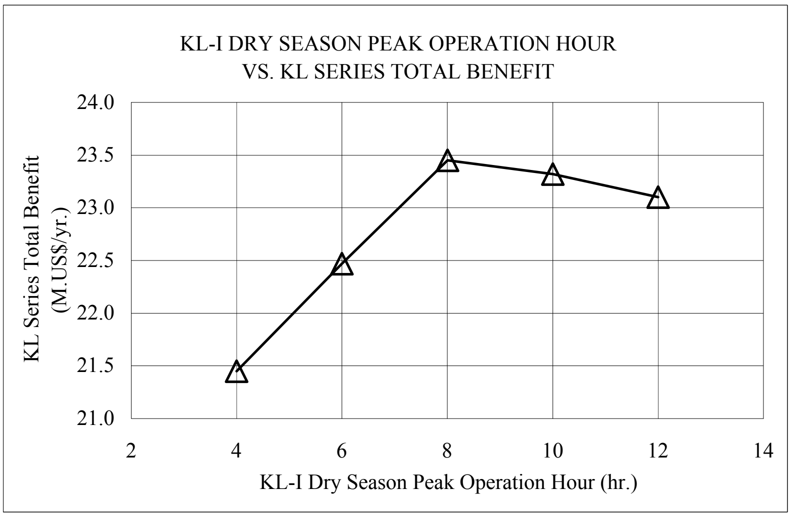


Figure 7.2.2 Results of 2nd Screening

Chapter 8

CHAPTER 8 FEASIBILITY GRADE DESIGN

8.1 General

As mentioned in Chapter 6, the scheme selected in the optimum development plan is composed of a dam type regulating pond and underground powerhouse. The plan and profile of the optimum development plan are shown in Drawing 1 and 2 respectively. This section describes the feasibility grade design of the main structures. The main features of the selected scheme are as shown in the table below:

Main Features of Optimum Development Plan

(1)	Catchment Area of Regulating Pond	8.1	km^2
(2)	Full Supply Level (FSL)	El. 597.0	m
(3)	Minimum Operation level (MOL)	El. 577.0	m
(4)	Gross Storage Volume	652,000	m^3
(5)	Effective Storage Volume	475,000	m^3
(6)	Gross Head	131.5	m
(7)	Rated Head	117.8	m
(8)	Maximum Plant Discharge	43.1	m^3/s
(9)	Installed Capacity	44.8	MW
	90 % Firm Peak Power in the Dry Season (Dec. – Mar.)	44.8	MW
	90 % Firm Peak Power in the Wet Season (Apr. – Nov.)	8.6	MW
(10)	Annual Energy Production	47.3	GWh/year

8.2 Khani Headworks

8.2.1 Optimum Intake Discharge from Khani River

The Khani headworks are planned to be constructed between the Khani consolidation check dam and the tailrace outlet of the Kulekhani II hydropower station. They are composed of an intake structure, a head pond and a siphon crossing the Khani River as shown in Drawing 3.

The intake water from the Khani River and the discharge from the Kulekhani II hydropower station are conveyed to the Yangran regulating pond through a 3.5 km-long connection tunnel of free flow type.

The optimum intake discharge from the Khani River is examined by comparing net present value (NPV) for each discharge. Cases for comparison are 1.0, 2.0, 3.0 and 4.0 m^3/sec that are selected based on the results of hydrology analysis in Section 3.2. The result of study indicates that the optimum intake discharge is 2.0 m^3/sec as shown in the following table.

Comparison Study for Optimum Intake Discharge of Khani Headworks

Item	Unit	Alternative Intake Discharge			
Intake discharge	(m^3/sec)	1.0	**2.0**	3.0	4.0
Design Discharge	(m^3/sec)	14.3	**15.3**	16.3	17.3
NPV	(10^3 US$)	21,415	**21,428**	21,348	21,168

8.2.2 Khani Headworks

(1) Intake Structure

An intake structure is located downstream of the consolidation check dam that was constructed at the middle point between the confluence of the Khani and Khali Rivers and the Kulekhani II hydropower station.

The maximum design discharge of 2.0 m^3/s is taken from the existing stilling basin of the consolidation check dam. The intake structure consists of inlet, connection channel, sand trap basin, river outlet and connection pipe.

The stilling basin will function as an intake bay since the water depth in the stilling basin is maintained at 2.3 m by a sub dam located downstream of the basin. A channel connects the stilling basin to the sand trap basin. The sand trap basin has a side spillway to release excess water to the Khani River through the river outlet. A pipe connects the sand trap basin and the headpond.

The 100-year probable flood of 470 m^3/s is applied to determine the top of the intake structure at EL.611.5 m. Flood water level (F.W.L.) is set at EL.611.3 m as a result of non-uniform flow calculation.

(2) Headpond

The headpond structure is located just downstream of the existing tailrace outlet of the Kulekhnai II hydropower station to receive the available discharge of 13.3 m^3/s from the Kulekhani II hydropower station and that of 2.0 m^3/s from the Khani intake.

The headpond is a box culvert, which is selected for the safety against overflow caused by riverbed rising, with a reinforced concrete structure of 19.0 m in length, 10.0 m in width and 10.0 m in height. The top of the headpond is set at EL. 605.6 m which coincides with the 100-year probable flood water level considering allowable riverbed rising of 4.0 m from the present riverbed at EL.598.5 m.

The normal operation level is set at EL. 601.0 m, which is the tail water level of the Kulekhani II hydropower station. The bottom elevation of the headpond, EL. 595.0 m, is equal to that of the existing outlet apron.

A spillway of 10.0 m width is provided at the downstream end of the headpond. A pair of gate slots of 3.0 m width is also installed at an entrance of a siphon for installation of intake stoplog gate. During maintenance of the connection tunnel, the outflow from Kulekhani II hydropower station and inflow from the Khani intake are released from the spillway by installing the intake stoplog gate by a wheel crane. The outflow is discharged through a culvert type outlet channel to the Rapti River, considering possible riverbed aggradation along the Khani River in future.

(3) Siphon

A siphon structure is located at the downstream of the headpond to lead the available discharge to the regulating dam through the connection tunnel.

To avoid severe damage from flowing water or flooding of the Khani River, the siphon conduit is embedded in the foundation rock under the river deposit where limestone bedrock is available. The foundation rock lies 12.0 m under the present riverbed at the deepest point.

The siphon is a concrete conduit type with a cross section of 2.5 m in width and height. Total length of the conduit is 75.0 m.

8.2.3 Connection Tunnel

The 3,500m long connection tunnel is designed as a free flow tunnel. The tunnel section is a standard horseshoe shape of 3.25 m diameter with a concrete lining in order to minimize friction loss. Tunnel support is applied in accordance with design criteria shown in Table 8.2.1.

A typical section of the connection tunnel is shown in Drawing 4. The lining thickness is 20 cm without reinforcement bars for standard section. The connection tunnel will be driven in the Mahabarata Thrust (MT). In the section crossing MT, the reinforced concrete lining of 30 cm thickness is applied to secure long-term tunnel stability.

A work adit is provided to shorten the construction period of the long connection tunnel. The length of the work adit is 500 m. It is connected with the connection tunnel at the 1,300 m downstream from the beginning point of the connection tunnel. The tunnel section is vertical leg horseshoe shape with 4.2 m in width and height.

8.3 Regulating Dam

8.3.1 Comparison of Dam Axis

In the feasibility studies carried out by Nippon Koei in 1988 and by NEA in 1999, the dam axes were laid out at about 1,250 m upstream from the confluence of the Yangran and Kesadi Rivers as promising site (US site).

In this study, the geological investigations were undertaken mainly targeting reaches further downstream (DS site), between about 1,000 to 1,150 m upstream from the confluence, to compare the site conditions with that in the US site previously surveyed. The bedrock of the US site mainly consists of phyllite, while dolomite in the DS site. These investigations revealed that sound bedrock for foundation of concrete gravity dam will need to be deeper at the possible axis in the DS site than that in the US site and that weathered or relaxed portions especially existed at the abutments of the axis.

Both sites were compared from the viewpoints of geological conditions, economic and storage capacity as follows:

- The US site has more suitable geological conditions of foundation and abutments than the DS site to dam construction.
- This results in less excavation and concrete volume for the US site.
- The US site cannot accommodate a sufficient storage capacity for reservoir sedimentation separately from that for regulating operation.

Therefore, a site was identified at 50 m downstream from the US site as the middle stream site (MS site) that ensures a sufficient storage capacity, and has the same bedrock material as the US site. Then main features of each site were compared as follows:

Dam Axis		US Site	**MS Site**	DS Site
Dam Height	M	50	**52**	58
Crest Length	M	105	**110**	153
Concrete Volume	10^3m^3	61.3	**68.5**	76.8
Gross Storage Capacity	10^3m^3	570	**652**	807
Sediment Capacity/*	10^3m^3	52	**123**	158

/* Alllocatable capacity estimated from gross storage capacity deducting the required regulating volume

The MS site, located at 1,200 m upstream from the confluence, was selected as the most promising site since it was superior to the others from the viewpoints of geological conditions, economic and storage capacity.

8.3.2 Design of Regulating Dam

The regulating dam located on the Yangran River is for controlling daily operation of power generation and aims at storing the available water fed by a connection tunnel from the regulating pond. Most of the water flowing into the regulating pond is used at the time of peaking hours of 4 hours per day.

As the type of the regulating dam, it is conceivable that concrete gravity type and fill type are applicable to the site. Judging from availability of embankment fill material and ensuring sufficient storage volume along its narrow upstream reach, the concrete gravity type is more suitable.

The dam site is located 1,200 m upstream from the confluence with the Kesadi River. Phyllite foundation rock is exposed at this dam site. It is conceivable from the geological investigations that the foundation rock at bottom and on both banks of the dam is sound enough for a concrete gravity dam.

A roller compacted concrete (RCC) type dam, 52.0 m height, 110.0 m in total length and crest elevation at EL 600.0 m will be constructed in association with a power intake, as shown in Drawing 5.

The dam is designed with a downstream slope of 1:0.8 and an upstream fillet of

1:0.25 in slope on the basis of stability analysis under the conditions of the site geology and seismic coefficient of 0.15.

The 200-year probable flood of 280 m^3/s for the Yangran River is adopted as the design flood for the dam, spillway, chute and stilling basin. An 80.0m wide non-gated spillway has a capacity of passing the 10,000-year probable flood of 690 m^3/sec to secure the dam safety in an extraordinary flood.

The spillway crest elevation is set at EL.597.0 m with an effective storage volume of 475,000 m^3 out of a gross storage capacity of 652,000 m^3 in the regulating pond.

A sediment flushing facility, associated with two sluice gates of 3.0 m in width and 2.0 m in height is provided at EL.564.0 m to flush fine sediment deposits of sediment deposit.

Apart from the sediment flushing facility, two outlet channels are provided at the bottom of the dam. One channel is utilized as a temporary diversion channel during construction and will be plugged by concrete in the last stage of construction. The other channel will be provided for emergency use in the case of maintenance of the sediment flushing facility.

An inclined type power intake is constructed just upstream of the dam on the left bank side. The bottom of the power tunnel at the intake is set at EL.568.0 m. This elevation is set at 9.0 m below the minimum operation level of EL. 577.0 m, taking an intake water depth without air entraining into account.

8.3.3 Sabo Structures on Yangran River

To control the amount of sediment transported from the upstream reach of the regulating dam, two check dams, namely the check dam No.1 and No.2, are proposed as sabo structures along the Yangran River. The check dam No.1 and No.2 will be constructed around 1,600 m and 900 m upstream from the confluence with the Kesadi River, respectively.

(1) Check Dam No.1

The main purpose of the check dam No.1 is to protect the regulating dam from damage due to debris flows.

This check dam will also support the foot of an unstable landslide located 2,000 m upstream from the regulating pond.

The check dam site is determined at a point approximately 1,600 m upstream from the regulating dam, where solid rock is exposed and the river width is narrow. The effective height of the dam is proposed to be 15 m and its sediment control capacity is estimated to be 57,000 m^3, assuming a design sediment slope of 1/25, as follows:

Sediment Trapping Capacity:	50,000 m^3
Sediment Retarding Capacity:	5,000 m^3
Sediment Detaining Capacity:	2,000 m^3
Sediment Control Capacity:	57,000 m^3

(2) Check Dam No.2 (Siltation Dam)

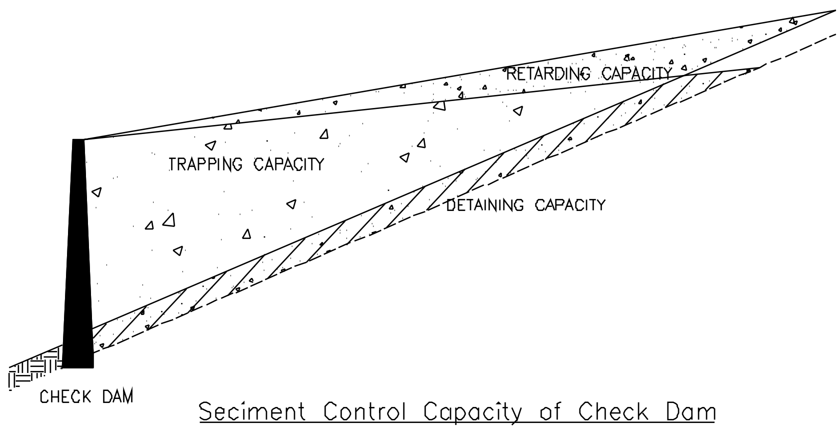
The check dam No.2 aims at trapping sediment and protecting the regulating pond from being buried with sediment deposits. The check dam functions as a siltation dam.

The dam site is determined at a point approximately 900 m upstream from the regulating dam, where solid rock is exposed and the river width is narrow. The effective height of the dam is proposed to be 15 m and its sediment control capacity is estimated to be 45,000 m^3, assuming a design sediment slope of 1/50, as follows:

Sediment Trapping Capacity:	36,000 m^3
Sediment Retarding Capacity:	4,000 m^3
Sediment Detaining Capacity:	5,000 m^3
Sediment Control Capacity:	45,000 m^3

Details of this check dam (siltation dam) are shown in Drawing 6.

It is assumed that suspended and bed load of 6,400 m^3, forming a part of sediment discharge of 16,100 m^3 from the upstream reach, will be captured by this check dam per year. The sediment control capacity of 40,000 m^3, except a sediment detaining capacity of 5,000 m^3, will be fulfilled by this sediment deposit within 6 years. Therefore, it is inevitable to excavate and remove this sediment deposit in dry season periodically.



Seciment Control Capacity of Check Dam

(3) Design Discharge of Check Dam

For the design discharge of debris flow for the sabo structures in Japan, the larger value of a 100-year probable flood or the recorded maximum flood should be adopted. In this study, 100-year probable floods, which are larger than the recorded maximum ones, were selected as the design discharges of the major structures as described in Section 3.2.6. Therefore, the 100-year probable flood is adopted as the design discharge of sabo structures. The design discharge of debris flow is computed by considering the density of debris flows from the following equation:

$$Q_{sp} = C_* / (C_* - C_d) \cdot Q_p$$

Q_{sp} : Design discharge of debris flow (m^3/s)

Q_p : Design discharge without sediment content (m^3/s)

C_* : Bulk density of deposited sand (approx. 0.6)

C_d : Density of debris flow (0.30～0.9 C_*)

$$C_d = \rho \tan\theta / (\sigma - \rho) / (\tan\varphi - \tan\theta)$$

ρ : Unit weight of water (11.8kN/m^3)

σ : Unit weight of gravel (25.5kN/m^3)

φ : Angle of shearing resistance of deposited sand (35°)

θ : Stream gradient (°)

Drainage area at the construction site	:	7.0 km^2
Stream gradient	: $\tan\theta$	0.16
Density of debris flow	: C_d	0.25 < 0.30
	$C_* /(C_*-C_d)$	2.0
Design discharge without sediment content	: Q_p	181 m^3/s
Design discharge of debris flow	: Q_{sp}	370 m^3/s

Consequently, the design discharge of the check dams was determined to be 370 m^3/s.

8.3.4 Countermeasures against Landslide

There is a landslide (R-1) on the right bank near the backwater of the regulating pond which is divided into upstream and downstream blocks by a gully. There is some possibility that frequent drawdown of the regulating pond during daily operation will affect the stability of Landslide R-1. Therefore, the slope stability is analyzed by taking account of the conditions of the regulating operation.

Based on the results of the analysis, it is proposed that the head portion of the downstream block be removed and an embankment counterweight be at the toe portion to stabilize it. On the other hand, the protection of the lower slope by riprap at the upstream block should be carried out to maintain stability against collapse.

These countermeasures on this landslide are presented in Drawing 7 and 8.

Apart from the above measures, a proper drainage system should be applied to stabilize the downstream block. This system would consist of drain ditches on the excavated berms and horizontal drain boreholes at the toe of the lowest excavation slope. This drain system would lead rainwater to the outside of the sliding area where gully erosion protection should be provided in the form of ground sills that would also quickly drain flow to the Yangran River.

8.3.5 Countermeasures against Slope Failure

Through the field investigations, slope failures are common in the Yangran River basin. It is judged that most of them are mainly caused by surface erosion due to high rainfall intensity. Therefore, it is considered that ordinary countermeasures against land/valley erosion are applicable, as shown in Drawing 9. These measures consist of logs with brushwood, and will be applied as protection from surface erosion.

8.4 Power Tunnel

8.4.1 Headrace Tunnel

A 350m long headrace tunnel is designed as a single lane pressure tunnel with circular section. It is planned to be located on the left bank of the Yangran River.

The optimum diameter of the headrace tunnel was determined so as to minimize construction cost of the headrace tunnel and power revenue loss due to head loss for alternative diameters, ranging from 3.9 m to 4.3 m. As shown in the table below, the optimum diameter is determined as 4.1 m.

Comparison of Headrace Tunnel Diameter

Item	Unit	Alternative Diameter				
Tunnel Diameter	(m)	3.9	4.0	**4.1**	4.2	4.3
(1) Annual construction cost	10^3 US$	96.2	98.7	**101.0**	104.3	107.0
(2) Power revenue loss	10^3 US$	33.1	29.6	**26.6**	24.0	21.8
Total Annual Cost (1)+(2)	10^3 US$	129.3	128.3	**127.6**	128.3	128.8

The lining thickness is set at 0.45 m, based on common practice of 1/8 to 1/10 of the diameter of the pressure tunnel. The lining will be reinforced to resist internal water pressure.

Typical section of the headrace tunnel is shown in Drawing 10.

8.4.2 Penstock

The 190m long steel penstock is designed as an underground type in a single lane tunnel. A circular section is adopted for the vertical shaft and a circular section with flat bottom for the horizontal tunnel for ease of construction. It is bifurcated at the upstream of the powerhouse to accommodate two steel penstock lines.

The optimum diameter of the penstock tunnel was determined so as to minimize construction cost and power revenue loss due to head loss for each alternative diameter ranging 3.2 m to 3.6 m. As shown in the table below, the optimum diameter is 3.4 m.

Comparison of Penstock Tunnel Diameter

Item	Unit	Alternative Diameter				
Tunnel Diameter	(m)	3.2	3.3	**3.4**	3.5	3.6
(1) Annual construction cost	10^3 US$	220.5	225.0	**226.0**	236.8	242.1
(2) Power revenue loss	10^3 US$	94.0	88.5	**82.4**	76.6	73.8
Total Annual Cost (1)+(2)	10^3 US$	314.5	313.5	**308.4**	313.4	315.9

The working space between the steel liner and surface of the excavated rock is 60 cm. It will be filled by concrete after installation of the steel liner. Backfill grout at the crown of the tunnel and contact grout around the whole perimeter of the steel liner will be applied along its entire length.

The typical section of the penstock is shown in Drawing10.

8.5 Powerhouse

8.5.1 General

As a result of the alternative layout study, an underground powerhouse is selected in Chapter 6. It is planned to be located in the sound dolomite layer with a thickness of about 150 m. The powerhouse cavern is selected as a tall horseshoe type for ease of construction. The turbine level is set at EL. 467.1 m to secure enough draft head to avoid cavitation. The size of the cavern is 17 m width, 31 m height and 74 m length for housing 2 units of generating equipment and main transformer.

The access tunnel is connected to the erection bay of the powerhouse from the access road. The generated power is transmitted to the transmission tower, which is installed at the access tunnel portal through the access tunnel. A 132 kV GIS is arranged in the underground powerhouse since there is no space outside due to the steep slope around the access tunnel portal. The layout of the underground powerhouse and access tunnel is shown in Drawing 11.

8.5.2 Layout

The powerhouse consists of a four-unit monolith consisting of the machine bay housing generating equipment, erection bay, main transformer bay and control building. The scale of the underground cavern was decided by considering the space required for installation of generating equipment and auxiliary equipment, turbine equipment, concrete foundations and working space.

The spacing between units is set at 15.0 m based on the dimension of the spiral case of the turbine and the thickness of initial and secondary concrete around the casing .

The size of the erection bay was determined so that erection work of the stator and rotor can be done simultaneously. The space is 14.0 m in width and 12.0 m in length including the non-working range of an overhead traveling crane.

The space of the main transformer bay was determined to take into account the size of the main transformer, working and inspection space and required partition wall thickness. Two main transformer rooms of 7.0 m in width and 11.0 m in length are arranged in parallel in the transversal direction. The GIS room is arranged upstairs of the main transformer room.

The valve floor situated at EL. 458.5 m is the lowest floor and is where the inlet valves, drain pump and draft manhole will be accommodated. The turbine setting level is determined at EL. 461.7 m to secure the draft head required to avoid cavitation.

The bifurcated penstock line (i.e. two lines) from the spherical branch enters the powerhouse at the turbine setting level. The inlet valves are arranged at the end of the penstock pipes and connected to the spiral case. The valve floor elevation is decided so as to keep sufficient clearance under the penstock for easy inspection and maintenance.

The turbine floor is situated at EL. 464.0 m where the cubicle, battery, governors, oil tanks and air compressor are provided. An access gallery to inspect the turbines is provided on this floor.

The generator floor, main transformer room and erection bay are situated at EL. 469.0 m. The erection bay is arranged between the machine bay and main transformer room. The access tunnel is connected with this floor.

The GIS room is situated at EL. 476.5 m just above the main transformer room.

The control and relay rooms are provided at EL. 473.2 m in the control building unit.

The details and room arrangement of the powerhouse are shown in Drawing 12.

8.5.3 Stability of Powerhouse Cavern

(1) General

The excavated cavern is stabilized by the following support system.

- Shotcrete: To protect the excavated surface and prevent the development of partial loosening of the rock mass from blasting.

- Rock Bolt: To prevent the development of partial loosening of the rock mass by anchoring.
- PS Anchor: To prevent the development of distressed rock zone and provide a confining pressure to the rock mass.

The stability of the cavern for the underground powerhouse was subject to the D-shape type. The analysis of stability during excavation was carried out utilizing the two-dimensional finite element method (FEM).

The required PS anchors will be calculated so as to support the loosened rock mass of the side walls of the cavern, which is obtained by the stability analysis mentioned above.

(2) Analytical Model

The geological conditions surrounding the underground powerhouse are a dolomite layer as shown in Figure 8.5.1. Phyllite and slate layers exist upstream and downstream of the dolomite layer respectively.

The mesh models created for FEM analysis are shown in Figure 8.5.2. The design values used for the stability analysis are summarized in the Table 8.5.1.

(3) Initial Stress

Initial stresses at the powerhouse are estimated on the basis of covering depth as follows.

$$\sigma_y = \gamma \cdot H$$
$$\sigma_x = \nu / (1 - \nu) \cdot \sigma_y$$
$$\tau_{xy} = 0$$

where, σ_x : Horizontal stress (MPa)
σ_y : Vertical stress (MPa)
τ_{xy} : Shear stress (MPa)
ν : Poisson's Ratio of Rock
γ : Unit Weight of Rock (kN/m^3)

(4) Disturbed Zone by Blasting

The modulus of deformation was determined taking into account the influence of blasting during excavation work. Since the magnitude of the influence of blasting varies depending on the depth from the excavated surface, the disturbed areas are empirically categorized into the zone-I (surface to 1.0 m depth) and zone-II (1.0 m to 3.0 m). The design values of those zones are shown in the table above (refer to (2) Analytical Model).

(5) Analysis Results

The stress distribution is shown in Figure 8.5.3. The maximum compressive stress of 36.5 MPa occurs on the sidewall. It is smaller than the 50 MPa unconfined compressive strength of the dolomite.

Distressed rock zone is defined as the area where the safety factor is less than 1.2. The safety factor is calculated as the ratio (r + d min)/r, where “r” is Mohr’s circle radius and “d min” is the minimum distance between the failure envelope line and Mohr’s circle. As shown in Figure 8.5.4, the distress zone reaches about 13 m.

PS anchoring is essential to prevent collapse or sliding of the cavern. The PS anchors shall be arranged properly to prevent development of a distress zone and to give confining pressure to the rock mass.

(6) Design of Cavern Supporting System

As a result of the calculation of the PS anchors, 18 m long PS anchors with 100 t design tension force will be placed at intervals of 2.0 m and 2.0 m on the upper part of the side walls. In the lower part of side walls, PS anchors of 15 m in length with 60 t design tension force will be placed at intervals of 2.0 m and 2.0 m. In the arch portion, 18 m long PS anchors with 100 t design tension force will be placed at intervals of 2.0 m and 2.0 m.

Rock bolts of 5 m length will be arranged between the PS anchors. The thickness of shotcrete is 32 cm at maximum with reference to other projects.

The supporting system is shown in Drawing 13.

8.5.4 Drain Holes

To mitigate water pressure acting on the powerhouse, a drainage system of underground water around the powerhouse will be required. The drain holes will be provided along the arch and side walls of the powerhouse and arranged between the PS anchors and rock bolts. If a large amount of leakage water is observed during construction of the access tunnel, long drain holes will be drilled from the work adit around the powerhouse.

8.6 Tailrace

8.6.1 Tailrace Chamber

As mentioned in Section 6.3.3, a tailrace tunnel of free flow type is adopted. The tailrace chamber is provided at the conjunction of the tailrace tunnel and draft tunnel. The tailrace chamber aims at dissipating hydraulic energy of water from the draft tunnel when there is a sudden load increase and to lead water smoothly to the tailrace tunnel. The tailrace chamber is shown in the Drawing 14. The typical section of the chamber is vertical leg horseshoe shape, being 7.5 m in width and 35 m in length. The height of the chamber changes from 12 m to 5.5 m.

At the upstream end of the chamber, a gate chamber is required for operation of the draft tube gate. The gate chamber is 7.0 m in height, 5.0 m in width and 20 m in length.

8.6.2 Tailrace Tunnel and Culvert

The tailrace tunnel leads the water from the tailrace chamber to the tailrace outlet. Total length is 2,100 m and the tunnel section is vertical leg horseshoe shape with a concrete lining. The optimum diameter of the tailrace tunnel was determined to minimize construction of the tailrace tunnel and power revenue loss due to head loss for alternative diameters ranging 4.2 m to 4.6 m. As shown in the table below, optimum diameter is 4.4 m.

Comparison of Tailrace Tunnel Diameter

Item	Unit	Alternative Diameter				
Tunnel Diameter	(m)	4.2	4.3	**4.4**	4.5	4.6
(1) Annual construction cost	10^3 US$	768.4	783.8	**797.1**	812.4	825.6
(2) Power revenue loss	10^3 US$	184.5	167.3	**150.0**	135.0	121.9
Total Annual Cost (1)+(2)	10^3 US$	952.9	951.1	**947.2**	947.5	947.5

The route of the tailrace tunnel crosses the Main Boundary Thrust (MBT) in the Kesadi River. As mentioned in Section 6.3.3, Siwalik Sandstone is fractured in a 130m section of the tailrace route due to the MBT. Consideration the poor geological condition and large seepage flow led to adopt a tailrace culvert in a 350 m section crossing the Kesadi River by cut and fill method.

A 20cm thick concrete lining is also applied in the tunnel section. The 100cm thick reinforced concrete culvert is also applied so as to support the overburden pressure. Typical section of tailrace tunnel is shown in Drawing 15.

8.6.3 Tailrace Outlet

The tailrace outlet is located 200 m downstream from the confluence of the Kesadi River and the Rapti River. The tailrace outlet is designed as an overflow weir type. The weir length is 10 m. The crest of the weir is also set at EL. 463.7 m to keep the tail water level at EL.465.5 m, which is almost same as the 100-year probable flood at the tailrace outlet.

8.7 Access Tunnel

An access tunnel is provided for the transportation of generating equipment, main transformer, parts of the steel penstock, construction materials, and excavated material and as access for operation and maintenance after completion. In addition, the access tunnel is used for air ventilation and running cables.

The entrance is set at EL 530.0 m and is connected with the underground powerhouse at the erection bay, where the elevation is EL. 469.0 m. The total length of the access tunnel is 800 m, and its average slope is about 7.6 %. The typical section is a vertical leg horseshoe section with clearance of 5.6 m width and 5.45 m height. The required space is determined to pass construction equipment and transporting the generating equipments.

Two kinds of lining are adopted. One is the concrete lining of 30 cm thickness and the other is shotcrete lining of 10 cm thickness. The concrete lining is applied at the entrance of tunnels and connecting points with the powerhouse and sections where bad geological condition are encountered. The bottom of the tunnel is paved with 20 cm thick concrete for ease of traffic passing.

The details of the access tunnel are shown in Drawing11.

8.8 Hydro-mechanical Equipment

8.8.1 General

The preliminary designs of the hydromechanical works are based on the principal design conditions, i.e., water levels, size, quantity, sill elevation, etc., which are determined from hydraulic analysis, water requirements, etc., as described in relevant chapters in this report.

This chapter outlines the main features of the hydromechanical works. Design data of hydro-mechanical equipment is summarized Table 8.8.1. The preliminary design will be subject to review and finalization in the detail design stage.

8.8.2 Intake Gate

One intake gate, 5.0 m in width and height, will be provided at the penstock inlet for shutting off water inflow to the penstock to allow the inspection and maintenance of penstock, inlet valves and water turbines. The gate will be operated under no flow condition and will usually be kept in a fully opened position. The gate will be closed under no flow condition for the inspection and maintenance of the waterway. The gate will be designed to enable the water flow to be shut off in an emergency case in the waterway. The gate will be opened to fill the waterway after the completion of inspection/maintenance works.

The gate size is determined to meet the diameter of the headrace tunnel, i.e., 5.0 m wide by 5.0 m high.

An electrically driven stationary type wire rope winch hoist is applied for the operation of the gate.

8.8.3 Intake Fixed Trashrack

One trashrack, 5.0 m in width and 35.0 m in height, is provided at the upstream side of the intake gate in order to prevent drifting logs and trash from entering the waterway.

The trashrack is comprised of a screen panel, top and bottom embedded beams and intermediate supporting beams.

8.8.4 Sand Flush Gate

A one lane sand flush gate, 3.0 m in width and 2.0 m in height, is provided at the lower part of the dam for flushing trapped sand from the dam.

The sand flush gate consists of a main gate, guard gate and conduit.

High-pressure slide type gate is applied for the main gate and guard gate.

A hydraulic cylinder hoist is applied for the gate, so that the thrust force produced by it may forcibly be able to close the gate against sand deposited on its sill.

A one lane steel conduit is provided to protect the dam concrete from abrasion by sand.

8.8.5 Bottom Outlet Gate

A one lane bottom outlet gate, 2.0 m in width and height, is provided at the lower part of the dam for lowering the dam water level in an emergency.

The bottom outlet gate consists of a gate and conduit.

A high-pressure slide type gate is applied for the bottom outlet.

A hydraulic cylinder hoist is applied for the gate, so that the thrust force produced by it may be able to forcibly close the gate against deposited sand on its sill.

8.8.6 Diversion Gate

One slide type diversion gate, 3.0 m in width and height, is provided at the diversion inlet of the dam to permit placing of the concrete plug following the diversion closure. A track crane will be applied for the operation of the gate.

8.8.7 Penstock

One complete lane of tunnel type steel penstock with one bifurcation and two branches will be provided for supplying the maximum water discharge of 43.1 m^3/s for two water turbines of 22.4 MW output each.

The penstock has a diameter varying from 3.4 m at the beginning point and 2.2 m at the branch pipes and approximately 190 m total length.

The steel penstock extends from the head pond with an upper vertical portion, a lower horizontal portion and a bifurcation.

The steel penstock will be connected to each short distance pipe of the turbine inlet at the powerhouse.

8.8.8 Draft Tube Gates and Monorail Hoist

One set of slide type draft tube gates and monorail hoist will be provided at the end of the draft tube for inspection, maintenance and repair of the two sets of water turbine and generating equipment.

The monorail hoist consists of hoisting units and a traveling unit will be used for operation of the gate.

8.9 Generating Equipment

(1) Number of units and unit capacity

The number of units and unit capacity of the turbine/generators of the power station are determined taking into account the following matters:

- Installed capacity of the powerhouse is 44.8 MW and the power station will be operated for daily peak load of 4 hours.
- For the same installed capacity, as the number of units decreases, the generating equipment cost decreases. Therefore, the number of units can be reduced up to two units for reliable operation of the power station.
- Besides, the Kulekhani III hydropower station is arranged in tandem with the two existing power stations, the Kulekhani I and II hydropower station (KL-I and KL-II), each having two units. A small capacity of regulating pond is provided in KL-III. However, the same number of units with KL-I and KL-II, which have two units of generator, is desirable for operation since operating units of KL-III depends on the operation of KL-I and KL-II.
- There is no impact on the power system if two units (22.4 MW per unit) are selected because the Integrated National Power System (INPS) in the year of 2007 which is envisaged to be the commissioning year of this KL-III, will reach about 760 MW as described in the foregoing Chapter 5.
- There are no significant problems concerning the transportation weight limit of the access roads if two units are selected since the maximum weight for the transportation seems to be about 30 ton adopted in the KL-I and KL-II projects under the Contractors' responsibilities.

Accordingly, the two units plan was determined for the KL-III.

(2) Single line connection diagram

A skeleton diagram for the two units plan is shown on the Figure 8.9.1, in which a unit system comprised of turbine-generator-transformer is adopted.

Outdoor space for installing 132 kV conventional type switchgear is not available due to steep land, so GIS type compact switchgear will be adopted for installation in the powerhouse. Besides, synchronizing operation is carried out using 132 kV circuit breaker(s) without provision of 11 kV circuit breaker(s), which saves the indoor space and cost.

An emergency power supply for station service (in case of 132 kV failures) is fed from the 11 kV distribution line from the Hetauda Substation for exclusive use or the 11 kV line (under construction) tapped off from the Hetauda Cement Line for permanent use after due betterment or improvement.

(3) Layout of major equipment and control equipment.

The main transformers and 132 kV GIS will be arranged in the underground powerhouse as shown in Drawing 12. The control room must also be provided inside the powerhouse since a surface control house is not possible owing to the very steep land area. A 132 kV power cable will be laid in the cable trench of the access tunnel to connect to the 132 kV transmission line through outdoor gantry structures.

(4) Control and Communication systems

A SCADA control system will be applied for using a distributed computer system with fast ether network in consideration of indication, control, telemetry and communication between the power station and a new load dispatch center master station (LDC) at Siuchatar, which is under construction under kfW grant.

The existing communication system among the Siuchatar substation, KL-II and Hetauda substation is PLC type. As the KL-III is connected to the existing 132 kV transmission line after taping-off, the same PLC system will be applied for the KL-III. However, if an optical fiber communication system is applied for the new LDC in the 66 kV line (instead of 132 kV line) to the Hetauda substation by replacing the existing GW by the OPGW under the LDC project, such communication equipment will be applied in the detailed design.

The same protection relay systems for the said transmission line sections will be used, such as distance relay (digital type is preferable for short distance instead of static type) using the above communication system.

(5) Salient futures for generating equipment

The salient futures of the generating equipment are listed below:

a) Turbine

- Type : Vertical shaft Francis
- Number of Units : 2
- Rated Output : 23 MW
- Rated Speed : 500 rpm

b) Generators

- Type : Conventional (suspended)
- Number of Units : 2
- Rated Capacity : 26.4 MVA
- Frequency : 50 Hz
- Power Factor : 0.85 (0.9 or 0.95 might be applicable if 132 kV line between Hetauda and Siuchatar/Tankot substations is strengthened by stringing another line on the existing towers.)
- Excitation system : Brushless excitation system

c) Main Transformer

- Type : Indoor, special 3-phase, Forced-oil-circulation water-cooled (OFWF)
- Number of Units : 2
- Voltage Ratio : 11 kV/132 kV
- Capacity : 26.4 MVA

d) Overhead Travel Crane: 65/20/3 tons

e) 132 kV Switchgear : Indoor GIS with single bus for incoming and outgoing feeders

8.10 Transmission Line and Switchyard

Three plans of transmission lines for the Kulekhani III Hydropower Project to connect with the Integrated National Power System are discussed and evaluated in Chapter 5, in which a "Loop-in loop-out connection with 132 kV Hetauda-Siuchatar line" is recommended for its economical advantages and because this plan does not require extension of the substation equipment in Hetauda. The planned route length from the Kulekhani III hydropower station to the existing tower to tap-off is about 1.7 km with seven towers, as shown in Figure 8.10.1.

The energy generated from the Kulekhani III hydropower station will be transmitted to Kathmandu and Hetauda via the Siuchatar-Hetauda line, presently a single circuit strung on the existing double circuit towers, and by a second circuit to be strung in a future transmission line project to increase the reliability of the power supply.

(1) Transmission line facilities

The same size of the existing conductor, Bear (ACSR 260 mm^2) is selected and has enough capacity to transmit the generated energy. Seven sets of double circuit towers will be constructed from No. 1 tower located near the portal of the access tunnel to No. 7 tower located near the existing tower for restringing. The method of connecting the new lines with the existing conductors through the restringing is outlined in Figure 8.10.2.

Galvanized steel towers will be used to support the line with symmetrical and vertical arrangement of the double circuit with two overhead earth wires for protection from lightning. A composite fiber optic overhead ground wire (OPGW) is to be used for one of the two to connect the communication system of Kulekhani III hydropower station to the optical fiber communication system of the new LDC in Siuchater through the 66 kV Siuchatar-Hetauda lines.

A typical skeleton steel tower is shown in Figure 8.10.3. The line route runs across extremely steep hills with relatively short spans except for the longest span of 600 m for crossing the Rapti River, Tribhuvan highway and Ropeway, so that huge uplift or pulling down must be loaded on the new towers. Due to the huge loads, specially designed solid towers are necessary. The required weight span of the towers may reach +3,000 m (compression) and -2,000 m (uplift) from the observation of a 1:5000 scaled topographical map. Only the tension type is to be employed for the seven towers. However, the final specifications and types of towers can be determined only after the route survey in the detailed design stage.

Insulators will be of brown glazed porcelain (254ϕmm x 146mm) for minimizing visual impact in the natural environment. A typical insulator string is shown in Figure 8.10.3.

(2) Substation facilities

For an underground powerhouse, a 132 kV GIS will be installed in the powerhouse along with the main transformers. A gantry structure for the double circuit line will be constructed near the portal of the access tunnel to connect to No. 1 tower, along with lightning arresters.

No extension of the substation facilities in the Hetauda substation outdoor switchyard is required as discussed above, except for the communication system.

8.11 Access Roads and Temporary Facilities

A 4.2 km long permanent access road will be constructed from Sanutar village to the regulating dam and check dam site, and 1.6 km to the access tunnel for the underground powerhouse. An irrigation canal is provided at the side of the access road to supply the irrigation water to Sanutar and Gumaune village. Temporary construction roads are also required to the work adit of the connection tunnel (1.2 km road), the tailrace outlet (1.0 km) and to the spoil banks and base camp (1.5 km). A bridge between Tribuban Highway and Sanutar village is needed and will be 150 m long and 7 m wide.

In addition, the causeway to the Hetauda cement quarry needs to be reinforced for access to the work adit of the connection tunnel.

The temporary facilities for the construction works consist mainly of temporary buildings (contractor's office, camp, etc.), workshops, a concrete batching plant, an aggregate crushing plant and a spoil bank. These temporary facilities can be arranged around Ghumaune village, located on the left bank of the Yangrang River.

Four spoil banks could be located as shown in Drawing 1. The total volume is estimated at 790,000 m^3. Typical sections of access road are shown in Drawing16.

TABLES

Chapter 8

Table 8.2.1 Design Criteria

Rock Grade	Description	Q-value	Tanaka's Method	Thickness of Shotcrete (cm)	Rock Bolt		Length of Rock bolt (m)	Steel Support (m)	Concrete Lining (m)
					Sectional direction (m)	Longitudinal direction (m)			
Q1	Excellent	Q>40	B	5	-	-			Plane concrete (t=0.2 m)
Q2	Good	10<Q<40	CH	5	1.5	2.0	0.4 De		Plane concrete (t=0.2 m)
Q3	Fair	4<Q<10	CM	10	1.5	1.5	0.4 De		Plane concrete (t=0.2 m)
Q4	Poor	1<Q<4	CL	10	1.2	1.2	0.5 De	1.2	Reinforced Concrete (t=0.3 m)
Q5	Very Poor	Q<1	D	15	1.0	1.0	0.6 De	1.0	Reinforced Concrete (t=0.3 m)

De means diameter up to the excavation surface

Table 8.5.1 Design Values

Item	Dolomite	Phyllite	Slate	Distressed Zone Ⅰ	Distressed Zone Ⅱ
Modulus of Deformation (MPa)	3,000	1,000	1,000	1,200	2,100
Poisson's Ratio	0.24	0.21	0.18	0.29	0.24
Shear strength (MPa)	2.5	1.2	1.2	1	1.7
Internal friction angle (°)	50	45	45	39	44.5
Unit weight (kN/m^3)	26	26	26	26	26
Tensile strength (MPa)	3.9	2.9	2.9	1.6	2.7

Table 8.8.1 Design Data of Hydro-mechanical Equipment

(1) Gate

Item	Type	Quantity	Dimensions	Remarks
Intake Gate	Steel made fixed gate	1	5.0 m×5.0 m	Design Head 31.7 m
Intake Fixed Trashrack	Steel made slant trashrack	1	5.0 m×35 m	
Sand Flash Gate	High pressure slide gate	2	3.0 m×2.0 m	Hydraulic cylinder hoist, Conduit 3.0 m×2.0 m×50m long
Bottom Outlet Gate	High pressure slide gate	1	2.0 m×2.0 m	Hydraulic cylinder hoist, Conduit 2.0 m×2.0 m×50m long
Diversion Gate	Slide Type Gate	1	3.0 m×3.0 m	
Draft Tube Gate	Vertical lift slide gate	1	5.0 m×2.0 m	Wire rope lift type electrically operated monorail hoist

(2) Penstock

Type	Quantity	Diameter	Length	Remarks
Tunnel Type	1lane with 1 bifurecation	3.4 m to 2.2 m	190 m	Max. static head: 135.3 m External pressure: GWL. 700 m – penstock center