Table 2.5-1 Summary of Puwa Khola Irrigation Water Supply Plan

No.	Project name	starting year	estimated cost	out-going	salient features	remarks
1	Puwa Khola-Maipo Khan irrigation plan	(2046) 1989	none	---	canal length = 10+300 Km Cultivated Command Area (C.C.A) = 250 ha	not on DIO's program at present
2	Thulo puwa khola ISP, Puwamajhuwa	(2048/49) 1991/92	Rs5,066,000	89.00%	canal length = 6+150 Km C.C.A = 175 ha discharge = 525 lit/sec	construction almost completed
3	Puwa Khola (upper) irrigation plan	1984	Rs16,436,400	5.60%	main canal = 13.3 Km branch canal = 10 Km tunneling = 450 m drop = 100 m discharge = 1,105 lit/sec C.C.A = 450 ha	not on DIO's program at present
4	Puwa Khola (middle)irrigation plan	(2045) 1988	none	Rs 6.0 lakh	canal length = 12 Km canal slope : 1/500~1/10000 discharge = 1,100 lit/sec C.C.A = 500 ha (Ilam N.P.-3,4, 5&6 blocks) tunneling = 200 m	8 Km of canal digging was made only, not on DIO's program at present
5	Lamaduwali ISP	(2047/48) 1990/91	Rs2,400,000	100%	canal length = 5+5005 discharge = 240 lit/sec (200 lit/sec) C.C.A = 80 ha	under operation, located at d/s of F/S-project intake site

Remarks : 1.DIO: District Irrigation Office
2.ISP : Irrigation Water Supply Project

CHAPTER 3

CHAPTER 3 CONDITIONS IN THE PROJECT AREA

3.1 General

The Study area is located in Ilam district of the Mechi zone in the Eastern Development Region of Nepal, and includes Ilam N.P. The Study area is shown in the location map.

Ilam district is situated at N. Latitude 26°40'~27°8' and E. Longitude 87°40'~88° 10' with total land area of 1,703 km^2 and population of 229,200 (1991). On its eastern side the district is adjacent to the West Bengal state of India, with the border comprising the Mechi river and Singal mountain range. The western side of the district is contiguous to the districts of Morang and Dankuta, while the northern side touches on Panchitar district and the southern side borders on Japa district.

Due to generally high elevations in the district, temperature range is large, with maximum temperature at 35°C and minimum temperature at 0°C. The district falls within a high rainfall zone with mean annual precipitation at 2,400 mm.

Major rivers in the district are the Mechi Riner, Mai khola, Deumai khola, Jogmai khola and Puwa khola. With the exception of the Mechi river, all of these empty into the Mai khola which in turn flows into the Kankai river.

The population of the district is a mosaic of ethnic groups and languages. Speakers of Nepalese comprise 64% of the district inhabitants. Speakers of other languages include Lai (14%), Limbu (9%), Magar (4.5%), Taman (4%), Bodesherpa (1.5%), Grun (1%), as well as various other languages.

Occupation by sector is 96% in agriculture, 2% in public service, 1% in commercial activity, and only 0.1% in industry.

3.2 Industry

(1) Agriculture and Animal Husbandry

Agriculture and animal husbandry are the mainstays of the district economy. Major crops cultivated, areas cultivated and production totals are given below:

	Cultivated area (ha)	Production (m. ton)
Rice	12,400	27,300
Maize	15,100	22,700
Cane	2,900	2,600
Wheat	3,100	4,100
Potato	3,200	20,800
Vegetables		2,900
Tea	400	--
Cardamon	740	--

The district is self sufficient in staples, with about 20% of total staple production being exported outside the district.

Principal livestock raised in the district are:

	nos.
Cattle	73,500
Water buffalo	20,000
Sheep	6,200
Goats	72,500
Hogs	11,200
Poultry	159,800
Water fowl	700
Total	343,900

Total number of livestock is 1.5 times the human population of the district.

(2) Tea

Located at relatively similar latitude and elevation as Darjeeling, India, Ilam district with its abundant rainfall is a major tea growing area in Nepal, producing almost all tea consumed in the country. Both government run and private tea estates exist in the area; however, government run estates are both larger and better equipped for large scale tea production.

1) Tea Estates under the Nepal Tea Development Corporation

At present, 4 government run tea estates exist in the area. Of these, the Kanyam estate is the largest.

Table 3.2-1 Government Tea Estates

Tea estate	Ilam	Kanyam	Soktim	Chilimkot
Total area (ha)	55	215	97	--
Cultivated area (ha)	48	174	73	--
Employees (permanent)	30	50 (28)	40	--
Seasonal labor	200	450	235	--
Production (t)	38	85 (200)	84	--
Power generating capacity (KVA)	(70)	(212)		

Note: according to 1986 statistics. ( ) indicates figures obtained during this Study.

With the exception of fire drying, processing of tea (agitation, blowing, rolling etc.) is done by electrically operated equipment and machinery. Power is supplied by private generators at the estates.

2) Private Tea Estates

There are 8 private tea estates in the area centered on Kanyam and Phikkal. Total estate area is 93 ha which is roughly equivalent to the area at the government run Soktim estate.

Table 3.2-2 Area of Private Tea Estate

Tea estate name	Devi Bhakta	Mayalu	Punam Lapcha	Dipali	Shreeantu	Maya	Bakabari	Laxmi
VDC	Kanyam	Kanyam	Kanyam	Kanyam	Shreeantu	Phikkal	Phikkal	Laxmipur
Area (ha)	15	1.2	15	5.6	24	1	24	7

3) Cardamom Farm

Cardamom spice, prized for its aroma and taste, is produced in the Ilam district. Total production of this spice in the district is 440 tons per year, which accounts for 60% of the total cardamom spice yield in Nepal of 740 tons. Cardamom production is under the jurisdiction of the Tea Development Corporation, and is cultivated in valley bottoms where shade is ample and soil relatively moist. At present, drying is by wood fire; however quality control is difficult and use of wood contributes to destruction of the natural forest. In order to produce high quality cardamom spice, producers express strong desire for access to power to operate electrical equipment for drying.

4) Other Industries

In general, industry remains undeveloped in Ilam district as a whole. However, animal husbandry is widespread with milk production (both cattle and carabao) at 20,000 *l* daily. Of this, 12,000 *l* is shipped outside the district. Animal husbandry is particularly prevalent in the northeast part of the district (Gorkhe, Paspatinagar, Samalbung, Sthreeantu, Kanyam, Naya Bazar, Soyang) and Mangarbare in the west of Ilam Bazar where cheese, butter and other dairy products are produced for shipment to Kathmandu and other major population centers outside Ilam district. In the case of the cheese factory at Paspatinagar visited in the course of the Study, the management expressed the desire for availability of 10 kW of power in order to process 6,000 kg of milk daily in the future. The present power available is 5 kW, being utilized to process 3,000 kg of milk daily.

In addition as indicated in Table 3-3 various small scale industries are concentrated in and around Ilam N.P.; however, given the scale of these, their demand for power is considered minimal.

Table 3.2-3 Small Industries in Ilam District

Area No.	1	2	3	4	5	6	7	8	9	Total
Main Village / Industry	Ilam	Mainajhuw	Shantidand	Fakfok	Banjho	Soyak	Jogmi	Gorkhe	Shantipar	
	Barbote	Puwamaju	Mangalbore	Ektoppa	Sakfara	Sidhithumk	Mabu	Pastatinagar	Kolbung	
	Sumbek	Chameito	Jeetpur	Amchowk	Ivang	Chisapani	Jamuna	Fikkal	Irauntar	
	Sulbunk	Maipokhari	Dhuseni	Fuetoppa	Dahakari	Godak	Pyang	Shreeantu	Samalbung	
		Sakhejung	Sangrumba	Gojumakhi		Darabari	Namsbing	Panchakyanya		
							Soyang	Naya-Bazar		
Dairy		1(0)	3(0)				6(2)	5(3)	3(2)	18(7)
Hosiery	5(5)								1(1)	6(6)
Water-Mill				1(0)		1(1)		2(0)	2(0)	6(1)
Fabric			3(0)					1(0)		4(0)
Rice-Mill	4(4)									4(4)
Sewing	2(2)									2(2)
Brick	2(2)									2(2)
Bread	1(1)									1(1)
Stationary	1(1)									1(1)
Hotel	1(1)									1(1)
Shoe	1(1)									1(1)
Furniture	1(1)									1(1)
Total	18(18)	1(0)	6(0)	1(0)		1(1)	6(2)	9(4)	5(2)	47(27)

Note:

1. According to 1988 statistics (source: Ilam Profile)
2. ( ) indicates figures for the Project area
3. Commercial operations such as tailors, shoe shops, carpet sellers, hateries, etc. are not included

3.3 Transportation to Ilam N.P. and the Project Site

Ilam Bazar can be reached by car from Kathmandu. It is also possible to take a one hour flight from Kathmandu as far as Biratnagar, normally a 12 hour car journey. The remaining distance from Biratnagar via Char Ali can be covered in approximately 5 hours by car.

Equipment and materials shipped from abroad are unloaded at Calcutta port in India and carried overland via Jogbani adjacent to the Indian border to Biratnagar from which they are then transported through Nepalese territory to Ilam Bazar.

Access to the area is summarized in the following diagram.

	(one hour by airplane)		
Kathmandu	400 km km	Biratnagar	12 hours by car
Calcutta (Jobani)	6 km	Biratnagar	
Biratnagar	100km	Charali	
Charari	80km	Ilam Bazar	5 hours by car

There is a small road from Ilam Bazar to the Puwa Khola intake site but an additional access road of about 2 km is necessary to transport materials and equipment for the Project.

As for the Mai Khola power house site, it is possible for vehicles to cross the river during low flow in the dry season. However, construction of an access road of about 300 m in length along the left bank from the bridge on the Mai Khola will be required for transportation during the rest of the year as well as for future maintenance.

In consideration of the overall topography of Nepal, transportation conditions to the site are good.

3.4 General Meteorology

Meteorological observations in Nepal are performed by the Hydrology and Meteorology Department of the Ministry of Water Resources. As indicated in Figure 3.4-1, there are 3 weather observation stations upstream of the power house site, and in the vicinity of Ilam. Observation periods for these 3 stations as well as the weather station in the Kankai river regime in the south of the Project area are as follows:

Station no.	Name	Date observations commenced	Date observations finished
ST. 1407	Ilam Tea Estate	June 1966	
ST 1410	HimalGaun	February 1968	
ST 1417	Jaubari	June 1973	August 1981
ST 1411	Soktim tea estate	June 1960	

3.4.1 Temperature

Observations from ST 1407 above are indicated in Table 3.4-1. As can be seen in the table, average temperature is 18.5~19.5°C. In terms of monthly mean temperature, December~February exhibit low temperatures around 11~14°C, with higher temperatures in May~September at 20~23°C. Annual minimum temperature is most common in January, dropping as low as 0°C. Annual maximum temperature occurs during July~August reaching 35°C.

Table 3.4 -1 Monthly Mean Temperature at ST 1407

No. : 1407
Station : Ilam Tea Estate
Latitude : 26°55'N
Longitude : 87°54'E
Elevation : 1300 m

(°C)

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	nn.ave.
1971	--	14.9	18.1	16.7	16.9	18.2	18.8	18.3	17.7	16.4	12.8	10.3	--
1972	11.7	11.7	18.5	20.3	22.1	22.5	22.3	22.6	21.2	19.6	17.5	14.6	18.7
1973	12.6	15.3	18.8	22.8	21.2	21.6	22.8	22.6	22.4	20.2	17.6	14.2	19.3
1974	12.3	14.0	18.3	20.1	20.7	21.3	22.8	25.0	23.4	23.9	17.6	12.4	19.3
1975	12.7	15.0	19.2	21.9	21.5	22.2	21.4	22.8	21.2	21.0	16.6	13.2	19.1
1976	12.6	13.9	19.4	21.9	20.8	21.6	21.8	21.5	21.6	20.1	17.4	13.6	18.8
1977	12.1	14.4	19.9	18.7	19.8	21.4	22.0	22.2	22.0	18.9	16.4	13.5	18.4
1978	11.2	13.2	17.6	20.8	21.4	22.0	22.0	23.0	21.6	20.2	16.3	15.1	18.7
1979	13.4	13.5	18.5	21.7	23.6	23.8	22.1	22.9	21.7	20.0	17.8	13.6	19.4
1980	12.0	13.6	17.2	22.8	20.2	22.6	22.5	22.2	22.0	19.4	17.8	14.8	18.9
1981	12.2	14.6	17.4	18.8	20.6	22.6	22.4	22.4	21.9	20.5	17.6	14.1	18.8
1982	14.1	13.0	17.1	19.7	22.8	22.0	22.4	22.8	21.8	20.3	16.0	13.2	18.8
1983	11.6	13.0	17.6	19.8	20.7	22.7	22.5	22.7	22.2	21.5	18.4	13.1	18.8
1984	11.3	14.1	19.6	22.0	21.4	22.9	21.7	23.2	21.1	20.7	17.9	14.2	19.2
1985	12.7	13.5	19.1	21.9	21.3	22.6	22.0	23.0	21.7	20.4	17.1	15.1	19.2
1986	13.6	15.0	19.8	19.9	20.9	22.9	22.0	23.0	21.0	18.9	16.9	14.0	19.0
Average	12.4	13.9	18.5	20.6	21.0	22.1	22.0	22.5	21.5	20.1	17.0	13.7	18.8

3.4.2 Precipitation

Precipitation in the Ilam area is indicated in Table 3.4-2~3.4-5. ST 1407 and ST 1410 are located at ridge tops of EL 1,300 m and EL 1,650 m, respectively, and ST 1,417 is located at the upper reaches of the Mai khola at EL 3,050. Total annual precipitation is 1,300~2,500 mm (20 year average of 1,700 mm) at ST 1407, 1,800~3,000 mm (19 year average of 2,400 mm) at ST 1410, and 2,700~3,500 mm (4 year average of 3,200 mm) at ST 1417. Precipitation shows a tendency to be greater at higher elevation, and in the upper catchment area. On the other hand, ST 1411 is at EL 530 m, with high rainfall of 2,000~3,000 mm (16 year average of 2,600 mm). In comparison with the annual mean precipitation for the country as a whole of 1,400~1,500 mm, rainfall in Ilam is relatively greater.

The rainy season in Nepal as a whole is from June to September, with the dry season extending during the remaining months of the year. The Ilam area shows this same seasonal pattern. In terms of month wise rainfall, October~March in the dry season show the least precipitation, with rainfall beginning to increase in April and May, reaching peak concentration in June~September. 70~80% of total annual rainfall occurs in the said June~September period.

Table 3.4-2 Precipitation at ST 1407

No. : 1407
Station : Ilam Tea Estate
Latitude : 26°55'N
Longitude : 87°54'E
Elevation : 1300 m

(mm)

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Total
1971	1.0	1.0	6.0	223.0	140.0	352.0	212.0	271.0	182.0	24.0	9.0	0.0	1421.0
1972	14.0	30.0	16.0	48.0	82.0	292.0	326.0	144.0	370	44.0	5.0	0.0	1371.0
1973	12.0	0.0	14.0	9.0	164.0	403.0	307.0	220.0	206.0	216.0	32.0	0.0	1583.0
1974	24.0	0.0	35.0	59.0	170.0	316.0	740.0	316.0	196.0	79.0	0.0	2.0	1937.0
1975	13.0	4.0	3.0	73.0	108.0	493.0	646.0	134.0	384.0	15.0	0.0	2.0	1875.0
1976	14.0	48.0	0.0	63.0	206.0	301.0	327.0	499.0	124.0	10.0	0.0	0.0	1592.0
1977	0.0	0.0	4.0	125.0	112.0	147.0	281.0	313.0	107.0	189.0	20.0	23.0	1321.0
1978	9.0	2.0	104.0	64.0	180.0	276.0	285.0	237.0	151.0	66.0	37.0	0.0	1411.0
1979	2.0	28.0	0.0	41.0	33.0	154.0	475.0	257.0	218.0	111.0	8.0	77.0	1404.0
1980	0.0	0.0	22.0	1.0	204.0	192.0	412.0	270.0	222.0	123.0	0.0	0.0	1446.0
1981	12.0	0.0	28.0	95.0	167.0	306.0	686.0	435.0	136.0	0.0	0.0	0.0	1865.0
1982	0.0	4.0	34.0	59.0	54.0	330.0	436.0	100.0	185.0	62.0	5.0	0.0	1269.0
1983	18.0	1.0	7.0	32.0	150.0	279.0	681.0	287.0	162.0	27.0	0.0	10.0	1654.0
1984	18.0	18.0	0.0	80.0	86.0	373.0	551.0	182.0	421.0	25.0	0.0	0.0	1754.0
1985	0.0	29.0	2.0	33.0	143.0	206.0	732.0	170.0	264.0	199.0	7.0	25.0	1810.0
1986	0.0	2.0	4.0	179.0	155.0	137.0	437.0	224.0	447.0	98.0	19.0	12.0	1714.0
1987	5.0	33.5	75.0	72.0	135.0	250.0	448.0	857.5	469.5	173.0	26.0	4.0	2548.5
1988	6.0	24.0	39.0	78.5	200.0	108.5	544.0	481.5	100.8	10.0	49.0	0.0	1641.0
1989	44.0	46.5	24.5	9.5	193.5	428.0	511.0	234.5	557.0	48.5	3.0	11.5	2111.5
1990	0.0	56.0	46.5	58.0	205.0	423.5	370.5	349.0	367.0	62.0	0.0	0.0	1937.5
Average	9.6	16.4	23.3	70.2	144.4	288.4	470.5	299.1	263.5	79.1	11.0	8.4	1683.3

Table 3.4-3 Precipitation at ST 1410

No. : 1410
Station : Himali Gaun
Latitude : 26°53'N
Longitude : 88°02'E
Elevation : 1654 m

(mm)

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Total
1971	--	--	--	173.0	185.0	318.0	520.0	372.0	281.0	229.0	6.0	0.0	--
1972	8.0	32.0	30.0	100.0	98.0	318.0	512.0	316.0	509.0	30.0	12.0	0.0	1965.0
1973	16.0	11.0	14.0	80.0	150.0	607.0	398.0	393.0	306.0	282.0	66.0	0.0	2246.0
1974	20.0	0.0	14.0	72.0	188.0	460.0	872.0	365.0	323.0	19.0	0.0	0.0	2333.0
1975	6.0	3.0	0.0	44.0	143.0	874.0	845.0	181.0	634.0	24.0	0.0	0.0	2754.0
1976	14.0	57.0	0.0	89.0	171.0	690.0	296.0	531.0	288.0	44.0	0.0	0.0	2180.0
1977	0.0	8.0	37.0	162.0	172.0	217.0	544.0	447.0	229.0	260.0	63.0	26.0	2165.0
1978	10.0	28.0	87.0	77.0	257.0	520.0	619.0	385.0	238.0	62.0	55.0	0.0	2338.0
1979	8.0	54.0	0.0	91.0	45.0	394.0	780.0	620.0	294.0	113.0	13.0	86.0	2498.0
1980	2.0	5.0	39.0	1.0	238.0	289.0	523.0	468.0	240.0	96.0	0.0	0.0	1901.0
1981	26.0	10.0	45.0	178.0	264.0	291.0	728.0	397.0	294.0	4.0	6.0	0.0	2243.0
1982	0.0	13.0	27.0	93.0	46.0	505.0	407.0	128.0	424.0	114.0	39.0	4.0	1800.0
1983	24.0	32.0	9.0	40.0	227.0	447.0	1095.0	339.0	315.0	31.0	0.0	36.0	2595.0
1984	31.0	25.0	11.0	158.0	175.0	637.0	751.0	384.0	506.0	39.0	0.0	0.0	2719.0
1985	5.0	26.0	13.0	46.0	122.0	325.0	854.0	285.0	308.0	304.0	35.0	43.0	2366.0
1986	0.0	2.0	5.0	151.0	132.0	391.0	622.0	259.0	498.0	86.0	15.0	42.0	2203.0
1987	0.0	14.5	107.2	116.5	130.5	349.7	795.4	827.9	477.3	241.6	0.0	10.5	3071.1
1988	0.0	28.1	38.3	139.1	182.0	214.6	770.5	890.1	248.9	10.6	0.0	8.3	2530.5
1989	52.3	59.0	36.9	12.3	438.1	621.2	692.7	345.3	640.9	83.2	44.7	12.3	3038.9
1990	0.5	59.0	71.2	56.5	278.9	486.9	664.7	493.5	414.3	137.8	0.0	0.0	2663.3
Average*	11.7	24.6	30.8	90.4	182.1	447.8	664.2	421.3	373.4	110.5	17.7	13.4	2400.5

* Average values are calculated excluding missing observation data.

Table 3.4-4 Precipitation at ST 1417

No. : 1417
Station : Jaubari
Latitude : 27°04'N
Longitude : 88°00'E
Elevation : 3050 m

(mm)

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Total
1973	--	--	--	--	491.0	599.0	499.0	503.0	411.0	239.0	32.0	0.0	--
1974	--	--	--	--	--	--	--	--	--	--	--	--	--
1975	--	--	--	--	--	--	--	--	--	--	--	--	--
1976	0.0	6.0	4.0	170.0	243.0	492.0	760.0	1062.0	391.0	97.0	16.0	2.0	3243.0
1977	3.0	4.0	64.0	174.0	290.0	518.0	886.0	830.0	369.0	291.0	68.0	34.0	3531.0
1978	38.0	36.0	60.0	59.0	450.0	829.0	735.0	516.0	487.0	164.0	52.0	6.0	3432.0
1979	--	13.0	0.0	128.0	--	462.0	777.0	532.0	323.0	255.0	32.0	91.0	--
1980	0.0	9.0	102.0	114.0	427.0	362.0	705.0	617.0	385.0	6.0	0.0	0.0	2727.0
1981	31.0	4.0	141.0	143.0	253.0	274.0	--	--	--	--	--	--	
Average*	14.4	12.0	61.8	131.3	359.0	505.1	727.0	676.7	394.3	175.3	33.3	22.2	3233.3

* Average values are calculated excluding missing observation data.

No.	: 1411
Station	: Soktim Tea Estate
Latitude	: 26°48'N
Longitude	: 87°54'E
Elevation	: 530 m

Table 3.4-5 Precipitation at ST 1411

(mm)

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Total
1971	--	--	(258.0)	(206.0)	(254.0)	(454.0)	(854.0)	(915.0)	(1,015.0)	(246.0)	(0.0)	(0.0)	--
1972	23.0	0.0	12.0	88.0	150.0	559.0	760.0	644.0	472.0	48.0	10.0	0.0	2,766.0
1973	4.0	54.0	4.0	9.0	141.0	751.0	455.0	571.0	191.0	178.0	21.0	0.0	2,379.0
1974	29.0	0.0	21.0	44.0	128.0	486.0	1075.0	395.0	260.0	210.0	0.0	0.0	2,648.0
1975	0.0	1.0	2.0	34.0	178.0	923.0	529.0	150.0	839.0	82.0	3.0	0.0	2,741.0
1976													
1977													
1978								(282.0)	(368.0)	(0.0)	(12.0)	(0.0)	--
1979	0.0	47.0	0.0	29.0	78.0	281.0	923.0	565.0	341.0	72.0	17.0	109.0	2,462.0
1980	0.0	2.0	69.0	0.0	109.0	207.0	470.0	353.0	392.0	180.0	0.0	0.0	1,782.0
1981	29.0	24.0	29.0	155.0	201.0	334.0	646.0	413.0	384.0	25.0	0.0	0.0	2,240.0
1982	0.0	7.0	14.0	246.0	148.0	503.0	522.0	193.0	374.0	74.0	41.0	19.0	2,141.0
1983	26.0	2.0	4.0	15.0	176.0	334.0	688.0	364.0	403.0	50.0	0.0	6.0	2,068.0
1984	28.0	19.0	7.0	80.0	128.0	541.0	758.0	338.0	575.0	45.0	0.0	0.0	2,519.0
1985	0.0	12.0	48.0	32.0	259.0	322.0	919.0	398.0	316.0	324.0	7.0	54.0	2,691.0
1986	0.0	7.0	0.0	102.0	174.0	303.0	866.0	287.0	570.0	101.0	18.0	12.0	2,440.0
1987	3.1	27.5	70.3	104.2	100.0	305.8	603.8	998.3	618.5	246.0	0.0	2.1	3,079.6
1988	12.2	35.6	52.0	116.8	168.0	175.6	1036.8	988.0	388.5	18.0	76.0	3.0	3,070.5
1989	33.0	48.5	2.0	2.0	408.0	646.0	604.0	451.0	603.0	93.0	64.0	39.0	2,993.5
1990	0.0	63.5	6.0	35.0	480.0	550.0	1353.0	1219.0	501.0	0.0	0.0	0.0	4,207.5
Average*	11.7	21.9	21.3	68.3	189.1	451.3	763.0	520.5	451.8	109.1	16.1	15.3	2639.3

* Average values are calculated excluding missing observation data.

3.5 Topography

The Kingdom of Nepal extends east~west at the north of the Indian subcontinent. The country measures 800 km east~west and 120~200 km north~south. The northern extremity of the country comprises the high Himalayas ("the roof of the world"), with the highest peak being Chomoranma (Mt. Everest) with elevation of 8,848 m, and other tall peaks including Kanchanjunga (8,598 m) in the east and Apihimal (7,132 m) in the west.

Ilam district wherein the Project area is located is at the extreme east of the country, directly across the Indian border from Darjeeling. The district capital is Ilam N.P. in the central southeast of the district. Ilam N.P. is a hilltop township situated on a ridge at EL 1,200~1,300 m.

The Study area encompasses the Puwa khola intake site to the west of the ridge, and Mai khola at the southwest of the ridge, measuring 5 km north~south and 3 km east~west.

The river regime of the Study area comprises the Puwa khola, and the Mai khola into which the Puwa khola flows. The Mai khola is one of the main tributaries of the Kankai river, which traverses the Terai plain to empty into the Ganges river. The Ganges river continues eastward to eventually flow into the Bay of Bengal.

The mountain system in the Study area comprises the Ilam ridge which arcs from its starting point at Ilam N.P. at elevation 1,250 m for about 1 km (the extent of the township of Ilam Bazar) to the west before turning south, steadily losing elevation for about 2.5 km to the pass at Bhanjyang (EL 650 m). From there, the ridge turns southwest for about 1 km before again altering direction to the west. The Ilam ridge is generally asymmetric, with steep slope of 45° facing on the Puwa khola. Outcrops of base rock are numerous. The slope fronting on the Mai khola is around 30°, largely cultivated under terraced upland (maize) and paddy field.

3.6 Geology

3.6.1. Regional Geology

The Higher Himalayas, the Lesser Himalayas, the Siwaliks (Subhimalayas) and the Terai are the four major geological regions of Nepal. The Higher Himalayas (over 4,000 m elevation) and the Terai are the belts which lie at Nepal's northern and southern boundaries respectively. The Lesser Himalayas form a wide belt of mountainous terrain lying to the south of the Higher Himalayas. The Siwaliks lie between the Lesser Himalayas in the north and the Terai in the south.

The Higher Himalayas are mainly composed of gneisses and mica schist, and are perpetually covered by snow. The Main Central Thrust (MCT) is the tectonic structure that separates the Higher Himalayas from the Lesser Himalayas. The Lesser Himalayas are mainly composed of phyllites, quartzites, schists, limestones, dolomites, gneisses, etc. At some places, the above-mentioned lithologies are interbedded with one another .

The Siwaliks, which are very often termed as the Subhimalayas, are separated from the Lesser Himalayas by a major tectonic fault system, the Main Boundary Thrust (MBT). The Subhimalayas are composed of mollase formation represented mainly by sandstones, mudstones, shales, sandy beds, and conglomerates. The Subhimalayas are separated from the Terai region by the Himalayan Frontal Faults (HFF). The Terai is the southern most geological belt composed of thick alluvium deposited by the Nepalese tributaries of the Ganges. The Higher Himalayas, the Lesser Himalayas and the subhimalayas are dissected by tear faults. The rock foundation of the Terai region lies very deep due to which its tectonic feature is less known.

The Project area lies within the Lesser Himalayas.

The Lesser Himalayas form a strip of land stretching from east to west composed of valleys and ridges of various relief, morphology and topography.

The Lesser Himalayas are bordered on the north by the Main Central Thrust (MCT) and on the south by the Main Boundary Thrust (MBT).

Hard rocks such as gneisses are, very often, interbedded with weaker soft rocks such as phyllites and schists, due to which the overall stability of these rock masses has decreased. High intensity rainfall and high intensity of seismic activity play a dominant role in further decreasing the stability of the rock masses. As a result, the region is landslide prone, and many of these landslide areas are active.

The Project area lies in the Kathmandu Group of Lesser Himalaya characterized by the presence of metamorphic rocks represented by schist, phyllite, etc. The rocks of the Kathmandu Group are separated from the rocks of the Higher Himalayas in the north and rocks of the Siwalik Group in the south.

3.6.2 Lithological Units of the Project Area

The lithofacies of the Project area is composed of the rocks of the Kathmandu Group characterized of metamorphic rocks represented by the following schist, phyllite, etc. Fig 3.6-1 gives the distribution of the group.

(1) Mica-Schist

Mica-schist has been mapped extensively at the weir site, along headrace tunnel alignment and powerhouse sites. This is the most dominating lithological unit of the Project area. Minerologically, it is mainly composed of quartz, feldspar, muscovite and biotite; more specifically, this can be divided into muscovite and biotite schist. The rock has appeared in various states of weathering starting from fresh to completely decomposed residual soil. From a rock engineering standpoint, it can be said to show the following characteristics:

a: The foliation of the rock varies from place to place, with high anistropic characteristics.

b: This rock is less jointed due to ductile behavior of the rock.

c. Highly ductile nature due to foliation suppresses shear, and rock exhibits high continuity.

(2) Quartzite

This unit has been mapped interbedded with mica-schist at several places at the weir site and part of the headrace tunnel alignment, and generally exhibits lenticle structure. Quartzite is composed mainly of quartz with some mica flakes and iron oxide staining. It has 2 to 3 sets of joints. The weathering varies from fresh to moderate. The colour varies from white to brownish grey. The thickness of quartzite varies from 0.5 to 2.0 meters; however, in many cases overall formation thickness is 5~20 m consisting of several to several tens of overlaying simple layers.

(3) Gneissose-Schist

This unit has been observed in the power house site and along the tributaries of Mai khola joining from the right bank. It shows higher grade of metamorphism than the mica-schist and is composed of well developed grains of feldspar, quartz, biotite and muscovite. The thickness of the unit varies from place to place but is generally within 20 m. The degree of weathering varies from slight to moderate. The unit is more jointed than the mica schist with two to three sets of jointing noted.

(4) Residual Soil

Residual soil is a collective term for highly weathered rock originally found in-situ. The Project area has a relatively high average annual precipitation amounting to about 2,400 mm; rain intensity is also strong, particularly in the rainy season. Due to this fact and high susceptibility of mica-schist to weathering, a thick weathering profile, especially, has been developed on the right bank of the Mai khola. The foregoing were confirmed by drill hole no B-1 located in the vicinity of junction of the headrace tunnel and head tank. The residual soil originating from weathered mica schist is composed of clay and gravel size rock fragments, composed of quartz and feldspar and mica flakes. Nevertheless, the original texture and structure of the source rock is well preserved, as confirmed through surface reconnaissance of the area.

(5) Colluvial Deposit

A large quantity of colluvial deposits have been mapped on several slopes on the banks of Puwa khola and Mai khola and their tributaries. The texture of these deposits vary from clay to large boulders. The angular to subangular rock fragments are composed of mica-schist, quartzite and gneissose -schist. The thickness varies from place to place depending upon the topographical setting of the area and ranges from less than 1.0 m to about 10 m and more.

(6) Lower Terrace Deposit

The terrace deposits lie along the Puwa khola and Mai khola and comprise principally gravel intercalated with sandy layer. At the surface, large boulders of 50 cm ~ 3 m size are numerous. The terrace deposits are loose to semiconsolidated.

(7) Recent River Gravel Deposit

The recent river gravel deposits are found along the Puwa khola and Mai khola and their tributaries. These deposits are composed of rounded to subrounded material derived from quartzite, gneiss, amphibolite, etc.. A large quantity of silty material has also been found mixed with these sediments.

3.6.3 Engineering Geology for Hydropower Structures

(1) Weir Site

As indicated in Figure 3.6-2, the weir site is an asymmetrical 'V' shaped valley with left bank steeper than right bank. The river banks form vertical cliffs. The river valley at the proposed weir axis is roughly 29 m wide and the width of the river channel is around 12.5 m. The river valley is composed of young alluvial sediments of different size ranging from silt to boulders of quartzite, gneiss, amphibolite etc. The thickness of this alluvium is about 2.0 meters.

The rock cliff present on the left bank of the weir site is composed of fractured to blocky biotite schist with muscovite schist dipping downstream. The striking and dipping of rock is N82°W22°S. The major discontinuity systems on this bank have striking and dipping of N50°E20°S and N77°E68°E, respectively.

The right bank is composed of similar rock, but the striking and dipping is N80E°28°S. The major joint systems have striking and dipping of N76°E66°SE and N35E°88°S, respectively.

The rock on the left bank of the river is more stable than on the right bank due to the orientation of the bed rock.

The 4.0 m high weir can be founded on mica schist. The weir could be anchored to the bed rock present on both banks of the river for stability.

No major instabilities had been observed in the headwork area except small and insignificant slides in the vicinity.

(2) Intake Site

The intake structure can be located on the left bank of Puwa khola in similar rock condition as that of the left bank of the weir site. Striking and dipping are N98° W22°S.

(3) Headrace Tunnel Alignment

The headrace tunnel alignment passes through a fairly stable area. The lithological units expected along the alignment are mainly mica schist with some quartzite layers up to the head tank area, however the last 200 m of the headrace tunnel are residual soil and talus material. Due to inconsistent nature of dipping due to local folding and micro-folding of the rock, the headrace tunnel is expected to make different angles with the bed rock at various locations. The surface geology along the headrace tunnel alignment suggest the presence of many narrow weak zones along the tunnel. These weak zones could be stabilized by using appropriate engineering techniques. The last part of the tunnel may be critical because of the presence of overburden at that section.

(4) Head Tank Site

As shown in Figure 3.6-3, the head tank site is a gently sloping area composed mainly of residual soil and some colluvial deposit. The bore hole drilled in this area (B-1) suggests an overburden thickness of at least 33.0 m. The excavation work for the head tank area will be easy to conduct due to presence of residual soil.

(5) Penstock Alignment

As shown in Figure 3.6-4~8, the proposed penstock alignment passes through a gently sloping area except the last part close to the power house site, where the slope is about 40°. The depth of overburden along the penstock alignment varies from less than 1.0 meter to about 2.0 meters. The gentle section of the penstock alignment is composed of moderate to highly weathered mica-schist. The steep section of the proposed penstock alignment is composed of colluvium mixed with some residual soil. The slope of the steep section of the alignment is approximately parallel to the dip of the rock. Accordingly, minor rock slide along the dip may be expected. Small scale dip slips have also been observed in the vicinity. The anchor blocks for the penstocks should be founded on fresh rock after removing the weathered part.

(6) Powerhouse Site

As shown in Figure 3.6-9, the topography and geology of the proposed power house site suggest a surface power house. The power house site is composed of alluvium of the Mai khola. As indicated in Figure 3.6-12, the bore hole record of the power house (B -3) shows the thickness of overburden to be 8.9 m. Hard and massive mica schist underlies the terrace deposit . The right abutment of the Mai khola, in the vicinity of power house, will need some slope protection measures due to the possibility of dip slip.

(7) Tailrace site

The tailrace canal alignment is composed of river terrace deposit represented by semiconsolidated to unconsolidated sediments. The coarse part of the sediments is composed of quartzite, gneiss, etc., whereas the fine fraction is represented by silt. The thickness of overburden is 13.08 meters in bore hole No. B-4 as indicated in Figure 3.6-13.

3.6.4 Result of Seismic Refraction Survey

The obtained seismic results were interpreted for each site in comparison with the result of the ground surface observation mentioned above respectively. The detailed outcome of the exercise is presented in the following paragraphs.

1) Weir Site

Two seismic lines (SL - 1 and SL - 9) were executed at the weir site. The purpose of the seismic lines was to determine the thickness of talus and overburden deposit.

Generally, four different layers can be identified in the area. However, only three layers have been indicated in each seismic line in the drawings. The four layers are the following .

- Colluvial / alluvial deposit.
- Highly weathered and fractured rock
- Partly weathered and fractured rock
- Sound rock.

<u>Seismic line SL-1 and SL-9</u>

- <u>Colluvial / Alluvial Deposit of Weir Site</u>

This superficial layer has velocities of 540-630 m/s and 350-580 m/s along seismic lines SL - 1 and SL - 9 respectively. The layer velocity is higher in the cultivated area. The layer has a thickness of up to 11 m along SL - 1 and 5.5 in SL - 9. The layer pinches out on the steep slope.

- <u>Highly Weathered And Fractured Rock</u>

This layer has been characterized by velocities between 1,360 and 1,470 m/s. The velocity is higher in the lower portion of the left bank steep slope and on the right bank slope. The layer has a uniform thickness on the right bank as well as on the steep left bank slope. However, its thickness increases in the up slope area and varies between 3 and 12 m. The layer has not been identified in SL - 1.

- <u>Weathered to Partly Weathered and Fractured Rock</u>

The layer is prominent along SL - 1 and partly along SL- 9. This layer has been characterized by seismic velocities of 2,040 - 3,090 m/s along SL - 1 and 2,200 - 2,480 m/s along SL-9. The layer thickness varies from less than a meter to 17 m. As identified as a bottom refractor along SL - 9, velocity changes laterally up to 3,650 m/s, which represents sound rock in that section. A number of fractured zones have been identified along SL - 9, where the velocity drops to 1,720 m/s. The fractured zone is 14 to 17 m wide. The layer thickness varies from place to place and the maximum is determined to be about 17 m at some places. The maximum depth to the bottom refractor along SL - 9 is 12 m.

- <u>Sound Rock</u>

The bottom refractor along SL - 1 shows rather higher velocities measuring up to 3,650 - 5,370 m/s. The maximum depth of the refractor reaches up to 24 m in the up slope area and pinches close to the Puwa khola.

Higher velocities are observed on the left bank slope, where the velocity varies between 4,610 and 5,370 m/s.

2) Headrace Tunnel

The seismic lines SL - 2 SL - 3 and SL - 4 were executed at the headrace tunnel alignment. The total length of exploration was 2,700 m. Geophone spacing of 10 m was chosen along the alignment except in the 335 m long section along the SL-4 in the head tank area, where pegging was done at 5 m spacing.

Four layers have been identified along the headrace tunnel alignment. They are as follows:

- Loose surficial deposit
- Colluvial deposit / completely weathered to highly weathered fractured rock
- Highly weathered and fractured rock
- Partly weathered and fractured to sound rock

Seismic line SL-2

- Loose Surficial Deposit

The top layer has velocities between 400 and 670 m/s. The layer has uniform velocities with lateral variation up to 500 m/s. The maximum layer thickness reaches up to 13.5 m and has an average thickness of 7 m. The layer truncates the cliff area between chainage 0+760 and 0+780 m.

- Consolidated Colluvial Deposit / Decomposed Rock

This layer has velocities varying between 920 m and 1,370 m/s. The layer boundaries are distinct as the velocity contrasts are very sharp with the overlying and underlying layers. The thickness varies between 3 and 42.5 m. It truncates in the cliff areas in between chainage 0+760 and 0+730 m. Laterally, the layer has a uniform velocity with slight variation. The seismic velocities and surface geology indicate the presence of completely decomposed rock in this area.

- Highly Weathered and Fractured/ Weathered and Fractured Rock

This layer is not identified up to chainage 0 + 200 m in the first spread. However, it is traceable because its presence is marked by a sharp change in velocities with respect to the overlying as well as the underlying layers. The velocities vary between 1,730 and 2,400 m/s. The lateral velocity variation shows varying degree of weathering and fracturing in the rock. The layer thickness varies between 11 and 51 m. It has an average thickness of 30 m.

- Partly Weathered and Fractured Rock to Sound Rock

The bottom refractor shows the velocities between 2,500 and 5,060 m/s . In most of the section, the refractor velocity is above 3,000 m/s exhibiting sound rock condition. However, in some of the sections, the refractor has the velocities varying between 2,000 and 3,000 m/s indicating partly weathered and fractured rock. Relatively more fractured rocks of 47 m width are located in the section, where the seismic velocity is only 2,020 m/s . The depth to this refractor varies from 23 to 70 m.

Seismic Line SL-3

- Loose Colluvial Deposit

The superficial loose colluvial deposit has velocities varying between 330 and 590 m/s. This layer has a fairly uniform thickness averaging about 5 m and varying between 0 and 8.5 m . It pinches out in the steeper slope at the valley crossing.

- Colluvial Deposit / Completely Weathered Rock / Highly Weathered and Fractured Rock

This layer is rather uniformly distributed along the alignment. The layer shows velocities between 690 and 1690 m/s. Higher velocities above 1,000 m/s are determined up to chainage 0+360 m exhibiting highly weathered to completely decomposed and fractured rock, the lower velocities of 690 m/s are identified beyond chainage 0+360 m indicating consolidated colluvial deposit. The layer thickness varies between 5 and 18.5 m. with the average lying at about 12 m.

- Highly Weathered and Fractured to Weathered and Fractured Rock

This layer has velocities between 1,450 and 2,550 m/s. Weathering intensity is higher in the area after chainage 0+360 m, where the velocity of the layer varies between 1,450 and 1,990 m/s indicating the presence of highly weathered and fractured rock. The higher velocities above 2,000 m/s are determined up to chainage 0+360 m reflecting weathered and fractured rock. The layer thickness increases in the area having highly weathered and fractured rock. In general, thickness of weathered and fractured rock varies between 14 m and 37 m. Similarly, the thickness of highly weathered and fractured rock varies up to 77 m. The lower boundary of the layer in between chainages 0+190 m and 0+340 m is not very consistent and is shown by broken line in the seismic depth section (SL-3).

- Sound Rock

The bottom refractor has higher velocities between 3,250 and 5,000 m/s exhibiting sound rock at the depth. However, some of the sandwich zones show lower velocities of 1,670 and 2,500 m/s probably due to fracturing in the rock. The width of the fracture zone varies from 25 m to 100 m. The depth to the refractor varies between 24 and 96 m. Shallow depth is determined in the valley where the refractor is found at 24 m depth only.

Seismic Line SL-4

- Loose Surficial Deposit/ Decomposed Rock

Uniformly distributed top layer shows the velocities of 330 to 610 m/s. The layer consists of loose surficial deposit and decomposed rock in the area. Upslope area is mainly colluvial deposit. The layer thickness varies between 2 and 12.5 m with an average thickness of 5 m.

- Consolidated Colluvial Deposit/ Completely Weathered Rock

This layer has velocities between 560 and 1,100 m/s. As the rock is completely weathered in the area at that depth, it is not possible to draw the boundary between the consolidated colluvial deposit and completely weathered rock. However, the layer consists of predominately weathered rock. The layer thickness varies between 5 and 21 m with an average thickness of 15 m.

- Highly Weathered to Partly Weathered and Fractured Rock

The lateral velocities vary between 1,100 and 2,550 m/s. Weathering intensity is higher in the downslope area where the layer velocity is only 1,100 to 1,300 m/s. However, the velocities increase in the upslope area from 1,520 to 2,550 m/s exhibiting moderate to partly weathered and fractured rock. The lower boundary of the layer is quite consistent in the area which demarcates the bottom refractor with the higher velocities. The layer thickness varies up to 55 m with the average thickness of around 20 m.

- Partly Weathered and Fractured to Sound Rock

The bottom refractor shows the velocities between 2,280 and 4,990 m/s. Number of fracture zones are identified along the alignment where the velocity varies between 1,330 and 1,700 m/s. Fracture zones are most frequent in the vicinity of the head tank area. The width of the fracture zone varies between 19.5 and 36.5 m. Rest of the section has velocities higher than 2,200 m/s exhibiting partly weathered and fractured rock to sound rock. The depth to the bottom refractor varies between 2 m and 75 m.

3 Head Tank Area

Four layers are identified in the head tank area. They are as follows:

- Loose surficial deposit / decomposed rock
- Colluvial deposit / completely weathered rock
- Highly weathered and fractured to weathered and fractured rock
- Partly weathered and fractured to sound rock

Seismic line SL-7 and SL-8

- Loose Surficial Deposit/ Decomposed Rock

The layer has velocities between 300 and 520 m/s. The layer has fairly uniform velocities with slight variation up to 470 m/s. The layer thickness varies between 3 and 13 m. The layer velocity is not sharp laterally. Therefore, it is not possible to draw the boundary between decomposed rock and loose surficial deposit.

- Colluvial Deposit/ Completely Weathered Rock

The layer velocities vary between 520 and 870 m/s with the slight variation laterally. The layer thickness varies between 5 and 22 m. The layer velocities reflect probably colluvial deposit and/or completely weathered rock in the area.

- Highly Weathered to Weathered and Fractured Rock

The layer has velocities between 910 and 1,590 m/s. The lowest velocity of 910 m/s is determined along SL-5 and only slight increase in velocity along SL-4 from 1,100 to 1,240 m/s. Layer velocity is greater along SL-7 and SL-8 along which the velocity is between 1,520 and 1,590 m/s. The layer thickness varies between 9 and 40 m.

- Partly Weathered and Fractured to Sound Rock

The bottom refractor in the head tank area has the velocities between 2,280 and 3,780 m/s with a number of fracture zones along which the refractor shows the velocities between 1,070 and 1,960 m/s. The depth to the refractor varies up to 63 m as determined along SL-8.

4) Penstock and Powerhouse Site

Four different layers are identified along the penstock alignment (SL-5) and in the power house site (SL-6).

Seismic Line SL-5

Identified layers along the penstock alignment are as follows:

- loose colluvial deposit
- consolidated colluvial deposit/ completely weathered rock
- completely weathered rock/ highly weathered and fractured rock/ weathered and fractured rock
- weathered and fractured rock/ sound rock

- Loose Colluvial Deposit

The layer has velocities between 340 and 650 m/s. However, the layer has velocities predominately less than 500 m/s. The layer thickness varies between 1 and 7 m with an average thickness of 3.5 m.

- Consolidated Colluvial Deposit/ Completely Weathered Rock

Uniformly thick, this layer has velocities between 520 and 1140 m/s. The layer velocity increases in the downslope area above 800 m/s probably due to influence of groundwater and/or weathered rock. The layer thickness varies between 6 and 20 m with an average thickness of 10 m.

- Completely Weathered to Weathered and Fractured Rock

This layer shows velocities between 910 and 1,880 m/s. The layer shows higher velocities on the gentle slope in comparison to the flat section. Rock is rather completely weathered to highly weathered and fractured in the upslope area. The higher velocities in the downslope area exhibit weathered and fractured rock in the area. The lower boundary of the layer is sharp except in the section between chainages 0+285 and 0+360 m where the velocities are not in sharp contrast with the lower refractor.

- Weathered and Fractured to Sound Rock

The bottom refractor is distinctly identified in the downslope area where the velocity changes very sharply. The refractor has velocities between 2,030 and 4,530 m/s exhibiting weathered and fractured rock to sound rock. Number of fracture zones are determined in which the refractor shows velocities between 1,070 and 1,960 m/s with the width variation from 11.5 to more than 40 m.

Seismic Line SL-6

Four identified layers in the power house area and in the upslope area are as follows:

- loose colluvial deposit/ alluvial deposit
- consolidated colluvial deposit/ consolidated alluvial deposit
- weathered and fractured rock
- partly weathered and fractured to sound rock

- Loose Colluvial Deposit/ Alluvial Deposit

Top layer has velocities between 420 and 680 m/s. Hill slope consists of loose colluvial deposit with the velocities of 420 and 530 m/s. The layer has uniform thickness on the slope with an average thickness of 4 m. The alluvial deposit has velocities of 560 to 620 m/s. The layer pinches close to the Mai khola.

- Consolidated Colluvial Deposit/ Consolidated Alluvial Deposit

Consolidated colluvial deposit has velocities between 970 and 1,380 m/s. The layer shows higher velocities in the downslope area probably due to groundwater. The layer has fairly uniform thickness of 10 m. Consolidated alluvial deposit has velocity of 1320 m/s. Thickness of alluvial deposit varies between 6 and 13 m.

- Weathered and Fractured Rock

This layer shows fairly uniform velocities with slight variation between 2,370 and 2,770 m/s. The layer thickness varies greatly from 6 to 40 m. The lower boundary of the layer in the downslope area is not distinctly identified because of no sharp change in the refractor velocities. Therefore, the bottom refractor boundary is presented with broken lines.

- Partly Weathered and Fractured to Sound Rock

The bottom refractor has velocities between 2,530 and 4,120 m/s. No intensely fractured rocks are determined in the area. Only partly fractured rocks are determined in between chainages 0+06 m and 0+195 m. The depth to the refractor varies between 17 and 55 m.

3.6.5 Slope Stability

As the Project area is mountainous, large difference in topography is present. The slopes in and around the project area vary from cliffs to less than 5°. The Project area can be divided into two large topographical units clearly divided by Ilam ridge. The western part of the Ilam ridge is the left bank slope of the Puwa khola and the eastern part of the Ilam ridge is the right bank of the Mai khola.

Several types of landslide and slope collapse are seen in the Project area due to prevailing topography and geology. These are:

(1) Numerous and large scale sliding on the mountainside of the left bank of the Puwa khola, exhibiting surface layer collapse.
(2) Sliding on the gradual slope of the Mai khola, small scale in nature and mainly consisting of talus deposits
(3) Relatively large scale sliding along large fracture zone

Nevertheless, from a slope stability point of view, the Project area does not feature complex gradient structure, and may be categorised as a low hazard and low risk zone with regards to landsliding.

3.6.6 Field Exploration

(1) General

The subsurface investigation of the Project area was carried out by means of core drilling, trenching and seismic refraction. The exploration methodologies and the results are presented in detail in the following paragraphs.

(2) Core Drilling

Four drill holes namely B-1, B-2, B-3 and B-4 were drilled at the head tank and at the power house sites. The deepest drill hole had been drilled up to depth of 33.00 m at the head tank site, in which bed rock could not be encountered. Other drill holes measured 17.00, 11.06 and 14.00 meters respectively. The field exploration by drilling had been designed for finding out subsurface geological information at the headrace tunnel, head tank, powerhouse and tailrace areas. The drill holes encountered bed rock at various depths and elevations, the details of which are shown in the drilling core log presented in Figure 3.6-10~13. The bed rock encountered in the drill holes is composed of alternation of biotite schist and muscovite schist in a varying degree of weathering at the top and fresh mica schist at the lower part.

(3) Trenching

Due to the beginning of rainy season, the trench could be excavated with dimensions of 13.0 m long, 1.0 m deep, 4.0 m wide at the surface, and 1.0 m wide at the bottom. At the time of mapping of trench the river was flooded, so only 5.0 m of the trench (right bank of the Puwa khola) could be mapped with the information as outlined herein.

The rock at the bottom of the trench is medium to strong biotite rich schist with striking varying from N25°E to N85°E and dipping from 20°SE to 88°S. The difference in attitude is due to local folding of the mica schist. Two to three sets of discontinuities have been mapped. The joint systems have striking and dipping of N76°E66°SE, N35°E88°S, N50°E90°, etc. Recent river deposits occupy the vicinity of the bed rock.

(4) Seismic Refraction

(a) General

The seismic refraction survey was undertaken to investigate the subsurface geological conditions at various sites under the Project. The work consisted of refraction seismics at the weir, power house and intake sites as well as along the proposed alignments of tailrace canal and the headrace tunnel. The exploration had been conducted to evaluate the overall rock qualities and overburden extension.

In total, 4,780 running meters of seismic prospecting had been carried out; the methodology of prospecting and the findings of exploration are presented in the following pages. The location of seismic profiles is presented in Fig. 3.6-14(1), (2). The distribution of seismic lines at different sites is presented in Table 3.6-1.

Table 3.6-1 Distribution of Seismic Lines

S. No.	Location	Line No.	Length, m
1.	Weir Site	SL - 1 & SL - 9	185 & 150
2.	Headrace Tunnel	SL - 2	840, 630
		SL - 3 & SL - 4	and 1240
3.	Head Tank and	SL - 5, SL - 6	1060, 225
	Powerhouse Sites	SL - 7 & SL - 8	225 & 225
		9 nos.	4780
Total			

3.6.7 Seismicity of the Project Area

The concept of plate tectonics, sea-floor spreading and transform faults assume that the Himalayas are the result of collision between the Indian Plate and the Eurasian Plate. The plate movements are considered continuous even today with a rate of convergence of 5.6 cm/year. Since the plates are still moving, the Himalayas are more susceptible to earthquakes due to strain owing to tectonic activities in the region. In Nepal, three major tectonic features are associated with earthquake activities. Out of the three major faults of Nepal, the Main Central Thrust (MCT) is believed to be inactive, the Main Boundary Thrust (MBT) is believed to be active and the Himalayan frontal fault (HFF) is an active fault system; accordingly, the possibility of earthquake generation in this system can not be ruled out.

In Nepal, records of seismic activities are limited to the past 100 years. The list of recorded major earthquakes is presented in Table 3.6-2, and the magnitude and distribution of earthquakes as recorded in Nepal are shown in Figure 3.6-15. The latter indicates that the frequency earthquakes in the Eastern region is not high.

For the design of civil structures of the Project, as per the standard practice, a seismic coefficient value (Kh) of 0.08 is recommended, though the same value for the Arun 3 Hydroelectric Project has been taken as 0.12.

Table 3.6-2 Records of Major Earthquakes of Nepal

Date	Epicenter	Magnitude on Richter's Scale	Location and Damage
1916	80.00 N		Far western Nepal
	81.00 E	7.5	Extent of damage unclear
1/15/1934	26.5 N	8.4	Nepal - India border
	86.5 E		Deaths: 8,519 (Nepal)
	27.55 N	8.3	7,253 (India)
	87.09 E		Houses destroyed: 80,893 (Nepal)
5/27/1936	28.5 N	7.00	Western Nepal (Dhaulagiri)
	83.5 E		Damage: unclear
6/27/1966	29.6 N	6.0 (mb)	Far western Nepal/ Deaths: 42
	80.8 E		Houses destroyed: 3,969
7/29/1980	29.6 N	6.1 (Ms)	Far western Nepal/ Deaths: 178
	81.1 E		Houses destroyed: 13,258

3.7 Hydrology

3.7.1 Puwa Khola Discharge

(1) River Condition

The Puwa khola, site of the intake weir, has its head waters at elevation 3,000 m from which the river flows in a generally southerly direction to join with the Mai khola at elevation 400 m. The Puwa catchment is rectangular in shape, extending 6~7 km east to west and 22~26 km north to south. The main Puwa khola flows from north to south in the slightly eastern part of this catchment, with tributaries mainly flowing in from the west. At the middle and lower reaches under 1,600 m elevation, river gradient is 1/40~1/15. At the upper reaches river gradient is a steep 1/10. (see Figure 3.7-1 and Figure 3.7-2)

Forest cover is scant throughout the catchment, with hill slopes largely cultivated in the form of terraced paddy and upland field.

Total catchment area for the Puwa khola is around 162 km^2. Diversion site under the Project is approximately 10 km upstream from the Puwa khola confluence with the Mai khola. Catchment area at the diversion site is 125 km^2.

(2) Discharge Computation

As indicated in Figure 3.4-1, discharge level observations are performed at a single gauging station (GS 730). However, this station is located at a close 300 m downstream of the design intake site. This discharge data is applied directly to determining the generating discharge under the Project.

Discharge observations at GS 730 are indicated in Table 3.7-1. However, as can be seen from the tables, observations have not been regular since 1965. Supplemental discharge observations were performed 3 times under the Study, and the results of these are indicated in the following Table 3.7-2.

Table 3.7-1 Discharge Observation of The Puwa Khola at Sajbote Station No.730

Date		Width m	Area m²	Mean vel. m/s	Gauge ht. m	Disch. m³/s
1965	JAN. 18	14.0	5.2	0.35	0.60	1.8
	MAR. 10	9.8	3.3	0.30	0.50	1.0
	MAR. 22	9.8	3.1	0.23	0.48	0.7
	APR. 6	9.8	3.3	0.32	0.58	1.1
1966	FEB. 25	9.3	3.3	0.24	0.40	0.8
	MAY. 4	8.5	2.2	0.23	0.27	0.5
	JUL. 16	-	22.4	1.17	1.51	26.2
	SEP. 28	-	19.0	0.91	1.35	17.2
1967	APR. 5	7.0	2.5	0.44	0.51	1.1
	SEP. 20	-	21.0	1.15	1.44	24.2
1968	JUN. 10	10.6	2.7	0.48	0.52	1.3
	NOV. 6	11.0	4.6	1.07	0.90	4.9
1969	JAN. 4	9.0	2.4	0.68	0.57	1.6
	MAY. 1	6.5	2.1	0.75	0.60	1.6
	NOV.20	13.5	3.5	0.84	0.84	2.9
1970	FEB. 12	13.5	2.3	0.49	0.61	1.1
	APR. 6	6.3	2.0	0.46	0.54	0.9
1971	JAN. 12	14.5	2.8	0.67	0.80	1.8
	FEB. 3	14.5	3.1	0.45	0.58	1.4
	NOV. 30	14.5	4.5	0.59	0.88	2.6
1974	JAN. 1	13.5	3.7	0.66	0.79	2.5
1975	MAR. 26	8.5	2.4	0.32	0.55	0.8
1976	JAN. 25	14.0	3.9	0.44	0.91	1.7
1979	JAN. 11	8.0	2.6	0.42	0.71	1.1
1982	SEP. 9	15.6	7.2	1.26	1.20	9.1
	OCT. 28	14.0	4.6	0.85	1.08	3.9
1983	NOV.22	16.0	4.2	0.64	1.02	2.7
	DEC. 30	13.0	3.6	0.47	0.95	1.7
1984	FEB. 7	13.3	3.1	0.35	0.91	1.1
	FEB. 25	13.3	3.3	0.30	0.87	1.0
	FEB. 28	3.1	3.2	0.31	0.87	1.0
	FEB. 29	13.0	3.0	0.30	0.88	0.9
	MAR. 1	13.0	3.2	0.29	0.88	0.9
1985	SEP. 24	18.0	12.8	0.82	1.10	10.5
	DEC. 21	18.0	14.5	1.02	1.18	14.8
1986	JAN. 25	12.3	3.8	0.39	0.59	1.5
	JAN. 27	12.5	4.2	0.43	0.60	1.8
	MAY.27	17.2	4.7	0.45	0.66	2.1
	JUN. 6	18.0	6.1	0.57	0.75	3.5
	AUG. 3	33.5	18.1	1.06	1.30	19.1
	SEP.15	19.5	18.6	1.27	1.37	23.6
	NOV.28	18.0	4.6	0.70	0.72	3.2
1987	JAN. 13	17.7	5.2	0.27	0.57	1.4
	MAR.11	12.3	2.6	0.46	0.55	1.2
	APR.21	17.0	3.6	0.28	0.53	1.0
	JUL. 9	32.0	15.7	0.78	1.02	12.3
	NOV.26	18.0	5.7	0.63	0.37	3.6
	DEC.23	15.0	2.9	0.45	0.22	1.3
1988	MAR.21	11.0	3.0	0.40	0.21	1.2
	JUN. 10	16.0	4.7	0.41	0.31	1.9
	OCT. 3	20.0	5.6	0.55	1.27	3.1
	NOV. 5	20.0	5.6	0.55	1.27	3.1
	DEC. 9	18.0	4.1	0.41	1.20	1.7
1989	MAR.11	13.0	2.6	0.31	1.09	0.8
	NOV. 6	18.0	1.4	34.29	6.19	48.0
	NOV.27	16.5	4.0	0.62	1.19	2.5
1990	JAN. 6	12.0	1.7	0.47	1.06	0.8
	NOV.23	17.0	4.4	0.57	1.10	2.5
1991	JAN. 5	19.5	5.4	0.59	1.11	3.2
	MAY. 19	18.0	4.8	0.31	1.02	1.5
	JUN. 8	22.0	7.1	1.01	1.18	7.2
	SEP. 5	21.0	29.5	0.97	1.94	28.5
	SEP. 6	20.0	25.4	0.89	1.92	22.7
	SEP. 7	20.0	25.3	0.85	2.85	21.6
	SEP. 8	22.0	30.5	1.31	2.55	39.9
	SEP. 9	20.0	28.6	1.00	2.46	28.5
	DEC. 6	12.5	5.6	0.31	1.04	1.7
1992	FEB. 20	21.0	4.4	0.33	0.94	1.5
	MAY. 7	19.5	4.4	0.32	0.92	1.4

Table 3.7-2 Discharge Observations at the Intake Site

Date	Discharge (m^3/s)	Gauge highest
14 MAR. '93	1.15	0.91
8 MAY '93	1.56	1.09
10 MAY '93	1.61	-

note: water levels read from water level marker at GS 730

Applying the above data, a water level - discharge curve was computed and discharge calculated from the GS 730 water level data. Discharge calculations from water level data have been performed in the past applying monthly discharge figures for the 4 year period 1965~1968 from GS 730. During the Study, daily water level data for the period 1969~1986 were further obtained. From the foregoing data, the 10 year period for which almost no observation data are lacking was selected as a basis for computing discharge. Figure 3.7-3 ~ Figure 3.7-6 indicate water level - discharge curves, while daily discharge for each year are set out in Annex - II.

On the basis of the foregoing daily discharge data, a discharge curve was prepared, and year wise discharge and mean discharge values have been compiled in Table 3.7-3. The 10 year mean discharge curve is indicated in Figure 3.7-7.

Table 3.7-3 Discharge at Puwa khola Intake Site

Discharge / Year	Maximum	35 day	High water (95 days)	Average (185 days)	Low water (175 days)	Drought (355 days)	Minimum	Average
1972	187.6	50.6	25.7	3.7	2.0	1.5	1.3	18.1
1974	133.2	52.0	31.3	4.4	2.2	1.4	1.3	18.8
1975	100.1	45.2	29.0	3.2	0.8	0.8	0.8	16.5
1976	75.4	43.7	29.0	7.7	1.7	1.1	1.1	17.0
1978	52.4	35.7	27.2	5.4	1.8	1.2	1.2	14.6
1980	45.1	20.5	6.8	1.2	0.9	0.8	0.8	6.9
1983	30.0	17.5	8.5	1.6	0.6	0.5	0.5	5.5
1984	47.6	16.5	10.1	3.1	1.1	0.8	0.8	6.4
1985	33.1	14.0	10.2	3.3	2.3	1.7	1.6	6.6
1986	75.2	14.0	9.2	2.6	1.5	0.9	0.9	6.6
average	78.0	31.0	18.7	3.6	1.5	1.1	1.0	11.7

As discussed earlier, GS 730 and the intake site under the Project are located close to each other, with respective catchment areas almost the same at 125.8 km^2 and 125.1 km^2. Accordingly, for purposes of discharge computation under the Project, discharges at both locations are assumed to be the same.

Annual mean monthly discharge values and 10 year mean monthly discharge values are indicated in Annex - I.

(3) Flood Discharge

Flood discharge at the Puwa khola diversion site is computed as follows:

$$Q = \frac{1}{3.6} \cdot f \cdot R_t \cdot A$$

where:

Q	=	flood peak discharge (m^3/s)
f	=	run off coefficient (0.8)
R_t	=	precipitation during flood arrival time (mm/h)
A	=	catchment area (125.1 km^2)

In order to determine rainfall intensity during the flood arrival time (R_t), flood arrival time (T) is computed by the following formula, yielding a value of T = 2.3 h (= 138 min).

$$T = \frac{L}{W}$$

$$W = 72\left(\frac{H}{L}\right)^{0.6}$$

where:

T	=	flood arrival time (h)
L	=	length of river channel (km)
H	=	head (km)
W	=	flood arrival speed (km/h)

Furthermore, in order to determine the probable rainfall(mm/day) (R_{24}) with return period of 100 years necessary to compute rainfall intensity (R_t), data with the highest values in the vicinity of the Project area (at ST 1411) was adopted. In other words, using the annual maximum rainfall for an 18 year period and applying Iwai's formula, probable rainfalls were computed as shown in Table 3.7-4.

Table 3.7-4 Probable Rainfalls at ST 1411

Return period (years)	Annual maximum rainfall (mm/day)
10	252
20	278
50	310
100	333
200	356

From the above table, the probable rainfall with return period of 100 years (333 mm/day) was adopted as the design daily rainfall for the Puwa khola under the Project.

On the basis of the above, the rainfall intensity (R_t) is computed according to the following formula whereby $R_t = 48.2$ mm/h:

$$\frac{R_t}{R_{24}} = \frac{34710}{T^{1.35} + 1502}$$

where

T	=	flood arrival time(= 138 *min*)
R_t	=	rainfall intensity within flood arrival time (mm/hr)
R_{24}	=	design daily rainfall (= 333 mm/day)

From the above, probable discharge on the Puwa khola at the diversion site with return period of 100 years is 1,450 m^3/s, and this is set as the design flood discharge for the Project.

Detailed calculations for flood discharge are given in Annex II.

(4) Water Level - Discharge Curve for the Intake Site

Water level wise discharge was computed according to the following formula for the intake site on the Puwa khola, and a water level - discharge curve prepared.

$$Q = A \cdot v$$

where

Q	=	discharge (m^3/s)
A	=	flow cross section (m^2)
v	=	flow velocity (m/s)

$$v = \frac{1}{n} \cdot R^{\frac{2}{3}} \cdot I^{\frac{1}{2}}$$

where

n	=	roughness coefficient (= 0.05)
R	=	depth radius
I	=	water surface gradient (1/40)

The following Table 3.7-5 compiles the results of calculation. Water level - discharge curve is shown in Figure 3.7-8.

Table 3.7-5 Water Level and Discharge at Puwa Khola

Elevation	Water Level (m)	Flow cross section (m^2)	Flow speed (m/s)	Discharge (m^3/s)
756	1.0	22.6	2.89	65.3
757	2.0	50.2	4.40	220.9
758	3.0	81.4	5.63	458.3
759	4.0	115.1	6.73	774.6
760	5.0	151.2	7.61	1,150.6
761	6.0	189.9	8.38	1,591.4
762	7.0	231.4	9.09	2,103.4
763	8.0	276.7	9.59	2,653.6

According to Figure 3.7-8, water level for design flood discharge of 1,450 m^3/s is 760.8 m.

3.7.2 Mai Khola Discharge

(1) River Condition

The Mai Khola, where the tailrace outlet site is located, has its headwaters at EL 3,500 m, from which it flows in a generally southerly direction to be fed by the Jogmai khola at a point about 3.5 km southeast of Ilam town. From here, the Mai khola alters direction to the southwest, joining with the Puwa khola at a point 6 km south of Ilam town. The Mai khola continues its course, absorbing a number of tributaries along the way, to flow out of the hill area into the southern lowland, where it empties into the Kankai river. The catchment area of the Mai khola is around 417 km^2 at the point of confluence with the Puwa khola, of which the Jogmai khola catchment accounts for 157 km^2.

The river gradient of the Mai khola is a steep 1/10 or more until elevation 1,200 m. From that point downstream, gradient gradually becomes gentler to 1/20~1/30 at the point of confluence with the Jogmai khola at 600 m elevation. At the point of confluence with the Puwa khola at 400 m elevation, river gradient is 1/60~1/100. The power station and tailrace outlet site under the Project is located at elevation 470 m roughly midway between the Mai khola's points of confluence with the Jogmai khola upstream and the Puwa khola downstream.

(2) Flood discharge

Flood discharge at the Mai khola diversion site is computed as follows:

$$Q = \frac{1}{3.6} \cdot f \cdot R_t \cdot A$$

where:

Q = flood peak discharge (m^3/s)

f	=	run off coefficient (0.8)
R_t	=	precipitation during flood arrival time (mm/h)
A	=	catchment area (386.2 km^2)

In order to determine rainfall intensity (R_t) during the flood arrival time, flood arrival time (T) is computed by the following formula, yielding a value of T = 3.09 h (= 185.4 min).

$$T = \frac{L}{W}$$

$$W = 72\left(\frac{H}{L}\right)^{0.6}$$

where:

T	=	flood arrival time (h)
L	=	length of river channel (km)
H	=	head (km)
W	=	flood arrival speed (km/h)

Furthermore, probable rainfall (mm/day) (R_{24}) with return period of 100 years necessary to compute rainfall intensity (R_t) was determined at 333 mm/day as in the case of the Puwa khola as described in the previous section.

On the basis of the above, the rainfall intensity (R_t) is computed according to the following formula whereby R_t = 43.6 mm/h:

$$\frac{R_t}{R_{24}} = \frac{34710}{T^{1.35} + 1502}$$

where

T	=	flood arrival time(= 138 *min*)
R_t	=	rainfall intensity within flood arrival time (mm/hr)
R_{24}	=	design daily rainfall (= 333 mm/day)

From the above, probable discharge on the Mai khola at the power house site with return period of 100 years is 3,750 m^3/s, and this is set as the design flood discharge for the Project.

Detailed calculations for flood discharge are given in Annex II.

(3) Water Level - Discharge Curve for the Power House Site

Water level wise discharge was computed according to the following formula for the power house site on the Mai khola, and a water level - discharge curve prepared.

$$Q = A \cdot v$$

where

Q	=	discharge (m^3/s)
A	=	flow cross section (m^2)
v	=	flow velocity (m/s)

$$v = \frac{1}{n} \cdot R^{\frac{2}{3}} \cdot I^{\frac{1}{2}}$$

where

n	=	roughness coefficient (0.04)
R	=	depth radius
I	=	water surface gradient (1 / 70)

The following Table 3.7-6 compiles the results of calculation. Water level - discharge curve is shown in Figure 3.7-9.

Table 3.7-6 Water Level and Discharge at Mai Khola

Elevation	Water Level (m)	Flow cross section (m^2)	Flow speed (m/s)	Discharge (m^3/s)
433	2.0	64.1	3.43	220
434	3.0	156.7	3.52	551
435	4.0	290.6	4.75	1,380
436	5.0	437.2	6.13	2,680
437	6.0	586.4	7.36	4,310
438	7.0	737.9	8.47	6,250

According to Figure 3.7-9, water level for design flood discharge of 3,750 m^3/s is 436.7 m.

3.8 Water Quality

Simple water quality analysis was performed at the diversion site on the Puwa khola and the power station site on the Mai khola. Results are tabulated below:

Table 3.8-1 Water Quality Analysis on the Puwa and Mai Kholas

Item	Puwa	Mai
Sampling date	10-May-93	11-May-93
Water Temperature (°C)	18	20
pH	7	7.6
Electroconductivity (μ S/cm)	30	80
DO (mg/l)	6	10
COD (mg/l)	2	2
Common microorganisms	detected	detected
Colon bacillus	detected	detected

Water quality at either river is not considered to pose any problem under the Project, although values for the two show some variance.

CHAPTER 4

CHAPTER 4 PROJECT PLAN FORMULATION

4.1 Basic Approach

4.1.1 Optimum Site Selection in Consideration of Topography and Geology

There are certain geological and topographical conditions in Nepal which restrict hydropower development ; namely,

1. The prevalence of areas of slope failure including landslide zones; and,

2. A large flood volume accompanied by substantial sediment load and movement of boulders due to the lack of forested mountain slopes.

Special care must therefore be taken to select project sites with sufficient base rock, particularly in the case of an intake dam. The headrace tunnel route should be stable and safe from landslides, while the head tank should be located on solid rock, and the penstock route should offer sufficient stability by following ridges and, preferably, bare rock.

The power house must be located safely above flood levels and where there is little danger of falling rocks, etc., while there must be minimal chance of landslides or slope failure along the tailrace route.

4.1.2 Study on Type of Generating Scheme, and Installation of a Regulating Pond

The only power station which has a dam in Nepal at present is Kulekhani No.1. Operation of the dam permits year long discharge regulation and peak operation at both Kulekhani No.1 and the Kulekhani No.2 station downstream. In general, however, rivers throughout Nepal have high sediment loads and sedimentation, and consequently dams have not been included in other hydropower schemes. As a result, such schemes cannot respond to peak load during the low flow winter season and it has been necessary to apply emergency measures such load shedding.

In the case of the Puwa Khola, a tributary of the Kankai river, sediment load is also high and accordingly a dam such as that at the Kulekhani power station is not feasible. However, as the majority of sediment can be eliminated by a settling basin, a regulating pond is planned adjacent to the head tank allowing some degree of diurnal regulated operation, even though the envisioned scheme is not a dam type one.

Judging from the river discharge at the intake site, maximum generating discharge can be obtained from mid-May to early December and flow during this period is sufficient for discharge regulation. From mid-December to early May, the facility can switch to short hour daily regulated operation when necessary.

4.1.3 Turbine and Generator

Installed capacity of the power house is a comparatively small 6.2 MW and although the installation of a single generating unit is most cost-effective, two units are to be installed in consideration of transportation factors and stable operation. Moreover, the 7th Power Project will already be completed by the time the present Project is finished, and as generated power can be fully utilized within the grid, installation of two units at the this time is considered advantageous.

4.1.4 Scope of Power Transmission Plan

Under the 7th Power Project installation of a 33 kV phase transmission line is planned between the Anarmani power station and the Ilam power station, distance of about 55 km with conductor of ACSR (aluminum conductor - steel reinforced) dog size (section area 100 mm^2) The plan is judged sufficient for transmission of the estimated 6.2 MW of power planned under the present project. Accordingly, it will be sufficient to connect a single transmission line (4.7 km) to the 33 kV primary side of the 3 MVA substation to be constructed at Ilam Bazar under the 7th Power Project In this case if, for example power demand in the Ilam area is 3 MW the remaining 3.2 MW out of the 6.2 MW generated under the project could be fed into the national grid.

4.2 Formulation and Preliminary Comparative Study of Alternative Development Schemes

A 300 m difference in river stage occurs within a space of about 3 km between the Mai khola, which flows southwest through the southern part of Ilam N.P., and the Puwa Khola, which runs from north to south along the west side of Ilam Bazar, with the Ilam ridge sandwiched between the two rivers. This marked difference in water level gave rise to the idea of utilizing the river stage for hydropower generation and the implementation of a development plan study.

In the study, the five alternative schemes described hereunder were formulated on the basis of existing topographical and geological maps, river flow data, and other information. Intake sites for each alternative were to be located between the confluence of the Puwa Khola and its tributary the Ghatte Khola, and the vicinity of the Damai Bridge downstream, while discharge sites were to be located between 800 m upstream and 1500 m downstream from the Mai Khola Bridge.

The five alternatives are shown in Figure 4-1 while numerical values for each are presented in Table 4.2-1.

Table 4.2-1 Features of Each Alternative

Feature \ Alternative	A	NEA	B+C	B	C	Optimum Plan,D
Catchment area (km²)	125	125	125	125	125	125
Maximum generating discharge (m³/s)	2.5	2.5	2.5	2.5	2.5	2.5
Available head (m)	284	302	308	139	169	304
Maximum output (kw)	5,800	6,100	6,200	2,800	3,400	6,200
Canal length	3,650	4,610	4,950	2,850	2,100	4,317
Site index (I)	9.7	8.1	7.8	6.1	10.0	8.8

Hydropower project site indexes (I) are the criterion of profitability for each alternative and are expressed in the following formula.

$$I = \frac{\text{catchment area} \times \text{available head}}{\text{canal length}}$$

The numerator in this formula represents available energy, while the denominator is the indicator for construction cost.

As can be seen from Table 4-1, alternatives C, A, and NEA have high site index values. When alternative C is combined with alternative the B as B+C alternative however, the index value is lowered. Accordingly, alternatives A and NEA were selected for further study as optimum plan proposals. However, the actual optimum scheme was subsequently selected by methods described hereunder after more detailed investigation of topography and geology in the Project area.

4.3 Selection of Optimum Route on the Basis of Field Survey

The field survey on the alternative plan proposals focused on data directly related to construction including topography, geology, ground, and river discharge, as well as on indirectly related factors such as land use and access road conditions. The results of the survey, including problem areas in each of the alternatives, and the newly formulated optimum development plan (alternative D), are described hereunder.

Despite the steepness of the terrain, the only areas around the Project site which are not farmed are sheer cliffs, exposed rock outcrops or landslide areas. No wooded areas are seen. As for road conditions, the majority of roads, excluding roads in Ilam town, are extremely narrow and unsuitable for regular vehicular traffic.

Although access road conditions to the planned intake site for alternative C, which had the highest index value and therefore was considered most favorable, were poor, the topography and geology of the site were suitable for construction of a small scale intake facility. It was discovered however, that the planned water tank and penstock sites were situated in a landslide area where construction should be avoided. Moreover, the proposed power house site is quite far from existing national highways which are passable by regular vehicles, necessitating the conversion of farmland into an access road. Effective head in this alternative is comparatively small and accordingly the plan was considered unsuitable in terms of the effective use of energy.

Access roads for the alternative B intake site are not ideal, but this problem can be managed without directly affecting farmland along the planned canal route. Although there is little farmland along the design canal route (on the mountainside), landslide areas occurring along the route pose a major obstacle to construction. Alternative B is also comparatively expensive as the majority of facilities, including the canal, water tank, penstock, and power house are all located on the right bank requiring construction of a bridge. Moreover, as in the case of alternative C, effective head is small making this alternative disadvantageous in terms of effective energy use. For the above reasons alternative C was eliminated as unsuitable. Alternative B + C, a combination of B and C with operation of two power stations, was likewise ruled out.

The intake sites for alternatives A and NEA were the same as for alternative B with poor access road conditions but favorable topography and geology for construction of small scale intake facilities. The power station site for alternative NEA is the same as that for alternative B but with a longer canal route to utilize the maximum effective head, while the site for alternative A is further upstream with a slightly shorter canal route. It was discovered that landslide areas occur along the planned water tank and penstock sites for both alternatives and that fairly long access roads including that to the power house site, are required.

Based on the survey results described above, none of the alternatives were appropriate for the optimum development plan. Instead the following alternative (D) which presents comparatively fewer problems was selected. The intake site is the same as that for alternatives A, NEA and B. Although access road conditions present some difficulties, the topography of the area makes this unavoidable if the intake site is to be located on the Puwa khola. In order to increase power generation efficiency, the intake site was located as far upstream as possible. The criteria for selection of water tank, penstock and power house sites, were stable ground, avoidance of areas with anticipated faults or landslide areas, and proximity to existing national highways which are passable by regular vehicular traffic. Increase in the length of the penstock or canal to meet these criteria were considered acceptable.

The optimum development plan was selected according to the above approach. The water tank and penstock sites were located along the ridge in consideration of topography, and close to existing highways. The power house site was also located to facilitate the use of these highways. Comparison of advantages and disadvantages for each alternative considered is given in Table 4.3-1.

Table 4.3-1 Comparison of Alternatives

	Alternatives					Optimum development (D)
	A	NEA	B+C	B	C	
Construction	Δ	Δ	×	×	×	○
Site and road conditions	Δ	Δ	×	×	Δ	Δ
Effective energy use	○	○	○	×	×	○
Total	Δ	Δ	Δ	×	×	○

○ = favorable
Δ = fair
× = unsatisfactory

Features of the optimum alternative are given below.

Table 4.3-2 Features of Optimum Alternative

Catchment area (km^2)	125
Maximum generating discharge (m^3/s)	2.5
Available head (m)	304
Maximum output (kw)	6,200
Canal length (m)	4,317
Site index (I)	8.8

4.4 Study of Sites for Optimum Development Scheme

4.4.1 Intake Site

In consideration of topography and geology, the intake site was to be selected in an area from the confluence of the Puwa Khola and Ghatte Khola to the vicinity of the Damai Bridge about 350m downstream.

Two alternative sites which were thought to have solid base rock were selected for comparative study: an upstream alternative located 40 m downstream of the confluence with the Ghatte Khola, and a downstream alternative located 40m upstream from the Damai bridge.

The geological foundation both for alternative sites is crystalline schist which is suitable for a dam with a height of about 5~6m. Both alternatives also require construction of an access road through rugged terrain to allow transportation of materials to the site. The topography of the upstream alternative is steeper than the down stream alternative and river width is also narrower which would permit a slightly smaller and therefore more economic dam volume. At the same time, however, the site would increase construction works such as installation of coffer dams on alternate sides of the river during weir construction.

The river water level of the upstream proposal is about 3 m higher than the downstream alternative, making it slightly more advantageous in terms of energy use, but the headrace tunnel length is that much longer resulting in increased construction costs.

Although there is very little difference between the two alternatives, a landslide area located on the left ridge between the two sites does not affect the upstream alternative whereas there is a danger that it might adversely affect the intake function of the downstream alternative if sediment from the slide should directly flow into the intake structure and accumulate there.

As a result of the comparative study, the upstream alternative located about 40 m downstream from the confluence with the Ghatte Khola was selected as the optimum intake site both because it is not directly affected by the landslide area and because of its more advantageous energy use.

4.2.2 Headrace Tunnel

The headrace canal is planned as non-pressure tunnel joining the intake site and the head tank which is located on the top of the ridge on the right bank of the Mai khola. Geology, overburden, the settling basin, the location and arrangement of construction adits, and minimization of total length were all considered in selection of the canal route. The planned route passed through a mountain of crystalline schist, and excluding portions where weathering is closer to the surface near one part of the settling basin as well as the adit and water tank sites, the route is considered to be geologically stable bedrock with overburden.

Construction of adits at the intake and head tank sites was also considered. However, construction of an adit in mid-route was selected to avoid duplication of tunnel construction facilities and congestion of other construction works near the tunnel entrance, as well as in consideration of such factors as a safe margin for the construction period and emergency measures which might be implemented if poor geological conditions are encountered. The selected adit site is located upstream from the Puwa khola Upreti bridge with 60% of the tunnel upstream and 40% downstream, in consideration of topography, geology, division of tunnel works, blasting schedule and transportation of materials.

The head tank site is surrounded by farm land and there are no small streams which allow safe discharge of excess flow. Accordingly, the construction adit for the headrace tunnel will be used as the spillway to divert excess water into the Puwa Khola.

4.4.3 Head Tank

The head tank site was selected according to several criteria. These include an elevation of about 760 m, location on the ridge which is assumed to be stable ground with no landslides and little weathering, and accessibility for transportation of materials and equipment.

The selected site has a comparatively gentle gradient, and is located on firm bedrock along the ridge which is judged to be sufficiently safe for a head tank site. The west side of the site is level farm land and potential use of this as a regulating pond was also a determining factor in site selection. Construction materials and equipment can be supplied relatively easily to the site as it is located near the existing main road.

4.4.4 Penstock

Although some consideration was given to its relation with the head tank and power house sites, the penstock route was selected on the basis of alignment along the ridge which is judged to be stable bedrock with no danger of landslides and proximity to the existing road which will facilitate access of construction equipment and materials and the removal of surplus soil from cutting works. The proposed route intersects the main road in one place and care must be taken that the penstock does not impede traffic. However, this very proximity will make construction work comparatively easy.

4.4.5 Power House and Substation

The optimum power house and substation sites were selected on the basis of minimum occurrence of landslides or slope failure, and safety during floods. Accessibility was also important as heavy machines such as turbines, generators, and transformers must be carried to the site. Materials can be transported to the proposed site by an existing road as well as by fording the river during the dry season.

The width of the Mai khola riverbed near the proposed site is 100 ~ 150 m while the width of the water surface in the dry season is only 20 m. Cobble deposits which have accumulated on the riverbed are presently being removed to make crushed rock and thus the site has advantage of readily available aggregate material. The right bank of the river is level and judging by the comparatively large trees growing there it has not been subject to flood damage, one of the reasons for selection of this site.

The site is easily reached by crossing the Mai khola in the dry season; however, in consideration of transportation during the rainy season and maintenance after completion, construction of an access road on the right bank is considered desirable.

4.5 Selection of Optimum Scale

The optimum scale was determined by comparing benefit (B) and cost (C). In this method four different levels of maximum discharge were compared and as a result of the study summarized hereunder, a maximum discharge of 2.5m³/s was selected as the optimum scale.

(1) Output and Generating Capacity

Power output P (kW) is determined by the following formula.

$$P = 9.8Q \cdot H_0 \cdot \eta_1 \cdot \eta_2$$

where

Q	=	discharge (m³/s)		
H_0	=	effective head (m)	=	304 m at maximum discharge
			=	314 m at firm discharge
η_1	=	turbine efficiency	=	0.87 m at maximum discharge
			=	0.87 m at firm discharge
η_2	=	generator efficiency	=	0.95 m at maximum discharge
			=	0.93 m at firm discharge
$\eta_1\eta_2$	=	combined efficiency	=	0.83 at maximum discharge
			=	0.81 at firm discharge

Annual possible energy E_a(kwh) is calculated by the following equation.

$$Ea = 9.8 \times (\Sigma Q \times 24) \times H_e \cdot \eta_1 \cdot \eta_2$$

where :

ΣQ	=	annual available discharge (m³/sec.day)
H_e	=	effective head (=304m)
$\eta_1 \cdot \eta_2$	=	combined efficiency (=0.83)

Ordinary energy output E_f (kWh) is calculated by the following formula:

$$E_f = 365 \times 24 \times \text{firm output}$$

Secondary energy output E_r (kWh) is determined by the following equation:

$$E_r = E_a - E_f$$

(2) Benefit (B)

kW and kWh values applied in Nepal were used for benefit analysis.

$B = B_1 + B_2 + B_3$

where:

B	=	total benefit under the Project
B_1	=	kW value = US$ 83.05/kW/year
B_2	=	value of firm energy produced = US$ 0.075/kWh
B_3	=	value of secondary energy produced = US$ 0.0207/kWh

(3) Annual Cost (C)

Annual cost (C) comprises depreciation, interest, operation and maintenance cost, etc.:

$$C = construction\ cost \times (1 + interest\ during\ construction)(capital\ recovery\ factor + O/M, etc. cost\ ratio)$$

$$captital\ recovery\ factor = \frac{i(1+i)^n}{(1+i)^n - 1} = 0.10086$$

where:

i	=	*discount rate (= 10%)*
n	=	*life of facilities (= 50 years)*

Operation and maintenance costs, etc. are assumed at 1.5% of total construction cost.

Rate of annual expense = capital recovery rate + O/M cost, etc. = 0.11586

Interest during construction is computed according to the following:

$$0.4 \times t \times R$$

where:

t	=	construction period (3 years)
R	=	interest rate at 10%

(4) Comparison for Various Maximum Discharges (Maximum Output)

Table 4.5-1 indicates computation of benefit. Costs and B/C ratio are compared in Table 4-5. Figure 4-2 gives a comparison of B/C ratio.

On the basis of the above comparison, maximum generating discharge of 2.5 m³/s is determined as the optimum for the Project

Table 4.5-1 Benefit Computation

Maximum generating discharge (m3/s)	1.5	2.0	2.5	3.5	Remarks
Maximum output (kW)	3,700	5,000	6,200	8,700	
Firm output (kW)	2,700	2,700	2,700	2,700	
Potential generated energy (MWh)	29,912	37,218	43,771	55,059	
Firm generated energy (MWh)	29,912	29,912	29,912	29,912	
Secondary generated energy (MWh)	0	7,306	13,859	25,147	
Peak generation (MWh)	3,700	3,700	3,700	3,700	PEAK : 4 hr
kW value ($)	307,285	307,285	307,285	307,285	unit cost: $83.05/kW
Firm generated power value ($)	2,243,400	2,243,400	2,243,400	2,243,400	unit cost: $0.075/kWh
Secondary generated power value ($)	0	151,234	286,881	520,543	unit cost: $0.008/kWh
Benefit (B) (US$ '000)	2,551	2,702	2,838	3,071	0.0207

Table 4.5-2 Cost and B/C Comparison

unit: US$ '000

max. discharge (m³/s) max. output (W)	1.5 3,700	2.0 5,000	2.5 6,200	3.5 8,700	Remarks
1. Road, bridge	390	390	390	390	
2. Compensation	156	156	156	156	
3. Civil construction	6,234	6,447	6,635	6,980	
Intake facilities	333	333	333	333	
Settling basin	405	431	454	493	
Headrace tunnel	3,624	3,624	3,624	3,624	
Head tank	529	560	587	634	
Penstock	1,052	1,163	1,261	1,452	
Power house	245	293	328	388	
Tailrace	38	43	48	57	
4. Power generating equipment	3,200	3,460	3,540	4,200	
5. Transmission line	94	94	94	94	
6. Subtotal	10,074	10,547	10,815	11,820	
7. Temporary facilities	1,007	1,056	1,082	1,182	10% of ⑥
8. Contingency	1,319	1,378	1,413	1,531	10% of ①②④⑤ 15% of ③
9. Total	12,400	12,991	13,310	14,533	
10. Engineering fee	1,331	1,331	1,331	1,331	
11. Grand total	13,731	14,322	14,641	15,864	project const.cost
Annual expense	1,782	1,858	1,900	2,059	const. capital x 0.11586
Benefit (B)	2,551	2,702	2,838	3,071	
B/C	1,432	1,454	1,494	1,492	

CHAPTER 5

CHAPTER 5 FACILITIES PLAN

5.1 Basic Criteria in Plan Formulation

According to the actual site conditions and the results of study on the optimum Project plan, the basic approach for study and design for the civil engineering facilities are as follows:

(1) On the basis of study findings, generation method is a run-of-river type.

(2) Intake method is ogee type weir and tyrollean type water intake.

(3) Head tank has a regulating pond

(4) Maximum design generating discharge is 2.5 m^3/sec.

(5) Puwa khola design flood discharge is 1,450 m^3/sec at intake site. Mai khola design flood discharge is 3,750 m^3/sec at power house site.

5.1.1 Power Generating Method

The Puwa khola at the proposed site is a swift flowing torrent which passes through a deep ravine lined by steep cliffs. Geological conditions present no obstacle to construction of a concrete dam with a regulating capacity of about 20~30 m. Required regulating capacity is small, no more than 20,000 m^3. It would be natural under these circumstances to consider a pondage type dam if dam conditions, maintenance and economic viability permit.

In this case, however, due to the large body of the dam body, such a plan would require considerable large scale equipment, as well as construction of temporary facilities and roads, in addition to the various facilities necessary for dam maintenance, resulting in a significant increase in construction costs. In addition, farmland extends far upstream along both sides of the river reaching up to the mountain tops, while landslide zones and areas of slope failure occur in several of the steeper areas. Judging from the deposits of boulders and large rocks, large scale movement of earth and rocks accompanies sediment runoff during floods.

Surveys and studies on the volume of earth and rocks accompanying sediment runoff, including boulders, due to soil erosion have been conducted in Japan. Although these can not be considered as perfectly applicable to the proposed site, if 2,000 m^3/km^2 per year is set as the standard annual flow volume of earth and rocks, the volume at the site would be 250,000 m^3/year. Moreover, according to data and reports on sediment volume in the drainage area of the Karnali and Sapta kosi rivers, flow volume is 1.7 $mm/m^2\cdot$year for the former and 1.9 mm/m^2/year for the latter. If 1.8 $mm/m^2\cdot$year is applied to the proposed site, the annual sediment volume is 225,000 m^3, roughly equivalent to the largest sediment values obtained in Japan. If this is considered in combination with the flow of earth and rocks described above, it is clear that a medium scale regulating pond or storage reservoir would be filled in within a few years losing its function. When required regulating capacity is minimal, at most 20,000 m^3 (4 hour peak), as is the case at the proposed site, it is possible to remove new sediment deposits as they occur to ensure required capacity. This is one method of power station maintenance, but with this method the system of removal presents its own difficulties, such as spoil bank location, access roads, and equipment.

In consideration of economic viability, the reduction of pondage capacity due to sedimentation, and the difficulties of removing earth and rock deposits, the construction of a pondage dam at the proposed intake site was judged impractical at this stage. However, there is a strong demand for a local power source which has a regulating capacity in the dry season. Moreover, the extremely small maximum discharge (2.5 m^3/sec) needed at the site is an advantage. This discharge could be secured by utilizing part of the headrace tunnel which has a capacity of about 3,500 m^3 as determined by the working section of the headrace and its flow volume. This combined with a separate regulating pond with a capacity of 2,000 m^3, taking advantage of the topography near the water tank, would secure a total pondage capacity of 5,500 m^3, with the Project planned to be capable of peak generation in the dry season. In other words, the power generation method at the proposed site will be a non-pressure run-of-river type with the possibility of pondage type power generation.

With regards to the discharge inside the headrace tunnel, as shown in Drawing No. ILAM-F/S001, inner section of the tunnel section is W 2 m × H 2 (upper section semi circular) with a 0.1 m space at the tunnel entrance. Low water discharge is 1.1 m^3/s with gradient of 1/1,660 and roughness coefficient $n = 0.013$ and uniform flow depth.

Regulating pond is planned with surface area of 1,063 m^2 and effective depth of 1.9 m. Water volume is to be 2,000 m^3.

Required regulating capacities as calculated for peak operating hours of 1, 2, 3, and 4 for respective discharges of 1.5, 2.0, 2.5 and 3.5 m^3/s are indicated in Table 5.1-1.

Table 5.1-1 Required Regulating Capacity Considering Maximum Discharge and Peak Generation

unit: (m^3)

Discharge (m^3/s)	Peak period (hr)			
	1	2	3	4
1.5	1,440	2,880	4,320	5,760
2.0	3,240	6,480	9,720	12,960
2.5	4,680	9,360	14,040	18,720
3.5	8,640	17,280	25,920	35,560

As can be seen from the above table, when the required regulating capacity and the potentially available storage capacity of 5,500 m^3 are compared, not even 1 hour of continuous operation is possible in the case of the maximum generating discharge of 3.5 m, making a plan including peak operation of the scheme problematic. With a maximum generating discharge of 1.5 m^3/s, 3~4 hours of peak operation becomes possible, however, power house scale becomes small. In the case of discharges of 2.0 m^3/s and 2.5 m^3/sec, peak operation for 1.5 hours and 1.1 hours, respectively, becomes possible.

5.1.2 Intake Method

(1) Rubber Dam

Although the maximum intake volume at the site is small at about 2.5 m^3/s, the maximum output is 6,200 kW, representing a significant supply capability and demonstrating the important role the power station will fulfill as a local energy source. The power generation method is run-of-river type based on such factors as topography, river gradient, and accessibility which make construction of an intake dam with regulating capacity difficult as discussed in section 5.1.1. Nevertheless, the installation of a rubber dam for securing regulating capacity was studied.

The rubber dam would collapse onto the river bottom during flooding, allowing rolling boulders and large rocks to pass over it and preventing deposits at the intake site. When flooding subsides the rubber dam would be raised to a vertical position storing water up to the level of the dam's height which could be used for regulating.

The following results were obtained from the study.

1. Due to the size and weight of the rubber dam materials, a trailer would be required to take it to the site. The road width, gradient, and turn radius, however, make this impossible.

2. In addition to the bag which forms the body of the dam, additional facilities and an electric power source are needed to collapse and raise the dam. This would increase the overall costs.

3. There are very few cases in which a rubber dam has been used in a steep flowing torrent with heavy sediment runoff which includes rolling stones and boulders. In particular there are almost no cases in which it has been used in a swift flowing river with a gradient of about 1/40 such as that at the present site.

4. In Japan there are no examples of rubber dams being used as a regulating dam for power generation. As a rule, rubber dams are used in power generation when the intake level is constant.

With a small discharge of 2.5 m^3/s as at the present site the required regulating capacity is also small. Therefore, although the idea of using a rubber dam was a good one, for the present, its application seems impractical in terms of installation, construction costs, and lack of precedent. This alternative was accordingly abandoned.

(2) Tyrolean Type Intake Dam

The intake site is in rugged terrain with heavy rock and mud flow during the flood season.

For intake, the conventional approach is to raise water level by means of an intake weir, with diversion of discharge from the side of the structure. However, the following problems affect such an approach: (i) the subject diversion site is in area of poor road access under which conditions careful and regular inspection and maintenance become difficult, and (ii) gate operation during periods of diversion is anticipated to be difficult, and (iii) damage to the sand flush gate is highly likely.

The bar screen intake method on the other hand, is presently in use at the Tatopani hydropower station (1,000 kW finished in 1991, 1,000 kW under construction at present) which has similar topographical, access road, river conditions, and intake volume as the present site. This method, known as the Tyrolean method, has a record of past performance in mountain torrents, and is used where maintenance is difficult, where large sediment and rock flows occur during flood season, and where there is a danger of damage to gates or other facilities.

For these reasons, the Tyrolean method used successfully at the Tatopani hydropower station was adopted as the intake method for the present site.

5.2 Essential Planning Criteria

5.2.1 Effective Head Calculation

(1) Calculation Criteria

Effective head is calculated for maximum generating discharge of 2.5 m and firm discharge of 1.1 m^3/s. Head tank water levels for these respective discharges are as follows.

where:

tunnel section	=	W 2 m, H 2 m (upper section semi circular)
tunnel gradient	=	1/1660
roughness coefficient	=	0.013
tunnel outlet sill height	=	EL 755.0 m

Discharge (m^3/s)	Uniform flow depth (m)	Head tank water level (EL m)
2.5	1.038	756.90
1.1	0.58	755.58

If the turbine center elevation is set at EL 438.7 m taking into consideration safety vis a vis a design flood water level at the power house site of HWL 436.7 m.

(2) Types of Head Loss

Total head loss is the sum of the following head losses determined according to the relevant formula:

a. Water surface drop at head tank (due to screen)

b. Head loss along penstock route

Inflow head loss
Friction loss
Head loss due to bending of penstock alignment
Head loss due to narrowing of penstock pipe diameter
Head loss due to bifurcation
Head loss due to inlet valve
Other losses

(3) Effective Head

On the basis of computation applying the above, head losses are 14.20 m and 2.88 m and effective heads are 304.0 m and 314.0 m for maximum discharge (2.5 m^3/s) and firm discharge (1.1 m^3/s), respectively.

Calculation criteria for head loss are given in Annex III.

5.2.2 Water Hammer and Penstock Calculations

(1) Water Hammer Calculation Criteria

Starting point of the penstock at the head tank is EL 751.30 m. End point of the penstock is EL 437.10 m. (Elevation of the turbine center is 438.70 m.) As the head tank water level is WL 756.90 m for maximum discharge, starting point and end point of the penstock are subject to 5.60 m and 319.80 m of static head, respectively.

Penstock inner diameter changes from 1.10 m to 0.60 m after bifurcation. Total penstock length is 990 m.

Needle closing time is 30 sec.

(2) Water Hammer

Time required for one return trip by the pressure wave is approximately 2 sec. Since a closing time of 30 seconds allows for gradual closure, water hammer value is computed according to the formula for gradual closing time.

On this basis, water hammer at the penstock terminus is computed at 5% of the maximum static head of 319.80 m, or 16 m. Design pressure is accordingly 339.80 m including 6.2% of static head equivalent to 20 m.

Water hammer at the various points along the penstock is assumed proportionately on the basis of 0 at the penstock starting point and highest value at the penstock terminus.

(3) Penstock Strength Calculation

Design pressure for the penstock is static head plus water hammer. Necessary pipe thickness for design pressure is computed according to the following formula.

Design pressure:

start point of penstock:	0.56 kg/cm²
at the penstock terminus:	33.98 kg/cm²

$$P = 2(t-\varepsilon)\cdot\sigma_a \frac{\eta}{D}$$

where:

P	=	*allowable pressure* (kg/cm^2)
t	=	*steel plate thickness* (cm)
q	=	*safety margin thickness* $(0.20\,cm)$
σ_a	=	*allowable stress for steel*
		(steel is to have minimum tensile stress tolerance of $41\,kg/mm^2$
		$\sigma_a = 1{,}300\,kg/cm^2$ *)*
D	=	*pipe inner diameter*
η	=	*joint efficiency* (0.85)

Minimum steel thickness is computed according to the following formula:

$$t_{min} = \frac{D+80}{40} \geq 0.6cm$$

where:

t_{min}	:	minimum steel thickness (cm)
D	:	inner pipe diameter (cm)

On the basis of the calculations using the above formula, penstock dimensions are computed as follows:

Distance from Penstock Start Point (m)	Inner Diameter of Pipe (mm)	Pipe Thickness (mm)
0.0	--	--
200.0	1,100	6
242.0	"	7
285.0	"	8
360.0	1,050	9
420.0	"	10
585.0	"	11
642.0	0.950	11
737.0	"	12
849.0	0.850	12
895.0	"	13
933.0	"	14
967.0	"	15
982.0	"	16
990.0	0.600	12

5.3 Salient Features of the Project

The following salient features of the Project are determined on the basis of optimum scale selection for the diversion site, power house site, and intervening headrace and penstock route determined as the most appropriate for the envisioned power development.

Diversion water level at the intake site on the Puwa khola is determined at EL 759.0m. Diverted discharge is to be conveyed along an approximately 3.3 km headrace tunnel and then through a 1.0 km penstock to the power house. Tailwater is discharged into the Mai khola at tailwater level of EL 436.7 m.

Salient features for the scheme are as follows:

(1) Catchment area : 125.1 km^2

(2) Scheme type : run-of-river

(3) Intake weir

Type : natural overflow concrete dam

Intake method	:	Tyrolean type (intake channel: W 1.0 m, H. 0.75~2.0 m, L 16.5 m)
Weir height	:	4.0 m
Weir length	:	33.0 m
Overflow crest elevation	:	EL 759.0 m

(4) Headrace

Settling basin	:	standard inner section: W 5.0m, H 3.5 m, fan shaped, L 56.0 m	
Headrace tunnel	:	standard inner section (hood): upper semi circular radius 1.0m, lower section W 2.0 and H 1 m, L 3,200 m	
Head tank	:	W 5.0 m, H 2.0~7.5 m, L 32.5 m	
Spillway	:	inner section (hood): upper semi circular radius 1.0 m, lower section W2.0 m and H1 m	
Regulating pond	:	effective capacity:	2,000 m^3
		water depth:	2.4 m
		surface area:	925 m^2
Penstock	:	steel, 1.10~0.60 m radius, L 990 m	
Tailrace	:	inner section: W 2.0 m, H 2.0 m, L 30 m	
		HWL:	EL 436.7 m

(5) Power house and electro-mechanical equipment

Power house	:	single floor type, 395 m^2		
Turbine	:	horizontal axis Pelton - 2 units		
		output	:	2 × 3,300 kW
		effective head	:	304 m
		maximum discharge	:	2 × 1.25 m^3/s
Generator	:	horizontal axis, 3 phase, synchronous - 2 units		
		capacity	:	2 × 3,700 KVA
		voltage	:	11 kV
		power factor	:	0.85
		frequency	:	50 Hz
Transformer	:	outdoor, 3 phase, oil cooled type - 2 units		
		capacity	:	2 × 3,700 KVA
		voltage	:	11/33 ± 5% kV
Transmission line	:	pole suspension type	:	33 kV × 1circuit
		length to Ilam substation	:	4.7 km

5.4 Civil Structures Plan

5.4.1 Intake Dam

The intake dam site is at a narrow point in the river (river bed around 10 m), about 40 m downstream of the confluence between the Puwa and Ghatte kholas. This point is roughly 10 km upstream from the Puwa khola's confluence with the main Mai khola. River bed elevation at the site is 755 m

Crystalline schist is observed on the left bank to elevation of 800 m. Although this rock exhibits some minor cracking, hard outcropping forms a steep slope of 70° for about 25 m. Above this, slope changes to a gentler 50°. On the right bank, as with the left bank, outcropping of hard crystalline schist forms a gentle slope of 10° to a point roughly 10 m above the river bed. Slope above this continues at an angle of about 25°. The base rock line for this slope is at roughly 1 m depth.

Total width of the river bed is 20 m at the site, with actual width of flow averaging 10 m in dry season. Cobbles of over 5.0 cm diameter and boulders of 3~4 m^3 diameter are sporadically observed. No marked sediment layer is seen in the river bed, with base rock in most cases visible at the surface.

River gradient near the intake site is an extremely steep 1/40, and discharge flow is accordingly very rapid.

An arc shaped landslide zone of 20 m width and 800 m elevation is seen on the hill side of the left bank downstream of the site. However, this will have no impact on safety of the planned diversion structure. The right bank is stable slope and again no problems with regards to the stability of the envisioned structure are anticipated.

At places in the river bed where the base rock is in relatively deep ground, overlying layer will be replaced with concrete ("manmade rock").

The structure at the site is to be a concrete gravity dam with 4.0 m height to overflow crest elevation, and crest length of 33 m. Length of the overflow portion is likewise 33 m. Passing of the design flood discharge of 1,450 m^3/s will be by natural overflow (ogee type dam without gate).

A large amount of sand and gravel load is anticipated due to the fact that the river gradient at the site is very steep, and discharge from the Ghatte khola also enters the Puwa khola at the site. Accordingly, the surface of the overflow portion is to be strengthened with steel rebar reinforced concrete lining.

Grouting of the base rock is not planned as this rock is good crystalline schist, dam height is low and water pressure on the dam body is small. Rather, it is anticipated that careful concrete placement will serve to prevent seepage. The river bed downstream will be carefully improved to allow for smooth passage of the overflow discharge and lower river water level thereby facilitating the removal of sand and gravel carried down by river discharge. Passage of the design flood discharge of 1,450 m^3/s over the 33 m wide overflow portion of the dam will result in an overflow water depth of 5.7 m. Accordingly, overflow water level during flood is determined at EL 764.7 m. Results of dam stability analysis in this regard indicate that the envisioned structure meets required safety conditions against tilting or sliding, and for foundation bearing strength. Dam structure is indicated in Drawing No. ILAM- $^F/_S$ 002 Details for Intake Weir.

The Puwa khola in the Project area is a rapid flow river with falls or rapids observed at several locations, particularly where suspended bridges have been constructed (3 locations: Damai bridge, Chinte bridge, Upreti bridge). Although migration of small fish species upstream is extremely difficult in the drought season, it can be concluded that they have managed somehow to negotiate these points during the high water season. The power generating discharge to be diverted at the dam is a relatively small amount of 2.5 m^3/sec, with the remaining almost all of the river discharge passing freely over the dam. The characteristics of the river will remain essential unchanged, in that the dam creates no greater obstacle to fish migration than the rapids and falls already existing, and as a result no major impact on fish species is anticipated. Nevertheless, some impact will unavoidably occur to animal and plant species dependent on the river for their habitat, although the precise degree of such impact is very difficult to quantify.

On the basis of the above considerations, the fact that the catchment of the Puwa khola is small, and the fact that the main Mai khola is located downstream, no fish ladder is planned for the Project, although impact on fish species will be monitored in the course of the Project.

5.4.2 Intake

There are a number of types of diversion method possible to insure adequate intake from a river with narrow, rapid flow and high suspended load such as that under the Project. However, the Tyrolean type intake which has proven effective under the more severe condition of Tatopani hydropower scheme is to be adopted given topographical and geological conditions at the site, particularly the heavy slope failure activity in the area. This is considered appropriate despite the fact that some sacrifice in available head will occur. Intake gallery is located at the dam overflow crest, with length of 16.5 m, width of 1 m and bed gradient of 1/13.2. The intake will divert a maximum discharge of 2.5 m^3/s to the downstream settling basin.

A regulating gate (2 m × 0.7 m) will be installed at the terminus of the intake gallery. Discharge will pass through this gate and enter the intake chamber (3 m × 3 m × 8.15 m). The top of the chamber will serve as the valve operating platform, with one section (1.0 m × 1.5 m) open to allow access into the chamber for inspection and maintenance of gates and canal. A 75 cm step up will be designed at the face of the regulating gate inside the intake chamber to trap sediment. Above this will be the inlet (H 2 m × W 2.0 m) to the intake tunnel leading to the settling basin and next to it at the right lower portion on the intake chamber floor will be a sediment outlet (H 0.75 m × W 1.0 m) to the side spillway tunnel leading to the sand outlet. A gate (H 1.0 m × W 2.0 m) will be installed at the inlet of intake tunnel opening to allow for maintenance and cleaning of the facility and a gate (H 0.75 m × W 1.0 m) will be installed at a sediment outlet to discharge sediments outside during flood period. Discharge will pass through the 51.0 m length intake tunnel (gradient: 1/1,000) into the settling basin.

The regulating gate will serve to prevent entry of excess discharge into the diversion facilities during periods of high water. Inflow of sediment and mud into the facilities during periods of extremely high water level is unavoidable, and at such time it will be excluded by opening the sediment outlet gate to prevent buildup of sediment within the intake facilities. Also, a side spillway of appropriate dimensions will run from the intake tunnel before the settling basin, to connect with the sand flush tunnel extending from the settling basin. This spillway will serve for runoff of discharge in excess of the maximum design discharge and will be designed with consideration to requirements for uniform flow depth. (Refer to Drawing No. ILAM ~ F/S 002 Details for Intake Weir)
Drawing No. ILAM - F/S 007 is prepared and attached as an alternative design (side intake type) for reference.

5.4.3 Settling Basin

Suspended load contained in diverted discharge has the potential to settle in the headrace canal, thereby reducing flow passing capacity, as well as to enter the penstock and turbine equipment, thereby damaging the same. In order to prevent this, it is a general practice to construct a settling basin as close to the intake facility as practical. In the case of the Project, the settling basin must unavoidably be constructed in the hill area proximate to the intake facility due to the fact that the topography of the immediate area around the intake is steep, and landslide zone is located on the downstream, left bank mountainside. As there is no ponding at the diversion weir, precipitation of suspended load cannot be anticipated there. Accordingly, total length of the settling basin is designed at 56.0 m, with design precipitated particle size determined at 0.2 mm. Average flow speed in the basin is to be 0.3 m^3/s; effective width of flow cross-section is 5.0m; and terminus water depth is 1.81 m. Effective length for the basin is 40 m.

A flow velocity suppression zone of 8 m length, with gradual change in flow section, is to be established at the approach to the settling basin to gradually reduce the speed of discharge entering the basin, in order to allow for settling of smaller suspended particles that would otherwise be carried downstream in the case of excessive approach velocity.

A sand flush gate (1.0 m × 1.0 m) will be installed at the downstream end of the basin to remove sediment.

A 8.0 m flow speed change zone will likewise be included where the settling basin connects to the headrace tunnel to gradually reduce the flow cross-section. If the approach velocity of discharge to the tunnel is too rapid, suspended load is sucked downstream instead of settling to the bottom of the settling basin.

(Refer to Drawing No. ILAM - F/S 003 Details for Settling Basin)

Drawing No. ILAM - F/S 008 is prepared and attached as an alternative design (Double Basin type) for reference.

5.4.4 Headrace Tunnel

The headrace canal comprises a non-pressure tunnel 3,200 m long from the end of the settling basin, with canal gradient of 1/1,660. The tunnel alignment passes through mountain from the left bank of the Puwa khola to the right bank of the main Mai khola. Formation through which the tunnel is routed consists mainly of crystalline schist. Cracking and weathering are observed at one portion of the alignment where it passes through an argillized and weathered zone, as well as at water tank site; however, overall the tunnel route is through stable ground.

Selection of the headrace canal route was made as a result of insitu investigation based on 1/5,000 scale topomapping prepared from 1/25,000 scale topomapping (derived from 1/50,000 scale mapping) and aerophotos.

Since the maximum discharge in the tunnel is a small 2.5 m^3/sec, excavation and concrete sections will be the minimal possible, still allowing for construction work requirements. Water flow section will be H = 2.0 m and W = 2.0 m. Tunnel section is to be semicircular for the upper half and square for the lower half, with concrete lining of 20 cm thickness. Given ground conditions, either no lining or shot-crete could be considered for tunnel wall; however, in consideration of improving the roughness coefficient (0.013) tunnel wall and invert will be concrete lined. Entire tunnel section will be concrete lined as well at juncture points with intake inlet, settling basin, adits, water tank, etc. to ensure structural soundness. Also, in view of the relatively long length of the tunnel, adits will be established in order to facilitate construction works in line with the work schedule, to ensure safety and for operation and management purposes after Project completion.

(refer to Drawing No. ILAM - F/S 001 General Plan)

5.4.5 Head Tank, Spillway and Regulating Pond

The head tank functions to regulate the difference in penstock discharge and headrace discharge as a result of load fluctuation at the power station, as well as acting as the last point at which sediment may be precipitated from discharge prior to its entering the penstock and turbines, and thereby possibly damaging the same.

Design of the head tank will make optimum use of available topography and geology, with location to be between the headrace tunnel outlet and the penstock inlet. Screen (trash rack) will be installed at the intake apron immediately in front of the penstock inlet, and a sand flush valve at the sand flush outlet.

The spillway will serve to divert excess discharge during load regulation safely back into the river. In cases where operation under load regulating conditions is to be performed when the power station is not connected to the national grid, it would be assumed that overflow to the spillway will frequently occur. However, in the case of the Project which is envisioned to be connected to the grid, under which conditions water level regulated operation is performed in response to discharge amount, overflow to the spillway would be relativity limited.

Generally, such a spillway is appurtenant to the head tank. However, as no river is located nearby for safe runoff of excess discharge, the adit for tunnel construction is to be used as a spillway connecting to the Puwa khola.

Also, in order to enable peak generation during the dry season as well, a regulating pond is to be constructed at the side of the head tank with maximum capacity in conformity with the available topography. Regulating capacity is a somewhat small 2,000 m^3; however, effective depth is 2.4 m resulting in a water pressure value which is not excessive. Nevertheless, it will be necessary for careful concrete lining of the pond walls and bottom in order to prevent adverse impacts from seepage, etc. A drainage channel, regulating gate, and if necessary diversion facility for irrigation will be included at the head tank or the head tank or pond.

(refer to Drawing No. ILAM - F/S 004 Head Tank and Reservoir and Drawing No. ILAM-F/S 009 Details for Spillway [Adit])

5.4.6 Penstock

It was confirmed on the basis of site reconnaissance, seismic prospecting, and test drilling, that base rock in the Project area is crystalline schist.

The penstock route alignment begins in a southeastern direction along the ridge leading away from the headtank for a distance of 850 m down a slope of 5~20°. At this point the alignment shifts to a southerly direction for 150 m down a slope of about 40°. Just before the power station, the penstock will bifurcate into two pipelines connecting to the respective inlet valves leading to the two turbines inside the power station.

Distance from ground surface to base rock is estimated at 2.0~3.0 m for penstock route on gentle slope, and less than 1 m for penstock route on steep slope.

Penstock is to consist of steel piping, of lengths appropriate for transport to the site. Sections are to be welded insitu.

Anchor blocks will be constructed at required points along both bends and straight segments of the penstock route. Saddles will also be set at appropriate interval between anchor blocks.

Penstock diameters are as follows:

Ø 1,100 mm from the headtank to 285 m downstream.
Ø 1,050 mm from 285 m to 585 m downstream
Ø 950 mm from 585 m to 737 m downstream
Ø 850 mm from 737 m to 982 m downstream
Ø 600 m from the point of bifurcation

At the point of intersection between road and penstock alignment, care will be given to penstock design which will not impede traffic.

(refer to Drawing No. ILAM - F/S 005 - 1/4 ,2/4, 3/4 & 4/4 Penstock)

5.4.7 Generating Equipment Foundation

(1) Type of Foundation

Foundation for generating equipment is to be of reinforced concrete, and independent of the power house foundation (including power house floor).

This foundation must be established on firm rock with sufficient bearing capacity for the envisioned generating equipment

(2) Design Criteria

1) Equipment Weight (2 units)

Reinforced concrete foundation (2 units)	@ 134.2 t	268.4 t
Turbine (3,300 kW × 2 units)	@ 3.7 t	7.4 t
Generator (3,700 kVA × 2 units)	@ 21.5 t	43.0 t
Inlet valve (600 dia. × 2 nos.)	@ 10.0 t	20.0 t
Transformer (outdoor type × 2 units)	@ 19.0 t	38.0 t

2) Foundation Condition

Figure 3.6-9 indicates the subsurface conditions at the site. According to the findings of test drilling No. 3 carried out under the subject Study, ground consists of colluvium with soil and smaller fragments of rock from the surface to a depth of 4.30 m. N values are N = 20 at 1.0 m depth and N = 5 at 3.5 m depth. Ground below this layer is terrace deposit composed of fine to medium grained sandy soil. N value in this layer is N = 10 at 5.0 m depth from the surface. N value below this markedly increases to 14, 15 and then 16. At 7.0 m depth from the surface, N becomes 20 or more.

Given the weight of design equipment, the foundation bearing layer must be capable of supporting a load of 10 t / m², without the occurrence of nonuniform subsidence.

Accordingly, in light of the need to upgrade foundation, ground to 5.0 m depth is to be replaced with sand and gravel, and refill performed thin layer by thin layer, with each layer compacted by vibration compactor in order to improve the N value of the bearing layer to ≥ 10.

The following Table 5.4-1 describes each type of equipment and foundation.

Table 5.4-1 Generating Equipment Foundation

	Main Equipment	Equipment Weight	Installed Location	No. of Units	Dimensions and Weight of Foundation	Foundation Type	Improvement Works for Bearing Layer
1.	Turbine Runner Casing Inlet Valve Generator Stator Rotor Total	3.7 t x 2 = 7.4 10 t x 2 = 20 t 21.5 t x 2 = 43.0 t 35.2 t x 2 = 70.4 t	inside P/H	2 units	(2.6 x 2.6 x 3.3 + 1.5 x 3.3 - 1.2 x 1.5 x 1) x 2.4 = 134.2 t 134.2 t x 2 = 268.4 t	direct foundation contact pressure: $\frac{134.2+35.2}{18.76} = 9.03\ t/m^2$	earth replacement (sand works) to depth of 5.0 m from surface target N value: N ≥ 10
2.	Transformer	19.0t x 2 = 38.0 t	outside P/H	2 units	12m x 5.5 m = 66 m² 0.5 × 66 × 2.4 = 79.2 t	direct foundation contact pressure: $\frac{79.2+38}{60} = 1.8\ t/m^2$	
3.	Overhead traveling crane (16 t load)	@ 8.5 t x 4 wheels = 34.0 t	inside P/H	1 unit	vertical column weight = 44 t foundation footing = 2.2 m² = 4.84 m²	direct foundation contact pressure: $\frac{44}{4.84} = 9.09\ t/m^2$	
4.	Auxiliary electrical equipment	100 kg - 300 kg	inside P/H	2 sets		to be supported by power house structure floor	

5.4.8 Power Station

The site for power house structure and appurtenant facilities is located on the right bank of the Mai khola, about 700 m downstream of the Rajbuwali truss bridge.

The power station will contain a power generating equipment room and a switchboard room. A ceiling hoist will be installed in the power generating equipment room for assembly and maintenance work.

Floor surface level of the generating equipment room will be at ample height to compensate for predicted flood water level. Leakage occurring within the power station will be gravity drained outside the structure.

The power house structure will be insulated from the turbine and generator foundation. The weight of the power house structure itself as well as the live load of the overhead crane will be distributed to columns, through which load will be conveyed to the column foundation footings and from there to the foundation rock.

According to calculation, direct load to the vertical columns where live load of the overhead crane is greatest is 44 t. Accordingly, if footing dimensions are set at 2.2 m x 2.2 m, average contact pressure with the ground is 9.09 t/m², and the said footings must accordingly rest on foundation with N value of ≥ 10.

(1) Building Scale

1) Main Equipment

Turbine and appurtenant equipment	2 units
Turbine	
Inlet valve	
Governor	
Water supply system	
Generator and appurtenant equipment	2 units
Generator	
Exciting unit	
Supplemental equipment	
Total area for turbine and generator equipment:	@ 2.00 m² × 2
Overhead traveling crane	
Clearance	4.5 m
Max. lifting weight	16 t
No. of saddle wheels	2 × 2 = 4 wheels
Max. wheel load	8.5 t

Operation control panel	
Operation panel	2 units × 1.0 m × 2 m = 4 m²
Control panel	5 units × 1.5 m × 0.8 m = 6 m²
Transformer panel	5 m × 3.5 m = 17.5 m²
High voltage panel	4.5 m × 2.0 m = 9 m²

2) O/M Staff

Manager (cum engineer)	1
Civil engineer	1
Electrical engineer	1
Electrical technician	3 (3 shifts)
First grade technician	3 (3 shifts)

3) Required Area

The power house is built as a one storied structure. The required areas at each use are as follows:

① Turbine/generator room (2 units)

9.5 m × 20.0 m = 190.00m² (c.t.c. of column)

② Unloading bay (cum machine shop)

9.5 m × 5.0 m = 47.5 m²

③ Control room area

6.3 m × 20.0 m = 126.0 m²

④ Battery room area

6.3 m × 5.0 m = 31.5 m²

⑤ Transformer yard (outdoor)

5.5 m × 12.0 m = 66.0 m²

(2) Outline of Structure and Specification

The power house building is a one storied structure composed of a main building (eave height = 7.6 m), and attached lean roof building (eave height = 4.0 m). Total floor area = 395.0 m². An outline of the structure and its specifications are as shown in Table 5.4-2.

Table 5.4-2 **Power House Structure and Specifications**

Building/room		Structure	Roof	Exterior wall / door & window	Floor	Interior wall / partition wall	Ceiling
Main building	Generating room Unloading bay	-rigid frame -RCC	-RCC -sloping roof -water proofing	-RCC with mortar finish -steel door and window sash -paint coating	-plain concrete (1:3:6) -mortar finish, sealer coating	-mortar finish, paint coating	none
Lean roof building	Control room Battery room	-rigid frame -RCC	-RCC -sloping roof -water proofing	-brick wall with mortar finish -steel door and window sash -paint spray	-plain concrete (1:3:6) -mortar finish, sealer coating	-brick partition wall -mortar finish, paint coating	none

Note:
1. Ventilator, cooling unit, ceiling fan and kitchen unit will be installed.
2. Lighting will be mainly flourescent lamps

(3) Power House Site

The site is located at the above described location occupying a space of 2,000 m^2. The masonry wall along the Mai khola side is to be 100 m in length. The site area is to be graded level at EL 437.7 m. Elevation of the top of the masonry wall is EL 437.0 m. Design flood water level at the Mai khola power house site is EL 436.7 m.

(refer to Drawing No. ILAM - $^F/_S$ 006 - Power House Layout)

5.4.9 Tailrace

Design tailwater level at the power house site is EL 436.7. Even in the worst case where the water rises to the design flood water level, ample free board is achieved in that discharge is emptied into the Mai khola via the tailrace to a point about 30 m downstream of the power house site.

The tailrace is designed as open, masonry lined canal.

5.4.10 Access

The Project area is located in rugged terrain, particularly with regards to the intake site on the left bank of the Puwa khola (EL 760), and the adit site (EL 755 m) to which access by existing road is not possible.

Accordingly, it is necessary to construct new access road to these points as well as improve portions of existing road. Construction access road is to be 2.5 m width with road gradient of 10~15%, with turn arounds to deal with on-coming traffic.

Lengths of access road required to the intake, adit and head tank sites are about 2.6 km 1.5 km, and 0.3 km respectively. Access road will be paved with crushed stone produced at the river bed.

A. submerged road made of gabions and concrete pipes is to be constructed acrossing Mai Khola (river width 50m) for transporting heavy equipment to the Power house.

A road of about 700 m length for operation and maintenance of generating equipment will be constructed along the rights bank of Mai Khola.
(refer Map No. A-I-1 NEPAL-ILAM 1/10,000 in Annex)

5.5 Power Generating and Transmission Equipment Plan

5.5.1 Power Generating Equipment

Two units of power generating equipment will be installed in the power house given the requirements for transport, and operation and maintenance prevailing in Nepal.

(1) Turbine

In light of the effective head of approximately 300 m and power generating discharge of 2.0 m^3/s, the horizontal axis, pelton type turbine is to be adopted. Preliminary specifications for this equipment are as follows:

No. of units:	2
Type:	horizontal axis, 2 nozzle, pelton type
Output:	3,300 kW
Effective head:	304 m
Discharge:	2.5 m^3/s
Rotating speed:	600 rpm
Specific speed:	19.2 m-kW
Needle closing time:	30 sec
Deflector closing time:	2 sec
Water pressure rise factor	< 10%
Speed fluctuation factor	< 50%
Turbine start up:	both manual and automatic
Turbine shut down:	automatic

(a) Inlet valve

Type:	sluice value
Operation:	manual operation

(b) Governor

Type:	electric or mechanical
Needle operation:	motor operated or oil servometer
Deflector operation:	motor operated or oil servometer

(c) Water supply system
(refer to Figure 5.5-1 Power House)

(2) Generator

Excitation is to be brushless from the standpoint of ease of maintenance. Preliminary specifications for this equipment are as follows:

No. of units:	2
Type:	horizontal axis, 3 phase, synchronous
Output:	3700 kVA
Voltage	11 kV
Power factor	0.85
Frequency	50 Hz

Insulation class	F class
Cooling method	Outlet duct

(refer to Figure 5.5-1 Power House)

(3) Control

Semiautomatic control system is to be adopted under which a minimum of 2 staff can operate the turbine and generator equipment, and perform the necessary controls and measurement. As the Ilam area will be connected to the national grid prior to completion of the Project, synchronous operation of generating equipment under the Project (cycle, voltage) with the national grid will be the normal case. In the event of accident or emergency occurring in the grid, the power house under the Project will shut down as well and not resume operation until the problem affecting the grid has been resolved. However, in cases where accident on the grid is prolonged, the Project is designed such that the circuit breaker connecting to the grid would be opened, and independent operation performed for power distribution from the Ilam substation. When the accident affecting the grid is resolved, the power house would be shut down, the above described circuit breaker closed, and the generating equipment restarted.

The control room will house an 11 kV switchboard, low voltage switchboard, direct current power source for control of power house operation.

(refer to Figure 5.5-2 Single Line Diagram for Power House)

5.5.2 Transmission Equipment

Transmission under the Project is premised on the existence of the Ilam substation (33 kV / 11 kV, 400 V, 3 MVA) to be completed under the 7th Power Project. Power voltage from the generators will be stepped up at the power house substation to 33 kV, and power then transmitted along the 4.7 km, 33 kV line to be constructed under the Project to the 33 kV primary side of the Ilam substation where the power generated by the Project will then be fed into the national grid.

(a) Transformer

No. of units:	2 sets
Type:	outdoor, 3 phase, oil immersed, self cooling type
Capacity:	3700 kVA
Voltage	11 kV / 33 kV (± 2.5~5%)

(b) Cubicle (high tension panel)

No. of units:	1 set
Type:	33 kV indoor type
Circuit breaker:	vacuum or gas circuit breaker

5.5.3 Transmission line

Span between transmission line poles is to be 75 m, with a dead end type pole set additionally at each 1 km. Poles are to be of the galvanized tubular steel type. Conductor is to be ACSR (aluminum conductor steel reinforced), 3 phase, 3 wire and dog size (100 mm^2 section).

Design specifications are:

Outdoor temperature:	average: 30°C maximum: 40°C minimum: 0°C
Wind pressure load:	75 kg/m^2
Voltage:	33 kV
No. of circuit:	single
Conductor:	ACSR, "dog" 100 mm^2
Insulator:	pin insulator for pole (type-A) and suspension insulator for pole (type-B)
Pole:	galvanized tubular steel, assembly type

CHAPTER 6

CHAPTER 6 IMPLEMENTATION PLAN

6.1 Implementation Schedule

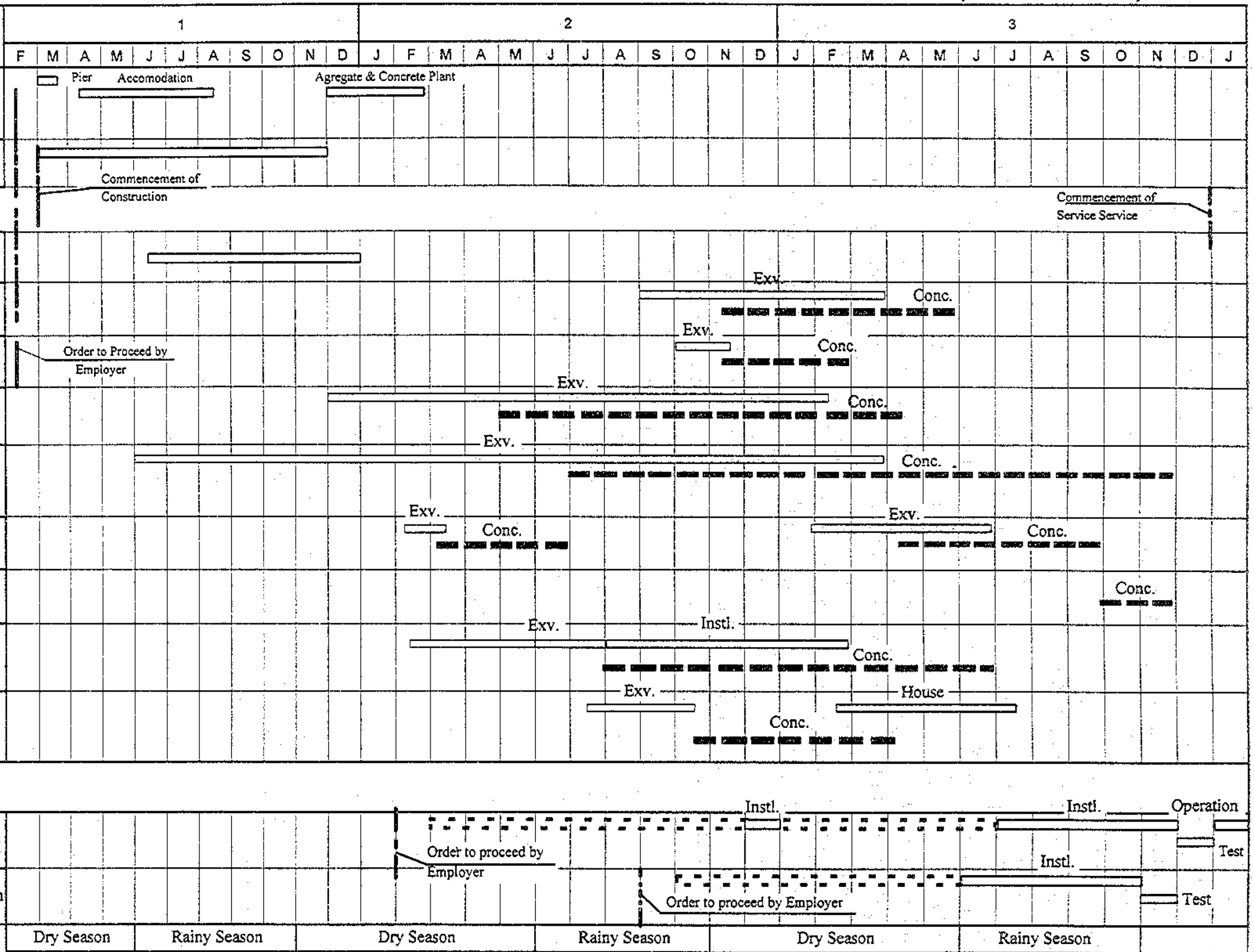
The facilities under the Project comprise mainly civil works including intake weir, settling basin, headrace canal tunnel, head tank with regulation pond, penstock and power house. The site area for the foregoing structures comprises extremely rugged terrain. Particularly in the case of the intake site and adit to be excavated along the tunnel route, there are no existing roads passable by construction vehicles and equipment. Accordingly, securing access to the said intake and adit sites for equipment and materials will be a key to the entire construction schedule, and so new road construction, widening of existing road and installation of necessary winch equipment will be given maximum priority.

The entire construction period under the project will require 36 months as indicated in Table 6.1-1.

Rainy Season : June ~ October
Dry Season : November ~ May

Table 6.1-1 Construction Schedule

ITEM	QUANTITY
PREPARATORY WORKS & TEMPORARY FACIRITIES	
ACCESS ROAD	Lgth. 3600 m
CIVIL WORKS	
CONNECTION ROAD	Lgth. 1500 m (ADIT)
INTAKE DAM	Exv. 490 m³ Conc. 1350 m³
INTAKE	
SETTLING BASIN	Exv. 2050 m³ Conc. 700 m³
HEADRACE	Lgth. 3200 m Exv. 17610 m³ Conc. 5740 m³
HEADTANK	Exv. 3240 m³ Conc. 2210 m³
SPILLWAY	Exv. 590 m³ Conc. 240 m³
PENSTOCK	Exv. 4110 m³ Conc. 1670 m³ Instl. 990 m
POWERHOUSE	Exv. 1680 m³ Conc. 1065 m³ House 395 m²
ELECTRO-MECHANICAL	
TURBINE & GENERATOR	3,700 KVA 2 units
TRANSMISSION LINE& SUBSTATION	33 KV cct × 4.7 km
REMARKS	



NOTE : Excavation — Concreate — Other Works

Exv. : Excavation
Conc. : Concrete Works
Instl. : Instalation

6.2 Preparatory Works

6.2.1 Construction Access Road

To achieve access to the Project intake site it will be necessary to construct 1.0 km of new road and widen 1.6 km of existing road from Ilam Bazar.

In order to carry out this construction in a timely manner, construction will be commenced simultaneously at multiple locations along existing foot paths. Road will have effective width of 2.5 m and gradient of 10~15%. At suitable locations, greater road width will be taken to establish turn-abouts to allow passage of traffic from opposing directions. In addition, new road construction will also be done for 1.5 km, for access to adit site. As tunnel construction is a critical path for the overall implementation schedule, the foregoing access road works will be commenced from 3 locations simultaneously.

Head tank and penstock route are located in relatively gentle terrain proximate to existing road. Existing road will be used to the extent possible, and 0.3 km of new road constructed for access to these sites. Where roads connect to existing district roads and farm roads, thorough liaison will be maintained with all concerned officials and personnel to ensure smooth coordination of works.

A submerged road of 50 m across the Mai Khola will be necessary for transport of equipment and materials to the power house site. However, 0.7 km of new road will be constructed along the downstream right bank from Mai khola bridge in view of high water levels anticipated during construction and operation and maintenance of the power house after construction.

6.2.2 Construction Equipment and Temporary Facilities

Main construction equipment for civil works under the Project are as follows:

Table 6.2-1 Construction Access Road

Item	Specification	No. of Units	Remarks
Bulldozer	D31A	2	for excavation/embanking
Crawler drill	air consumption: 3 m^3/min	2	for excavation
Leg drill	air consumption: 2 m^3/min	6	for excavation
Pick hammer	air consumption: 1 m^3/min	6	for excavation
Dump truck	6~7 t capacity	4	for transport
Backhoe	0.25 m^3	3	for excavation/loading
Compressor	mobile type	2	for excavation

Table 6.2-2 Intake Facility

Item	Specification	No. of Units	Remarks
Crawler drill	air consumption: 3 m^3/min	1	for excavation
Leg drill	air consumption: 2 m^3/min	2	for excavation
Pick hammer	air consumption: 1 m^3/min	6	for excavation
Bulldozer	D31A	2	for excavation (6t)
Backhoe	0.25 m^3	1	for excavation/loading
Dump truck	6t~7t capacity	2	use among multiple sites
Truck	4t~6t capacity	2	use among multiple sites
Vibrator	4t~6t capacity	3	for concrete works
Submerged pump	Ø 8"	1	use among multiple sites
Submerged pump	Ø 6"	2	use among multiple sites
Submerged pump	Ø 4"	2	use among multiple sites
Generator	45 kVA	2	use among multiple sites
Compressor	air consumption: 8 m^3/min	2	use among multiple sites
Truck crane	5 t lift capacity	1	use among multiple sites
Mixer	0.6 m^3	1	for concrete works
Tunnel support	steel	1	for concrete works
Rail	15 kg/m	500 m	for concrete works

Table 6.2-3 Tunnel

Item	Specification	No. of Units	Remarks
Leg drill	air consumption: 2 m^3/min	4	for excavation
Pick hammer	air consumption: 1 m^3/min	4	for excavation
Rocker shovel	RS20k class (0.17 m^3)	2	for excavation
Bulldozer	D31A	1	for disposing of muck (6t)
Submerged pump	Ø 6"	2	use among multiple sites
Submerged pump	Ø 4"	4	use among multiple sites
Submerged pump	Ø 3"	4	use among multiple sites
Concretete plant	15 m^3/h	1	0.6 m^3
Vibrator		4	for concrete works
Tunnel support	steel	2	for concrete works
Generator	100 kVA	2	use among multiple sites
Compressor	air consumption: 8 m^3/min	2	use among multiple sites
Ventilator	20 kW	2	use among multiple sites
Truck crane	5 t lift capacity	1	use among multiple sites
Rail	15 kg/m	7,000 m	for concrete works

Table 6.2-4 Headtank and Regulating Pond

Item	Specification	No. of Units	Remarks
Bulldozer	D31A	2	for excavation/refill (6t)
Crawler drill	air consumption: 3 m^3/min	1	for excavation
Leg drill	air consumption: 2 m^3/min	4	for excavation
Pick hammer	air consumption: 1 m^3/min	4	for excavation
Backhoe	0.25 m^3	2	for excavation/loading
Dump truck	6t~7t capacity	4	for transport, refill
Dozer shovel	0.25 m^3 capacity	2	for excavation/refill
Truck mixer	2 m^3 capacity	2	for concrete works
Vibrator		4	for concrete works
Submerged pump	Ø 3"	3	use among multiple sites
Track crane	5 t lift capacity	1	use among multiple sites
Generator	45 kVA	2	use among multiple sites

Table 6.2-5 Penstock

Item	Specification	No. of Units	Remarks
Crawler drill	air consumption: 3 m^3/min	2	for excavation
Leg drill	air consumption: 2 m^3/min	4	for excavation
Pick hammer	air consumption: 1 m^3/min	4	for excavation
Bulldozer	D31A	2	for excavation/refill (6t)
Backhoe	0.25 m^3	2	for excavation
Dump truck	6t~7t capacity	2	for transport
Truck mixer	2 m^3 capacity	2	for concrete works
Vibrator		4	for concrete works
Truck crane	5 t lift capacity	2	use among multiple sites
Generator	45 kVA	2	use among multiple sites
Welder	150 A	2	for steel pipe welding
Winch	single trunk, 15 kW	2	for steel pipe lifting
Rail	15 kg/m	700 m	for steel pipe

Table 6.2-6 Power House and Tailrace

Item	Specification	No. of Units	Remarks
Bulldozer	D31A	1	for excavation/land preparation
Backhoe	0.25 m^3 capacity	1	for excavation/refill
Dump truck	6t~7t capacity	2	for transport
Truck mixer	2 m^3 capacity	1	for concrete works
Vibrator		2	for concrete works
Submerged pump	Ø 4"	2	use among multiple sites
Submerged pump	Ø 3"	2	use among multiple sites
Truck crane	5 t lift capacity	1	use among multiple sites
Generator	45 kVA	1	use among multiple sites
Concrete plant	30 m^3/h, 45 kVA	1	use among multiple sites
Aggregate plant	20 kVA	1	for steel pipe welding

Principal temporary facilities at the Project site will comprise office, lodging, storage, explosives magazine, access road, power supply, water supply and sewerage, aggregate plant, concrete plant, etc. Placing of these facilities will take into consideration factors of operation and maintenance, possible incorporation into permanent facilities after construction, and removal where necessary following completion of the Project.

6.2.3 Principal Materials and Equipment

Table 6.2-7 Main Materials and Equipment

Item	Specification	Qty.	Remarks
Cement	Standard portland cement	3,500 t	
Rebar	deformed bar	230 t	
Gate	B = 1.0~3.0 m	6 nos.	
Steel pipe	Ø 1.10 m ~ Ø 0.60	990 m	for penstock
Turbine	3300 kW	2 nos.	horizontal pelton
Generator	3,700 kVA	2 nos.	
Main transformer	3,700 kVA	2 nos.	
Transmission line	33 kVA x 4.7 km	1	Power house ~ Ilam substation (under planning)

Main materials and equipment are as indicated in Table 6.2-7. Concrete aggregate is to be obtained as described below.

Total concrete volume under the Project is estimated at 13,300 m^3, excluding that required for temporary facility foundations. Coarse and fine aggregate quantity is 26,500 t. Aggregate for concrete is planned to be obtained from the gravel sediment layer in the river bed near the power house site.

It would be considered possible to utilize as well deposits in the river bed near the intake dam site, as well as excavated muck from the headrace tunnel, however this is not recommended due to time and cost entailed in the setting of quarrying and sorting equipment at multiple locations, as well as the fact that only small quantities in relation to the total aggregate requirement can be obtained.

The sediment layer in the river bed proximate to the power house site is capable of yielding a large volume of good quality aggregate. Although a breaker must be installed, coarse and fine aggregates can be obtained by filtering and washing alone. Accordingly, quarry site for aggregate will be at this location only for the entire Project construction. Stockpiling by grain size will be performed at the quarry site, for subsequent supply to the concrete materials depots at each of the construction sites. Concrete would then be mixed insitu at each of the separate construction sites.

6.2.4 Power for Construction

As there is no access to the existing power grid from the site, diesel generators will be set up at the site to provide power for construction works. The following generating equipment is considered necessary for construction under the Project:

Table 6.2-8 Power for Construction

Site	Unit capacity (kVA)	No. of units	Total capacity (kVA)	Remarks
Owners office	25	2	50	for owner's site office, lodging facilities, and water supply and sewerage
Contractor's office	25	2	50	for contractor's site office, lodging facilities, and water supply and sewerage
Access road for construction	25	2	50	
Intake	45	2	90	including settling basin
Adit	100	2	200	compressor, concrete plant
Head tank	45 35	1 1	80	including head tank
Penstock	25	3	75	
Power house	45	1	45	
Aggregate plant	20	1	20	
Concrete plant	45	1	45	2 x 0.6 m^3 mixer
Total			705	

6.2.5 Transport of Heavy Equipment

As Nepal is a landlocked country, equipment and materials to be brought in from off-shore for the Project (turbines, generators, switchyard equipment, penstock steel, heavy construction machinery, etc.) must be off loaded at Calcutta, and carried overland via Biratnagar (in Nepal) to the Project site. The single heaviest item is the generator rotor, with unassembled weight of 19 t, and freighting dimensions estimated at 6.0 m height, 1.6 m width and 1.6 m length. Although there are no tunnels along the mountain road from Anarmani to the Project site, road gradient, turn radius and other road conditions are such that prior survey is necessary to determine constraints on heavy load transport. This particularly pertains to the status of bridge load limits and truss beam materials. In some cases it will be necessary to construct temporary bridges and detours in order to transport equipment and materials required under the Project. Also, transport of the foregoing equipment should avoid the rainy season.

6.3 Intake Facilities

6.3.1 Intake Dam

Construction of the intake weir and other intake facilities will commence in the dry season following completion of the above described access road .

As base rock is exposed in the river bed at the intake site, it is estimated that the layer of deposited sand and gravel is thin (around 1 m). Downstream, however, this layer is considered to be somewhat thicker. Cobbles and boulders are also observed in the vicinity upstream . Construction will be commenced in the dry season when river flow is stable. Firstly, a portion of the river bed at the right bank side will be excavated in the direction of discharge flow to create a temporary bypass channel 5.0 m wide and 2.0 m deep. Next, coffer dams will be constructed immediately upstream and downstream of the dam site, and river discharge diverted to the temporary bypass channel. Excavation and concrete works for the left bank side of the dam structure and intake would then be performed. At about the same time, dam abutment works and excavation at both banks to stabilize slope outside the river bed would be commenced.

When the left bank dam structure and intake facility have been constructed to a height above the dam crest elevation, the coffer dams upstream and downstream will be shifted to close off the left side of the river. River discharge would be routed at this time through the sand flush gate on the left bank and the overflow portion of the left bank side of the dam. Dam foundation excavation and concrete works would then be commenced for the right bank side.

Coffer dam materials will consist of river bed gravel, cobbles and clayey soil located nearby, and sandbags, gabions, etc.

Concrete for dam construction will be prepared to the designated mix by concrete mixer (0.6 m^3 capacity) in the vicinity of the site, using the specified aggregate quarried nearby, and then transported to the dam site for placement. In placing concrete, foundation treatment, blocking, number of days for concrete curing, jointing, etc. will be carried out in strict accordance with dam construction specifications.

Given the conditions affecting transportation into and at the site, reserves of materials required for concrete preparation should be stockpiled in the site vicinity in advance of construction.

At the same time, large boulders upstream of the site judged to impede discharge flow will be removed where necessary to ensure smooth flow downstream.

6.3.2 Intake Inlet

Intake inlet is the Tyrolean type, 16.5 m in length in the direction of the dam axis, and located along the dam crest. Width is to be 1.0 m and depth 0.75~2.0 m. Water is diverted along this channel through a regulating gate and into the intake chamber on the left bank. Distance to the entrance to the intake tunnel is 20.5 m.

Accordingly, the intake structure can be considered as an integral part of the dam structure itself, and foundation treatment and concrete placement for the same will be done while river flow is diverted to the right bank side. When river flow is switched back to the left bank side, discharge flow into the intake will be shut off by the gate and discharge passed over the dam crest. The construction of the upper structure above the gate and the top of the intake chamber will then be carried out unaffected by river flow.

The tyrolean intake inlet at the overflow crest of the dam will be reinforced with durable concrete, rubber, steel and tough screen to withstand wear from sediment carried by stream flow.

6.3.3 Settling Basin

Due to the topography at the site, the settling basin is to be constructed underground. Accordingly, it will be necessary to excavate a large cavern of dimensions W = 5.0 m, H = 3.5 m and L = 56 m, with a resultant long span of concrete lining. The settling basin will include a sand flush gate to remove sediment and 170 m long sand flush canal. This will be used during construction for the removal of excavated muck and transport of materials.

Depending on rock condition, rock bolts and steel support will be used during excavation. For safety, the arch portion of the tunnel section will be excavated first, with concrete lining to be by the retrograde lining method. For concrete lining, concrete tunnel support will be used, with length of the same to be determined on the basis of placeable concrete amount.

6.4 Headrace Tunnel

6.4.1 Excavation

Due to the length of the planned tunnel (3,200 m downstream of the settling basin), its excavation will control the overall construction schedule for the Project. It is necessary to set up a tunnel excavation schedule which allows sufficient leeway for the possibility of encountering points of poor geology or water outpour, which would slow down progress. As the minimum cross-section possible for construction is to be adopted, with construction works thus being performed under rather constrained conditions, and the tunnel itself is long, a work adit will be excavated at the mid point of the tunnel to facilitate works.

Tunnel excavation is to be carried out from 2 faces, with length of each excavation segment at a about ratio of 3.3 : 6.7 vis a vis total tunnel length (upstream segment : 2,133 m; downstream segment: 1,050 m)

The entire tunnel cross-section is to be excavated at once, with muck to be removed by manually pushed rail trolley. Work is to be done in two shifts with a design progress rate of 4.5 m/day. Excavating equipment will consist of 2 units of leg drill and 2 units of pick hammer, with muck loading by rocker shovel (RS 20 K 0.17 m^3). Turn abouts will be established to allow for smooth passage of two directional traffic in the course of muck removal and carrying in of concrete and construction materials.

Steel shoring materials will be used to ensure safety during construction. Drainage pumps, piping and rock bolts will also be on hand to cope with sections of poor geology and water outflow. Ventilating equipment will also be installed for exhaust of dust and gas, to maintain both safety in the excavation shaft and to improve work efficiency.

6.4.2 Lining Works

Lining works will follow upon excavation. Concrete placement will be by the separated method, by which the entire arch portion of the excavated section will be lined first, to be followed by invert concrete placement. Lining is to be performed for the entire tunnel length to achieve a satisfactory roughness coefficient for tunnel wall. This will be done regardless of the surface quality of raw rock after excavation.

A concrete plant will be set up at the site (0.6 m^3 capacity). Mixed concrete will be transported by rail trolley to the placement site. Metal forming will be used for concrete placement for arch portion and tunnel wall. Design progress for concrete lining is 12 m/day.

As concrete lining is the final stage of tunnel construction, piping, rail, shoring materials, pumps, etc. will be removed simultaneous to the progress of this work.

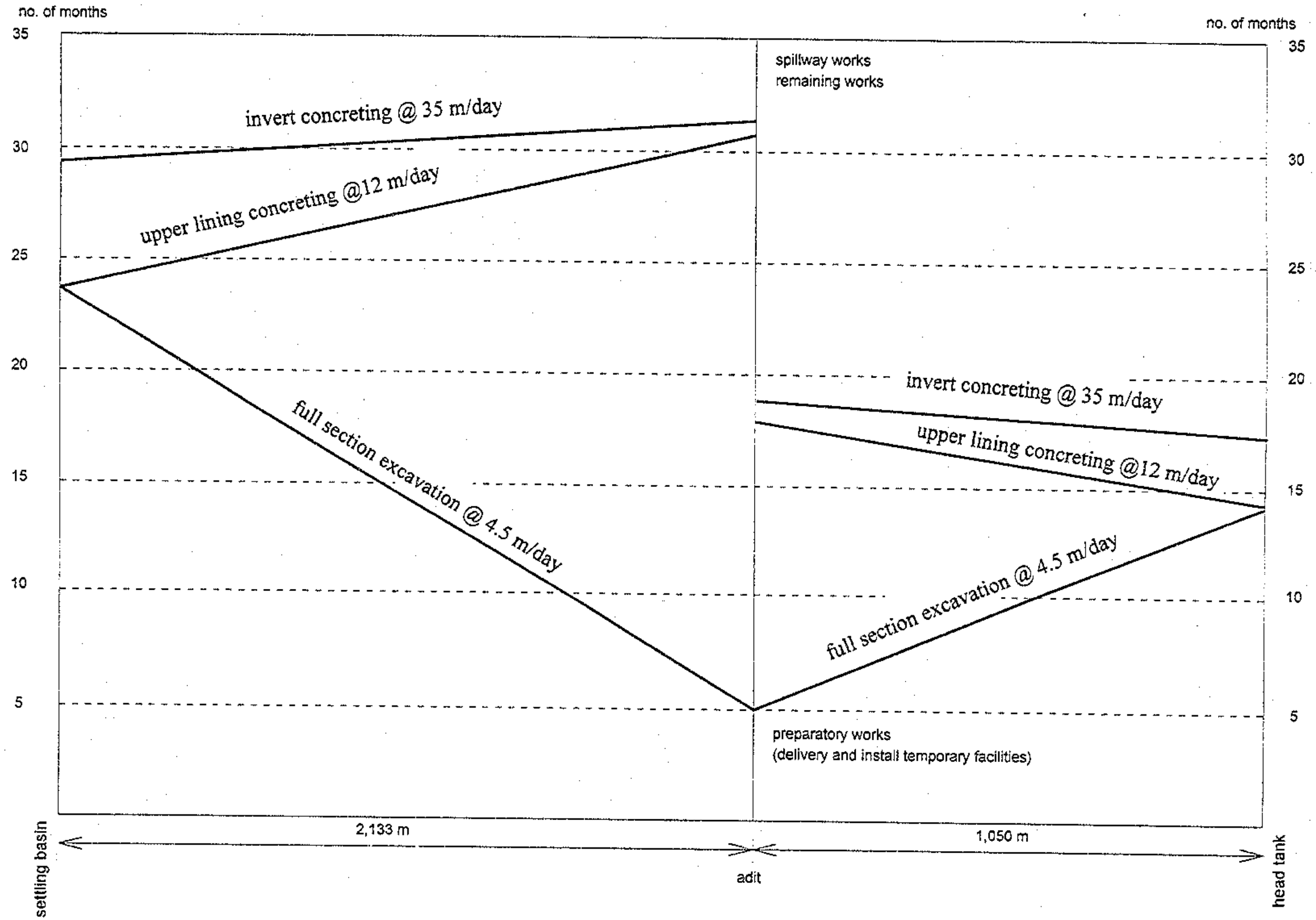
Placement of muck removal route, compressor, engine, concrete plant, cement storage, water supply and drainage facilities, explosives magazine, etc. must be laid out in the vicinity of adit and tunnel openings to ensure maximum efficiency of work.

Access roads must also be maintained in good condition to permit tunnel excavation works during both dry and rainy seasons.

6.4.3 Construction Schedule

Headrace tunnel construction schedule is as shown below.

Headrace Tunnel Construction Schedule



6.5 Head Tank with Regulating Pond

The head tank at the end of the headrace tunnel and the regulating pond on the west side of the tank are sited on a gentle slope. Both will be of reinforced concrete with respective depths of 2.0~7.5m and 3.9 m. Care must be taken in concrete placement to prevent seepage in the concrete walls of the head tank at higher points. This includes ensuring good concrete placement conditions, the use of cut-off plates and appropriate treatment of joints such as elimination of joint laitance.

Regulating pumps will be installed wherever topography requires and the plane figure of the regulating pond will thus be irregular. However the construction area is an extensive 2,000 m, allowing the use of machinery. Cutting, banking, filling, concrete forming, and the transport and placement of concrete must all proceed according to an efficient project schedule. As the structure must be watertight in order to fulfill its purpose great care must be taken in making placement joints as well as in collecting and removing permeated water carrying out inspections etc.

6.6 Penstock Pipe

The penstock pipe which is 990 m in total length, runs in a fairly straight course over relatively gentle sloping terrain except for the final 150m section which makes a large vertical and horizontal turn down the steeply sloped portion of the route (30~40°). Pipe diameter ranges from 1.10 m~0.60 m and the pipe's total weight is about 240 t requiring 12 anchor blocks and 150 saddles. Construction works will be divided into three sections in consideration of topography and construction schedule and excavation of the foundation concrete placement, transportation of the pipe, hoisting, and installation works will be separately undertaken for each section. The works will be implemented after ensuring a sufficient safety margin for such items as working hours, blasting schedule and installation of protective fencing.

Works undertaken in the gently sloped area present no difficulty; however, highly experienced staff are required for safe and efficient implementation of works in the steep section.

Concrete will be produced in the concrete plant erected for the Project and carried to the site by concrete mixer for placement, and a chute will be used in the steep section.

As the diameter of the pipe is small, steel piping will be light and easy to use. However, as the sections to be installed are quite long, equipment such as inclines and cranes may be used when necessary. The penstock pipe will be installed underground where it intersects the main road, and care will be taken during construction to avoid any impediment of traffic.

6.7 Power House

The power house area which covers 400 m^2 including the generating plant and transformer yard is comparatively large and level, facilitating construction work. The results of bore samples revealed that the first 6 m under the site is comprised of colluvium with soil and smaller fragments of rock which has poor ground bearing strength. A firm foundation with safe bearing strength is necessary to support the heavy rotary equipment such as the turbine and generator, as well as the load which will be placed upon it during hoisting of the machines by overhead crane. There are many possible approaches to this problem; however, in this case the weak surface layer will be removed and replaced with pour-mix concrete to provide the necessary bearing strength.

It is possible that failure of the cut slope could occur during excavation of the weaker layer due to water seepage from the Mai Khola which flows directly in front of the site or from the mountain behind. To avoid this, work will be undertaken in the dry season.

CHAPTER 7

CHAPTER 7 PROJECT COST ESTIMATION

7.1 Cost Estimation Criteria

Construction cost estimation is made according to the following criteria:

1. Full consideration is to be made to the natural conditions prevailing at the site, construction scale, and levels of construction technology available in Nepal.

2. Construction cost encompasses access road, land acquisition, intake facilities, settling basin, headrace tunnel, head tank, penstock, power house, electromechanical equipment, contingency, temporary facilities, and engineering administrative cost.

3. Construction quantities are computed on the basis of basic design.

4. To the extent possible, construction equipment available in Nepal will be used.

5. Generating, transmission and substation equipment is to be procured either locally or from a 3rd country.

6. Exchange rate applied to calculation is that as of October 1993 (Nepal Central Bank rate: US$ 1 = Rs 48.25)

7. Engineering administrative fee is determined at 10% of total construction cost, including costs for detailed design, supplementary surveys, maintenance of executing agency's temporary facilities, and executing agency's administrative personnel and management costs.

8. Contingency is computed at 10% of access road and compensation cost, 15% of civil works cost and 10% of generating and transmission equipment cost.

7.2 Construction Unit Cost

Construction unit costs adopted are those taken from feasibility studies for similar power generating projects undertaken recently in Nepal.

Major such unit costs are as follows:

Table 7.2-1 Unit Cost

Item		Unit	Unit Cost (US$)	Remarks
Excavation				
	Earth	m^3	5.5	
	Rock	m^3	13.0	
Embanking				
	Stone rip rap	m^3	13.5	
	Earth	m^3	7.0	
Refill				
	Earth	m^3	4.5	
Excavation				
	Tunnel	m^3	90.0	
Concrete				
	Mass	m^3	160.0	
Reinforced concrete		m^3	270-310.0	
Concrete lining		m^3	310.0	
Masonry		m^3	75.0	
Gabion		m^3	20.0	
Steel				
	Penstock pipe	kg	3.5	
	Gate	kg	9.0	
	Shoring	kg	2.3	
	Screen	kg	2.7	
Access Road				
	New	km	90,000	gravel paving
	Improvement	km	36,000	widening

7.3 Land Acquisition and Compensation

Farmland and wasteland at the head tank, regulating pond, penstock route and access roads will require land acquisition. However, the intake site and power house site, due to their location in the river bed, will not be subject to this requirement.

It is considered under planning for the Project that such land acquisition for access road should be borne locally in view of the fact that despite the temporary nature of the roads from the standpoint of the Project, they will remain after construction completion as a means of thoroughfare for local residents.

Compensation would be considered to cover loss of timber, dwelling removal, costs for resettlement, compensation for lost farmland and land acquired for temporary facilities under the Project. However, loss of timber will not occur in the course of construction, and the number of households requiring resettlement is limited to 2~3.

Accordingly, land acquisition and compensation costs will be restricted to only the head tank, regulating pond and penstock route sites. The amount for this is estimated at a relatively small US$ 156,000.

7.4 Project Cost and Annual Project Outlay

7.4.1 Project Cost

Project cost is estimated as shown below.

1. Summary

Table 7.4-1 Project Cost Summary

(unit: US$ '000)

	Item	Cost	Remarks
1.	Road	390.0	new construction: 3.5 km; road widening: 1.6 km
2.	Compensation	156.0	
3.	Civil Works	6,634.8	
	Intake	333.1	
	Settling basin	454.0	
	Tunnel	3,623.8	
	Head tank	586.6	
	Penstock	1,261.3	
	Power house	327.6	
	Tailrace	48.4	
4.	Electrical facilities	3,540.0	
5.	Transmission line	94.0	
6.	Subtotal	10,814.8	
7.	Temporary facilities	1,081.5	10% of (6)
8.	Contingency	1,413.2	10% of (1,2,4,5) 15% of (3)
9.	Total	13,309.0	
10.	Engineering fee	1,331.0	approx. 10% of (9)
11.	Grand total	14,640.5	

2. Breakdown (Unit : 1.0 US$)

(1) Access Road

Item	Unit	Qty.	Unit cost	Cost
Intake site				
New construction	km	1	90,000	90,000
Road widening	km	1.6	36,000	57,600
Adit site				
New construction	km	1.5	90,000	135,000
Head tank site				
New construction	km	0.3	90,000	27,000
Power house site				
New construction	km	0.7	90,000	63,000
Others				17,400
Subtotal				390,000

(2) Intake Facilities

Item	Unit	Qty.	Unit cost	Cost
Excavation				
Rock	m^3	490	13.0	6,370
Concrete				
Mass concrete	m^3	1,090	160.0	174,400
Reinforced concrete	m^3	210	310.0	65,100
Masonry	m^3	--		
Embanking				
Stone riprap	m^3	540	13.5	7,290
Gate (2 nos.)	kg	1440	9	12,960
Screen	kg	1800	2.7	4,860
Stop log		--	--	
Coffer works	1 set			62,120
Subtotal				333,100

(3) Settling Basin

Item	Unit	Qty.	Unit cost	Cost
Excavation				
Tunnel	m^3	1,080	90.0	97,200
Concrete	m^3	25	230.0	5,750
Reinforced concrete				
Lining	m^3	280	310.0	86,800
Masonry	m^3	--	--	
Embanking				
Gate (1 nos.)	kg	360	9	3,240
Steel	kg	5,940	2.3	13,662
Stop log	set	1	--	3,600
Subtotal				210,252

(4) Sand Flush (L = 170.0 m + inspection tunnel L = 36 m)

Item	Unit	Qty.	Unit cost	Cost
Excavation				
Rock slope	m^3	30	13.0	390
Tunnel	m^3	940	90.0	84,600
Concrete				
Tunnel opening	m^3	25	230.0	5,750
Lining	m^3	370	310	114,700
Masonry				--
Embanking				--
Gate				--
Steel	kg	13,000	2.3	29,900
Stop log	set	2	--	7,200
Others				1,208
Subtotal				243,748

Settling basin total: 454,000

(5) Tunnel (L = 17 m directly after diversion)

Item	Unit	Qty.	Unit cost	Cost
Excavation	m^3	190	90	17,100
Concrete lining	m^3	50	310	15,500
Concrete	m^3	--	--	--
Shot crete	m^3	--	--	--
Steel	kg	950	2.3	2,185
Gate	kg	4,500	9.0	40,500
Stop log	1 set	1		3,600
Subtotal				78,885

(6) Spillway (L = 137 m)

Item	Unit	Qty.	Unit cost	Cost
Cutting	m^3	--	--	
Excavation	m^3	450	90.0	40,500
Concrete				
Lining	m^3	190	310	58,900
Subtotal				99,400

(7) Tunnel (L = 3,200 + 34 m)

Item	Unit	Qty.	Unit cost	Cost
Excavation	m^3	16,970	90.0	1,527,300
Concrete				
Lining	m^3	4,500	310.0	1,395,000
Reinforced	m^3	1,000	270.0	270,000
Shotcrete			--	--
Rock bolt			--	--
Steel	kg	40,500	2.3	93,150
Others				
Subtotal				3,285,450

(8) Adit (L = 100 m)

Item	Unit	Qty.	Unit cost	Cost
Cutting	m^3	40	13.0	520
Excavation	m^3	550	90.0	49,500
Concrete				
Lining	m^3	200	310.0	62,000
Masonry	m^3	40	75.0	3,000
Steel	kg	9,700	2.3	22,310
Others (tunnel portion)				22,735
Subtotal				160,065

Tunnel total: 78,885 + 99,400 + 3,285,450 + 160,065 = 3,623,800

(9) Head Tank

Item	Unit	Qty.	Unit cost	Cost
Cutting	m^3	840	5.5	4,620
Excavation	m^3	330	13.0	4,290
Concrete	m^3	250	230.0	57,500
Reinforced concrete	m^3	530	270.0	143,100
Masonry	m^3	180	75.0	13,500
Refill	m^3	420	4.5	1,890
Gate	kg	720	9.0	6,480
Screen	kg	4,500	2.7	12,150
Valve	set			9,000
Stop log	set			3,600
Subtotal				256,130

(10) Regulating Pond

Item	Unit	Qty.	Unit cost	Cost
Cutting				
Earth	m^3	1,730	5.5	9,515
Excavation				
Stone	m^3	340	13.0	4,420
Concrete (base)	m^3	460	230.0	105,800
Reinforced concrete (wall)	m^3	610	270.0	164,700
Masonry (slope base)	m^3	180	75.0	13,500
Embanking	m^3	760	7.0	5,320
Refill	m^3	550	4.5	2,475
Gate (1 nos.)	kg	1,800	9.0	16,200
Stop log	set			7,200
Others				1,340
Subtotal				330,470

Head tank total: 586,600

(11) Penstock

Item	Unit	Qty.	Unit cost	Cost
Cutting				
Earth	m^3	1,990	5.5	10,945
Excavation				
Stone	m^3	2,120	13.0	27,560
Concrete	m^3	1,190	230.0	273,700
Reinforced concrete	m^3	300	270.0	81,000
Masonry (saddle)	m^3	180	75.0	13,500
Refill (including 890 m3 of embanking)	m^3	2,970	4.5	13,365
Steel pipe	kg	240,000	3.5	840,000
Others				1,230
Subtotal				1,261,300

(12) Power House

Item	Unit	Qty.	Unit cost	Cost
Cutting				
Earth	m^3	510	5.5	2,805
Excavation				
Stone	m^3	1,170	13.5	15,795
Concrete	m^3	145	230.0	33,350
Reinforced concrete	m^3	830	270.0	224,100
Masonry (slope base)	m^3	90	75.0	6,750
Embanking	m^3	1,980	7.0	13,860
Refill	m^3	1,260	4.5	5,670
Land preparation				25,270
(including stumping)				
Subtotal				327,600

(13) Tailrace

Item	Unit	Qty.	Unit cost	Cost
Cutting				
Earth	m^3	550	5.5	3,025
Concrete	m^3	45	230.0	10,350
Reinforced concrete				
Masonry (canal)	m^3	250	75.0	18,750
Embanking	m^3	200	7.0	1,400
Gabion	m^3	90	20.0	1,800
Stop log				
Others				13,075
Subtotal				48,400

7.4-2 Annual Project Outlay

Annual Project outlay is as shown in the following table.

Table 7.4.2 Annual Project Outlay

(unit: US$ '000)

	1st year	2nd year	3rd year	4th year	Total
Road / bridge	280.0	55.0	55.0	--	390.0
Land acquisition / compensation	110.0	--	46.0	--	156.0
Civil works	1,660.0	2,980.0	1,994.8	--	6,634.8
Electrical / transmission works	1,180.0	1,180.0	1,274.0	--	3,634.0
Temporary facilities	973.0	--	108.5	--	1,081.5
Contingency	350.0	630.0	433.2	--	1,413.2
Engineering fee	671.0	330.0	330.0	--	1,331.0
Total	5,224.0	5,175.0	4,241.5	--	14,640.5

7.4.3 Input-wise Project Cost

Input-wise Project cost is as shown in Table 7.4-3. This input-wise cost is applied to computation of foreign and local currency portions, as well as calculation of economic prices.

Table 7.4-3 Input-wise Annual Construction Cost

(Unit : US$)

	% of Foreign Currency Portion	1st Year	2nd Year	3rd Year	4th Year	Total	Foreign Currency Portion	Local Currency Portion
a. Labor								
unskilled	0	--	97,582	131,820	103,983	333,385	--	333,385
skilled (local)	0	--	243,065	328,349	259,009	830,423	--	830,423
skilled (expatriate)	100	--	72,531	97,980	77,289	247,800	247,800	--
b. Fuel	80	--	235,338	317,911	250,775	804,024	643,219	160,805
c. Construction machinery	75	--	661,374	893,432	704,758	2,259,564	1,694,674	564,890
d. Facilities	90	--	1,043,420	1,409,525	1,111,864	3,564,809	3,208,328	356,481
e. Construction materials								
cement	25	--	335,151	452,746	357,136	1,145,033	286,259	858,774
steel	70	--	151,055	204,056	160,964	516,075	361,253	154,822
f. Others	50	--	280,315	378,670	298,703	957,688	478,844	478,844
g. Temporary facilities	65	--	973,000	--	108,500	1,081,500	702,179	379,321
h. Compensation	0	--	110,000	--	46,000	156,000	--	156,000
i. Engineering fee	0	--	671,000	330,000	330,000	1,331,000	--	1,331,000
Subtotal		--						
Contingency		--	350,000	630,000	433,200	1,413,200	917,540	495,660
Grand total		--	5,223,831	5,174,489	4,242,181	14,640,501	8,540,096	6,100,405
Foreign currency portion		--	2,884,571	3,145,352	2,510,173	8,540,096		
Local currency portion		--	2,339,260	2,029,137	1,732,008	6,100,405		

CHAPTER 8

CHAPTER 8 EVALUATION OF THE PROJECT

8.1 Power Generation Cost

8.1.1 Hydropower Generation

The power generation cost of Ilam Small Hydropower Project consists of the initial capital cost (market price) of Rs 706.4 million (See Table 8.1-1) calculated in Chapter 7 and the operation and maintenance cost (O/M cost). The annual O/M cost is estimated at 1.5 % of the initial capital cost (Rs 10.60 million).

In the base case, as the construction term of the hydropower plant is 3 years and the operation term is 47 years, the total O/M cost is calculated at Rs 498.20 million (Rs 10.60 million/year × 47 years).

Table 8.1-1 Small Hydropower Plant Initial Capital Cost (Market Price)

(Unit : million Rs)

Item \ Year	Grand Total			1st Year	2nd Year	3rd Year
	Local	Foreign	Total			
1 Labour						
(1) Unskilled	16.1	----	16.1	4.7	6.4	5.0
(2) Skilled	40.1	12.0	52.1	15.2	20.6	16.3
2 Fuel Cost	7.8	31.0	38.8	11.4	15.4	12.0
3 Construction Equipment	27.2	81.8	109.0	31.9	43.1	34.0
4 Facilities	17.2	154.8	172.0	50.3	68.0	53.7
5 Materials						
(1) Cement	41.4	13.8	55.2	16.2	21.8	17.2
(2) Steel	7.5	17.4	24.9	7.3	9.8	7.8
(3) Others	23.1	23.1	46.2	13.5	18.3	14.4
6 Provisional Facilites	18.3	33.9	52.2	47.0	—	5.2
7 Land Compensation	7.5	—	7.5	5.3	—	2.2
8 Engineering fee	64.2	—	64.2	32.4	15.9	15.9
9 Contingency	23.9	44.3	68.2	16.9	30.4	20.9
Grand Total	294.3	412.1	706.4	252.1	249.7	204.6

(US$ 1 = Rs 48.25)

8.1.2 Diesel Electric Power Generation

(1) For calculation of benefit under economic analysis, the "alternative facilities method" is used. This method is conventional in the economic analysis of hydropower development projects. Under the method, the construction cost and O/M cost of least cost alternative facilities to the envisioned hydropower development with identical capacity is considered as the cost that would be saved if such alternative were not adopted. Therefore, the saved cost, namely, the total cost of the alternative is regarded as benefit under the Project. Accordingly, the internal rate of return calculated by this method is called the alternative economic internal rate of return.

(2) With regards to types of electric power facilities in Nepal, the prominent facilities are limited to hydropower facilities and diesel electric ones. The reasons for the above are that resources of coal, oil and natural gas have not been discovered and developed in the country as of yet, as well as the fact that transportation network for the conveyance of such fuels has not been developed either. Accordingly, a diesel electric power plant (with 3,100 kW × 2 units) is adopted as the alternative facility for this analysis.

As there has not been a recent example in Nepal of construction of a diesel electric power plant with the above capacity, the initial capital cost of such a diesel power plant was calculated on the basis of actual cases for diesel power plants in other developing countries. Results are as shown in Table 8.1.2-1, in which the initial capital cost is Rs 272.7 million (market price) and the construction term is 2 years.

(3) Initial capital cost of accounting price calculated on the basis of Table 8.1-2 is Rs 244.7 million (see Table 8.3-2).

Next, the replacement cost and O/M cost of the diesel power plant in the accounting price is identified. Assuming a project life of the diesel power plant as 20 years, the plant must be replaced every 20 years. Referring to past examples, the replacement cost is estimated at 80 % the initial capital cost (Rs 195.76 million).

The annual O/M cost of the diesel power plant is computed at Rs 12.03 million which is 1.5 % (Rs 3.67 million) of the initial capital cost plus the annual lubrication oil cost (Rs 8.63 million). Accordingly, the total O/M cost in the project cycle is calculated at Rs 565.41 million (Rs 12.03 million/year × 46 years). Then, the annual fuel cost, namely, the annual diesel oil cost is calculated at Rs 100.67 million (Rs 2.30 million/kWh × 43,771,000 kWh/year). Accordingly, the total fuel cost in the project cycle is calculated at Rs 4,731.49 million (Rs 100.67 million/year × 47 years). In addition, imported lubrication oil and fuel oil will be exclusively used. Accordingly, CIF border prices are applied to the prices for both of these in this case.