8. HYDROPOWER PLAN

8.1 GENERAL

Dam development with FSL.320 m was recommended in the Phase I Study as the most attractive alternative that will clear the required economic index for IPP project, as well as avoid inundation of most of the Thaviang Sub-District. However, premises adopted in that stage were uncertain in some key issues such as reservoir capacity volume, run-off in reservoir, and prospect of power export to surrounding countries.

Accordingly, re-examination of these issues has been emphatically carried out during this Phase II Study. With newly obtained information at this stage, confirmation of the appropriateness of the optimum development plan proposed in the Phase I Study was carried out.

8.2 RESERVOIR OPERATION

8.2.1 CONDITIONS FOR RESERVOIR OPERATION STUDY

In order to select the optimal development scale of the Nam Ngiep-I HEPP, a reservoir operation analysis was undertaken by establishing a simulation model, then power output and annual energy production were calculated for the selected alternative scales.

Parameters which were put into the simulation model as fixed conditions were reservoir capacity curve, tailwater level, inflow at dam site, evaporation, hydropower plant characteristics and peak operation hour.

(1) Reservoir Capacity Curve

The reservoir area and capacity curves were derived from the topographic maps at the scale of 1:10,000 that were prepared in this Phase II Study, as shown in Figure 8.2.1. Reservoir area and storage capacity for EL.320 m were estimated at 66.94 km^2 and 2,241.2 mil.m^3, respectively.

The obtained storage capacity was compared with the Phase I Study one as shown in Table 8.2.1. It was revealed that the difference was relatively small.

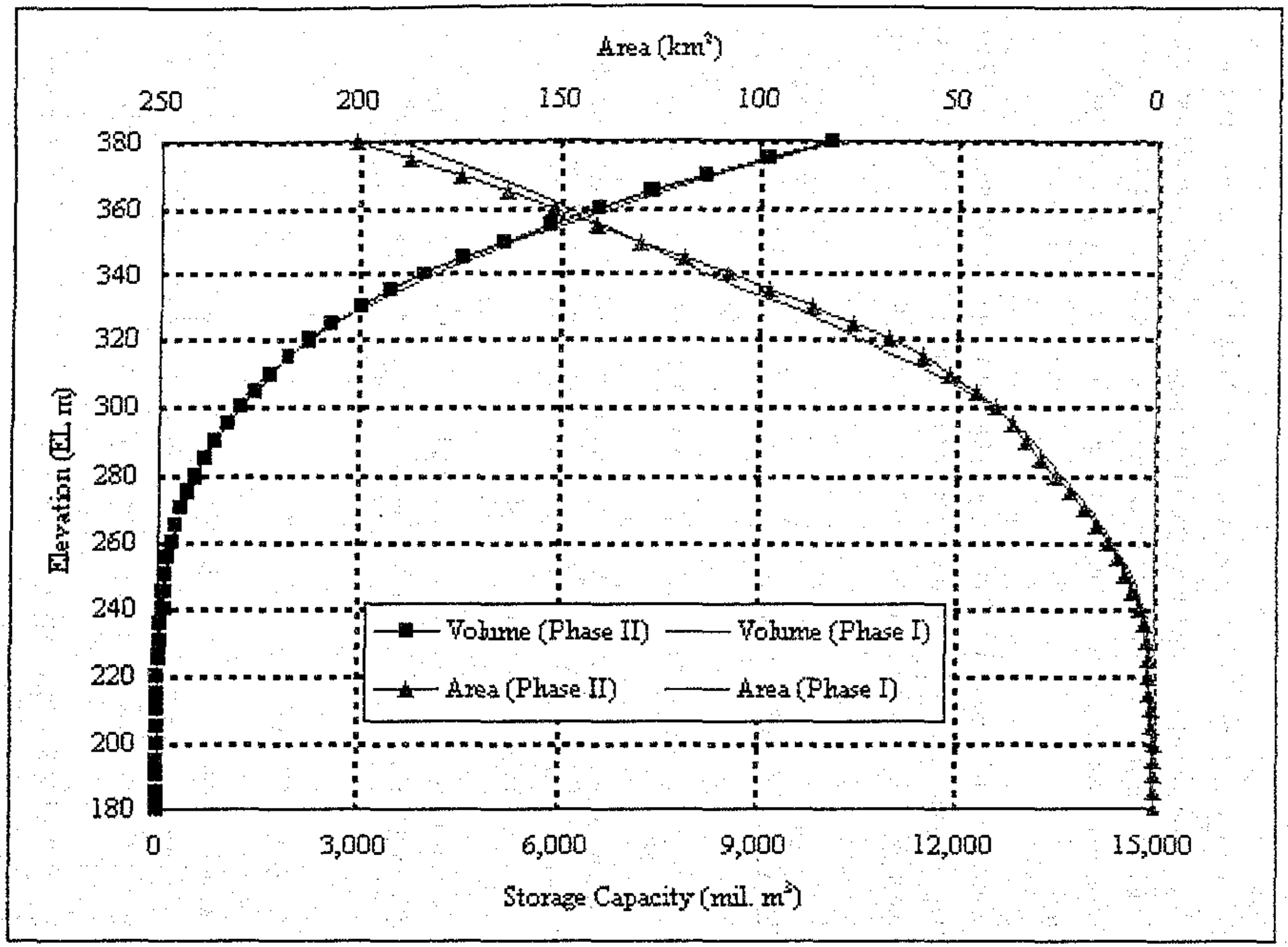


Figure 8.2.1 Reservoir Capacity Curve

EL. (m)	Area (km2)	Volume (mil. m3)
180	0.00	0.0
185	0.01	0.0
190	0.04	0.2
195	0.08	0.5
200	0.25	1.3
205	0.51	3.2
210	0.71	6.3
215	0.93	10.4
220	1.15	15.6
225	1.38	21.9
230	1.68	29.6
235	2.68	40.5
240	3.84	56.8
245	5.23	79.4
250	7.13	110.3
255	8.95	150.6
260	11.34	201.3
265	14.32	265.5
270	17.67	345.4
275	21.35	443.0
280	24.89	558.6
285	28.25	691.4
290	32.01	842.1
295	35.79	1,011.5
300	40.08	1,201.2
305	44.98	1,413.9
310	51.30	1,654.6
315	58.22	1,928.3
320	66.94	2,241.2
325	75.92	2,598.4
330	86.61	3,004.7
335	97.84	3,465.8
340	108.30	3,981.2
345	118.73	4,548.8
350	129.70	5,169.8
355	140.48	5,845.3
360	151.48	6,575.2
365	162.93	7,361.2
370	174.78	8,205.5
375	187.51	9,111.2
380	200.90	10,082.2

Table 8.2.1 Comparison of Storage Capacity

EL. (m)	Reservoir Capacity (mil. m3)		Ratio (%)
	Phase II	Phase I	
300	1,201.2	1,135.4	106%
320	2,241.2	2,279.4	98%
340	3,981.2	4,158.9	96%
360	6,575.2	6,781.5	97%

(2) Tailwater Level

Energy gradient profiles at the downstream of the powerhouse were prepared, based on the results of non-uniform flow computation for the section between the tailrace of the Nam Ngiep-I power plant, and the stream gauging station at B.Hatkham. The river cross sections surveyed in this study were used for the non-uniform flow computations.

Tailwater level for the energy calculation was based on this energy gradient, as the river flow is rapid at the dam site. It was estimated around EL.181 m for the plant discharges of respective alternative plant scales, and that was about 6 m higher than the value applied in the Phase-I Study (EL.175 m). This difference was mainly due to revision of the original riverbed elevations at downstream reach of the dam based on the topographic survey.

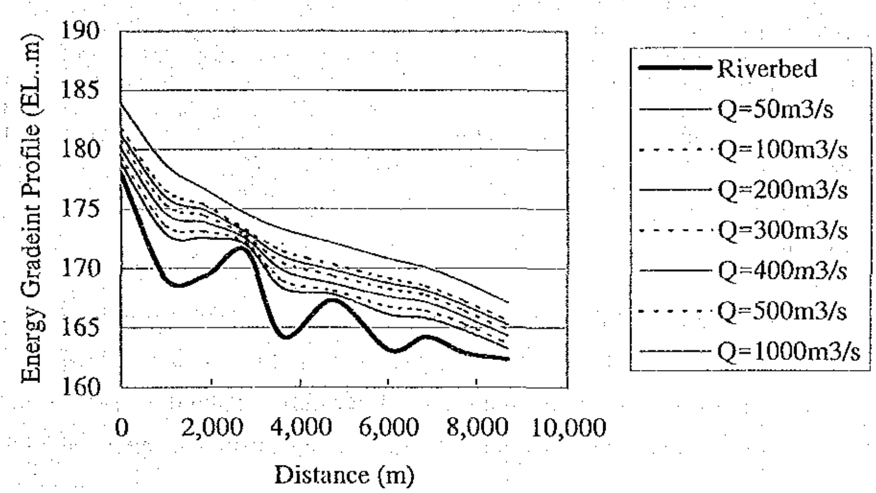


Figure 8.2.2 Energy Gradient of Nam Ngiep River Downstream of Powerhouse

(3) Inflow at Dam Site

The long-term sequence of inflow was estimated through hydrological analysis in this Phase II Study as detailed in Chapter 5 of this report. The estimated monthly inflow covers the 30 years from 1971 to 2000, and its annual average is 147.2 m^3/s. This is about 91% of the value at 162.3 m^3/s which had been applied in the Phase I Study.

Table 8.2.2 Inflow at the Dam Site

Site: Proposed Dam Site Unit: m^3/sec

Year	Month	1 Jan.	2 Feb.	3 Mar.	4 Apr.	5 May	6 Jun.	7 Jul.	8 Aug.	9 Sep.	10 Oct.	11 Nov.	12 Dec.	Annual Mean
1	1971	81.7	70.8	93.7	73.4	131.6	233.3	412.3	263.0	214.3	133.0	107.1	88.3	158.5
2	1972	74.0	60.8	53.3	73.1	91.1	242.0	231.6	279.5	143.0	118.0	86.8	71.6	127.1
3	1973	59.1	50.9	40.3	34.4	80.3	120.5	139.7	208.2	272.0	103.8	80.4	65.4	104.6
4	1974	55.5	46.9	37.2	36.5	55.7	92.9	159.6	243.1	197.8	84.8	70.4	58.6	94.9
5	1975	49.9	39.7	33.7	32.2	107.1	299.8	157.6	192.4	226.0	168.6	95.3	80.2	123.5
6	1976	65.0	72.4	48.4	44.4	68.6	150.8	175.2	316.1	230.3	184.8	106.4	86.0	129.0
7	1977	71.7	59.6	51.9	46.1	51.8	108.6	298.5	163.6	215.0	91.9	77.0	63.0	108.2
8	1978	53.7	43.9	39.0	67.9	40.8	288.6	250.6	302.2	349.5	161.1	110.8	92.1	150.0
9	1979	77.4	63.9	55.4	50.3	169.8	172.3	202.0	189.8	130.5	85.0	68.7	57.3	110.2
10	1980	48.2	38.5	35.3	34.3	100.0	233.2	276.1	325.9	296.6	136.6	110.6	91.9	143.9
11	1981	77.0	63.0	54.3	66.8	114.3	186.8	332.4	273.7	293.5	210.5	115.7	96.4	157.0
12	1982	80.6	66.0	63.2	93.9	130.3	234.9	266.8	403.5	382.6	237.7	145.7	120.3	185.5
13	1983	99.9	103.5	86.6	83.9	122.6	164.8	329.2	354.7	308.2	209.1	142.5	111.6	176.4
14	1984	96.0	81.4	64.7	69.1	114.6	178.4	219.9	276.9	262.7	131.7	105.3	86.4	140.6
15	1985	71.6	59.6	51.7	57.9	275.0	296.2	327.1	250.5	241.3	139.2	111.9	95.2	164.8
16	1986	78.8	64.7	55.8	77.1	78.7	201.3	139.2	231.8	202.8	131.3	88.8	73.9	118.7
17	1987	60.5	52.5	41.7	36.3	64.5	211.1	264.3	317.9	320.4	200.3	116.4	96.4	148.5
18	1988	98.1	76.0	70.2	90.4	128.7	301.7	338.4	307.2	255.0	206.9	126.4	105.6	175.4
19	1989	86.7	73.1	70.3	110.6	114.2	268.7	214.7	288.1	233.9	164.2	109.6	91.2	152.1
20	1990	76.6	63.2	60.2	58.0	75.7	346.8	547.0	340.5	300.5	321.0	163.9	133.7	207.3
21	1991	110.5	92.5	78.9	92.1	98.1	190.4	278.1	311.3	245.5	145.4	108.7	89.7	153.4
22	1992	81.0	64.2	56.1	47.5	52.3	186.4	223.7	224.6	155.0	101.3	81.4	74.1	112.3
23	1993	57.3	48.6	38.8	46.5	97.9	232.2	414.1	294.3	198.6	149.7	108.5	89.7	148.0
24	1994	75.8	66.6	69.4	84.3	102.5	247.5	258.3	300.9	249.2	179.7	124.9	100.7	155.0
25	1995	83.4	69.5	58.4	64.4	128.5	210.8	381.5	464.4	306.8	159.6	128.7	107.1	180.3
26	1996	88.0	74.4	63.6	63.2	118.6	220.8	295.1	435.6	288.2	158.0	157.3	110.0	172.7
27	1997	91.4	77.3	67.8	131.3	167.2	190.8	342.6	347.1	337.7	182.0	136.3	112.9	182.0
28	1998	93.8	79.6	65.0	66.2	107.6	167.9	247.1	214.9	185.9	104.4	85.2	71.2	124.1
29	1999	59.0	50.8	43.2	43.8	227.7	251.8	237.4	331.0	292.6	158.0	117.8	98.2	159.3
30	2000	81.9	70.1	58.4	89.7	152.8	229.5	209.4	295.8	305.0	151.2	110.8	91.9	153.9
Average		76.1	64.8	56.9	65.5	112.3	215.4	272.3	291.6	254.7	157.0	110.0	90.4	147.2

Source: JICA Study Team (2002)

(4) Evaporation

The reservoir evaporation rate used for this simulation was assessed by multiplying 0.8 to the monthly pan evaporation records observed from 1971 to 2000 as detailed in Chapter 5 of this report. The estimated annual average of open water evaporation is 1,268 mm. Annual average of net evaporation is obtained at 557.8 mm by subtracting the annual average rainfall from the open water evaporation.

(5) Hydropower Plant Characteristics

Combined efficiency of the generator and turbine was assumed to be 0.88 for export plant in this study. The rated head was taken as 97% of gross head derived by subtracting tailwater level at the maximum plant discharge from a reservoir water surface level between the Full Supply Level (FSL) and the Minimum Operation Level (MOL), i.e. two-thirds of drawdown between FSL and MOL. Minimum plant discharge was set at 50% of the maximum plant discharge for a single unit turbine.

(6) Peak Operation Hour

As detailed in Chapter 7 in this report, if the Nam Ngiep-I power plant generates power at a plant factor lower than 50%, a combined cycle plant would be more economical and EGAT would rather buy power from the combined cycle IPPs. Therefore, operation as an intermediate power plant was considered in this study for the Nam Ngiep-1 power plant. As alternatives, reservoir operation analysis was conducted for both cases of 16-hour and 12-hour operation.

It was also assumed that EGAT would not purchase energy from the Project on Sundays and Thai national holidays, thus the power plant for export would cease for 60 days per year.

8.2.2 RESERVOIR OPERATION RULE

Water balance in the reservoir was computed applying the following equation:

$$S_i = S_{i-1} + I_i - O_i - EV_i$$

Where, S_i : Reservoir water volume in the current month

S_{i-1} : Reservoir water volume in the previous month

I_i : Inflow into reservoir in the current month

O_i : Outflow from reservoir in the current month

Ev_i : Evaporation from reservoir in the current month

Data inputs to the model such as inflow, evaporation and water required for power generation were determined on a monthly basis.

Reservoir operation rules applied in this study were as follows:

(i) Initial reservoir water level is set at FSL.

(ii) If the reservoir water level becomes higher than the rated water level, the plant runs at its full installed capacity for peak energy production.

(iii) If the reservoir becomes full, the plant runs continuously and produces off-peak energy.

(iv) If the reservoir drops below the rated water level, but remains above MOL, the plant runs at its highest achievable capacity with that water level for peak energy production.

(v) If the reservoir drops to MOL, only so much water is discharged that the reservoir water remains at this level.

Fixed conditions and variables that are considered in the developed simulation model are listed in Table 8.2.3. Flow chart of the simulation is as illustrated in Figure 8.2.3.

Table 8.2.3 List of Fixed Conditions and Variables for Simulation Model

Item	Symbol	Unit	Description
1 Fixed Condition			
1) Reservoir HAS Curve	f1		(function of reservoir water level)
2) Tailrace HQ Curve	f2		(function of tailwater level)
3) Monthly Inflow Discharge	Vin	mil.m3	= 147.2 m3/s in annual average
4) Monthly Evapaporation Depth	Evap	m	= 557.8 mm in annual total
5) Combined Efficiency for Turbine and Generator, IPP	$\eta1$		= 0.88 for IPP units
6) Combined Efficiency for Turbine and Generator, EDL	$\eta2$		= 0.84 forr EDL unit
7) Ratio of Minimum Plant Discharge	μ		= 50% of max. plant discharge for one unit
8) Ratio of Loss Head	β		= 3% of gross head
9) Acceleration of Gravity	g		= 9.8
10) Riverbed Level	RBL	EL.m	178m
2 Independent Variables for Alternative Schemes			
1) Full Supply Water Level	FSL	EL.m	
2) Minimum Operation Level	MOL	EL.m	(min head/max head >70%)
3) Maximum Peak Discharge (in total for IPP and EDL)	Qpmax	m3/s	
4) Ratio of Peak Discharge for EDL	α		= 0, 5, 10, or 20%
5) Peak Hours for IPP	PeakHipp	hours	= 16 or 12 hours
6) Peak Hours for EDL	PeakHedl	hours	= 24 hours
7) Monthly Operation Days for IPP	OpeDipp	days	(no operation for Sundays and holidays)
8) Monthly Operation Days for EDL	OpeDedl	days	(operation for all days)
9) Unit Number for IPP	UnitN	Nu.	= 2 units
3 Subordinate Variables			
1) Reservoir Water Level at the Beginning of the Month	WL1	EL.m	= FSL or WL2 of previous month
2) Reservoir Storage Volume at the Beginning of the Month	S1	mil.m3	= f1(WL1)
3) Reservoir Area at the Beginning of the Month	A1	mil.m3	= f1(WL1)
4) Monthly Evaporation Volume	Vevap	mil.m3	= A1 x Evap
5) Reservoir Storage Volume (step 1)	Se1	mil.m3	= S1+Vin-Vevap
6) Max. Peak Discharge for IPP	Qpmaxipp	m3/s	= (1-α) x Qpmax
7) Max. Peak Discharge for EDL	Qpmaxedl	m3/s	= α x Qpmax
8) Monthly Outflow for Max. Peak Discharge for IPP	Vpmaxipp	mil.m3	= Qpmaxipp x PeakHipp x OpeDipp x 3600
9) Monthly Outflow for Max. Peak Discharge for EDL	Vpmaxedl	mil.m3	= Qpmaxedl x PeakHedl x OpeDedl x 3600
10) Monthly Outflow for Max. Peak Discharge (in total)	Vpmax	mil.m3	= Vpmaxipp + Vpmaxedl
11) Monthly Outflow for Min. Peak Discharge (in total)	Vpmin	mil.m3	= (Vpmaxipp/UnitN + Vpmaxedl) x μ
12) Reservoir Storage Volume for FSL	Smax	mil.m3	= f1(FSL)
13) Reservoir Storage Volume for MOL	Smin	mil.m3	= f1(MOL)
14) Rated Water Level	RWL	EL.m	= (2xFSL+MOL)/3
15) Rated Head devided by Effective Head	Hn/H		
16) Monthly Outflow for Peak Discharge (in total)	Vpeak	mil.m3	= Vpmax x Hn/H, Vpmax x $(H/Hn)^{0.5}$, Se1-Smin, or 0
17) Reservoir Storage Volume (step 2)	Se2	mil.m3	= Se1-Vpeak
18) Max. Non-Peak Discharge for IPP	Qnpmaxipp	m3/s	= (1-α) x Qpmax
19) Max. Non-Peak Discharge for EDL	Qnpmaxedl	m3/s	= α x Qpmax
20) Monthly Outflow for Max. Non-Peak Discharge for IPP	Vnpmaxipp	mil.m3	= Qnpmaxipp x (24-PeakHipp) x OpeDipp x 3600
21) Monthly Outflow for Max. Non-Peak Discharge for EDL	Vnpmaxedl	mil.m3	= Qnpmaxedl x (24-PeakHedl) x OpeDedl x 3600
22) Monthly Outflow for Max. Non-Peak Discharge (in total)	Vnpmax	mil.m3	= Vnpmaxipp + Vnpmaxedl
23) Monthly Outflow for Min. Non-Peak Discharge (in total)	Vnpmin	mil.m3	= (Vnpmaxipp/UnitN + Vnpmaxedl) x μ
24) Monthly Outflow for Non-Peak Discharge (in total)	Vnpeak	mil.m3	= Vnpeak x Hn/H, Vnpmax x $(H/Hn)^{0.5}$, Se2-Smax, or 0
25) Reservoir Storage Volume (step 3)	Se3	mil.m3	= Se2-Vnpeak
26) Monthly Spillout	Vspil	mil.m3	= Se3-Smax, or 0
27) Reservoir Storage Volume (step 4)	Se4	mil.m3	= Se3-Vspil
28) Reservoir Water Level at the End of the Month	WL2	EL.m	= $f1^{-1}$(Se4)
29) Reservoir Water Level (monthly average)	WL	EL.m	= (WL1+WL2)/2
30) Peak Discharge	Qpeak	m3/s	= f(Vpeak)
31) Spillout Discharge	Qspil	m3/s	= f(Vspil)
32) Tailwater Level under Peak Discharge	TWLp	EL.m	= f2(Qpeak+Qspil)
33) Effective Head under Peak Discharge	Hepeak	m	= (WL-TWLp) x (1-β)
34) Peak Discharge for IPP	Qpeakipp	m3/s	= (1-α) x Qpeak
35) Peak Discharge for EDL	Qpeakedl	m3/s	= α x Qpeak
36) Peak Output for IPP	Ppeakipp	MW	= Hepeak x Qpeakipp x g x η
37) Peak Output for EDL	Ppeakedl	MW	= Hepeak x Qpeakedl x g x η
38) Peak Energy for IPP	Epeakipp	GWh	= Ppeakipp x PeakHipp x OpeDipp x 3600
39) Peak Energy for EDL	Epeakedl	GWh	= Ppeakedl x PeakHedl x OpeDedl x 3600
40) Non-Peak Discharge	Qnpeak	m3/s	= f(Vnpeak)
41) Tailwater Level under Non-Peak Discharge	TWLnp	EL.m	= f2(Qnpeak+Qspil)
42) Effective Head under Non-Peak Discharge	Henpeak	m	= (WL-TWLnp) x (1-β)
43) Non-Peak Discharge for IPP	Qnpeakipp	m3/s	= (1-α) x Qnpeak
44) Non-Peak Discharge for EDL	Qnpeakedl	m3/s	= α x Qnpeak
45) Non-Peak Output for IPP	Pnpeakipp	MW	= Henpeak x Qnpeakipp x g x η
46) Non-Peak Output for EDL	Pnpeakedl	MW	= Henpeak x Qnpeakedl x g x η
47) Non-Peak Energy for IPP	Enpeakipp	GWh	= Pnpeakipp x (24-PeakHipp) x OpeDipp x 3600 + Pnpeakipp x OffDipp x 24 x 3600
48) Non-Peak Energy for EDL	Enpeakedl	GWh	= Pnpeakedl x (24-PeakHedl) x OpeDedl x 3600 + Pnpeakedl x OffDedl x 24 x 3600

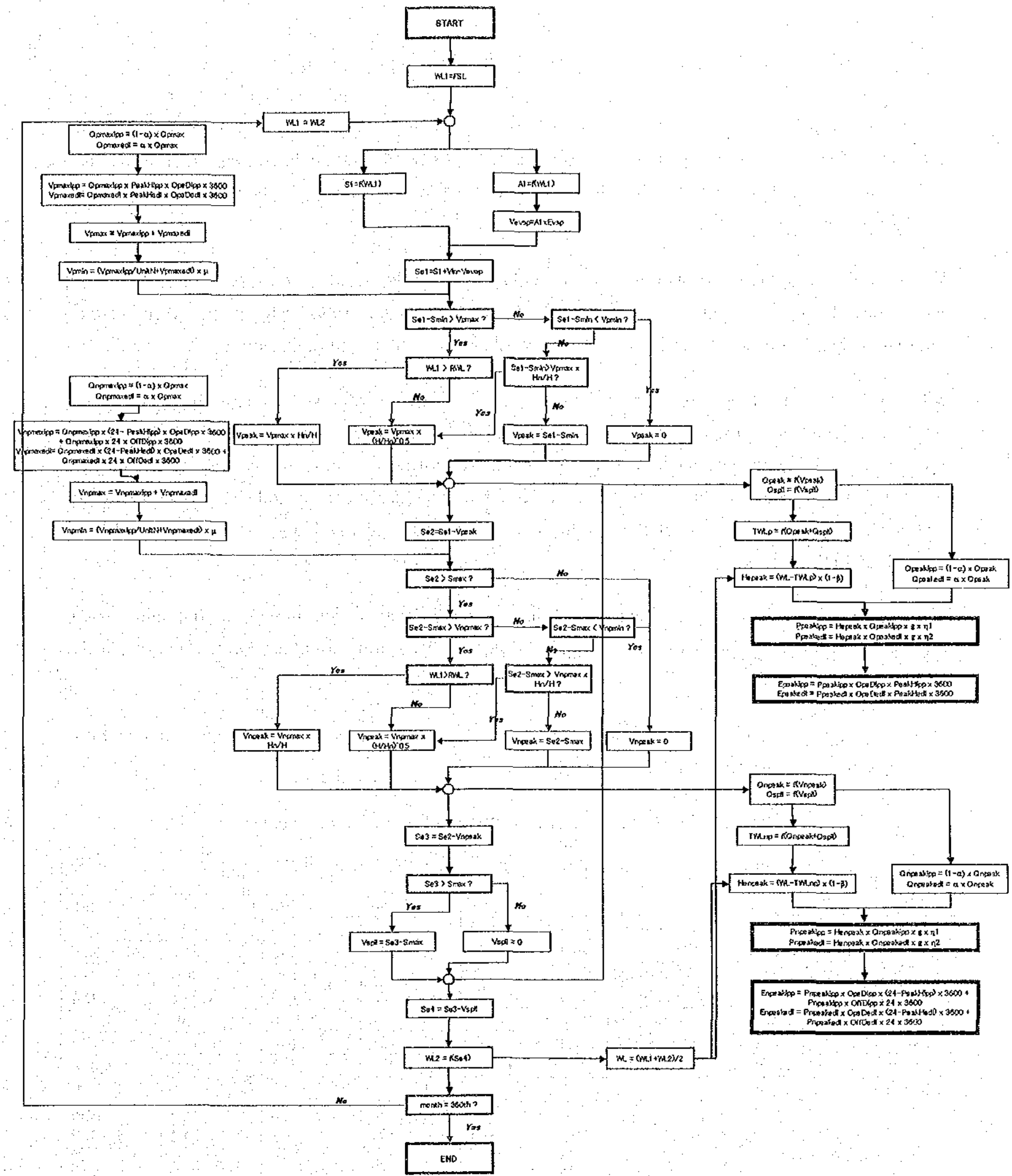


Figure 8.2.3 Flow Chart of Model Simulation

8.2.3 COMPUTATION OF POWER OUTPUT AND ENERGY

Under the fixed conditions and reservoir operation rules described above, development cases for alternative FSLs, MOLs and the maximum plant discharges were simulated and as a result, power and energy outputs were calculated with assumption that only power export would be expected to the Nam Ngiep-I power plant.

Based on the 1:10,000 scale map, most of the villages and paddy fields in the Thaviang Sub-District would be released from inundation if the FSL is set lower than around EL.320 m. Accordingly, alternative FSLs were set at 4 m intervals between EL.312 m and 328 m in this study.

MOL was set after taking into account the sediment level and enough water depth above the power intake sill, which is equivalent to about twice and half of the power tunnel diameter. The sediment level near the dam site, which corresponds to cumulative sediment volume in 100 years at 135 mil.m^3, was roughly assumed to be approximately EL.230 m by Area-Increment method. Moreover, the MOL was selected so that the ratio of the minimum water head to the maximum water head would not exceed the limit of 70%.

The maximum plant discharge for each development scheme was so determined that the annual peak energy for 95% dependability of the total analyzed period (30 years) should not fall below 80% of the annual average peak energy.

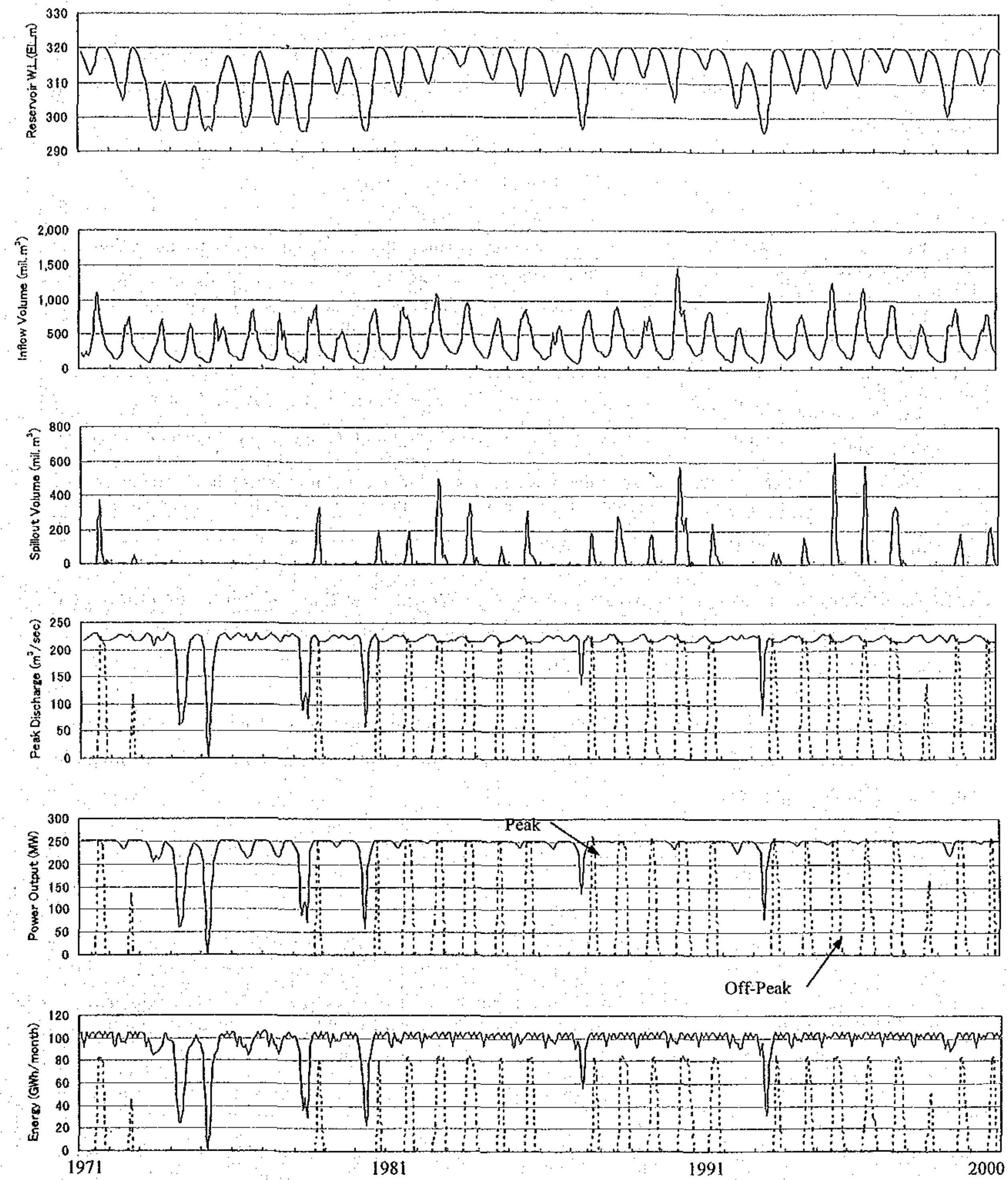
Annual energy production for respective development scheme in case of both 16-hour and 12-hour peak operation was estimated as shown in Table 8.2.4.

Table 8.2.4 Annual Energy Production for Alternative Scales

FSL	312m	316m	320m	324m	328m
16-hour Peak Operation					
MOL (EL.m)	284	288	296	292	296
Maximum Plant Discharge (m3/s)	230	230	230	238	242
Maximum Power Output (MW)	234	242	252	263	276
95% Dependable Peak Power Output (MW)	175	194	190	206	217
Annual Average Peak Energy (GWh)	1,082	1,126	1,173	1,227	1,284
95% Dependable Annual Peak Energy (GWh)	874	932	948	990	1,044
Annual Average Off-Peak Energy (GWh)	152	151	154	142	132
Annual Average Total Energy (GWh)	1,234	1,277	1,327	1,369	1,416
12-hour Peak Operation					
MOL	280	288	296	296	296
Maximum Plant Discharge (m3/s)	310	306	308	312	322
Maximum Power Output (MW)	312	322	338	349	367
95% Dependable Peak Power Output (MW)	237	258	252	282	289
Annual Average Peak Energy (GWh)	1,081	1,122	1,173	1,219	1,280
95% Dependable Annual Peak Energy (GWh)	874	934	944	988	1,047
Annual Average Off-Peak Energy (GWh)	203	213	213	204	186
Annual Average Total Energy (GWh)	1,284	1,335	1,386	1,423	1,466

Remarks; The above values were as estimated for the optimum MOL for respective FSLs. Maximum power output mentioned above is against average combined efficiency.

Simulation result for FSL.320 m (MOL.296 m) is represented in Figure 8.2.4. Annual average energy was estimated at 1,327 GWh per year, about 98% of the estimation in the Phase I Study (1,349 GWh).



Case: FSL320 m, MOL296 m, 2 units for power export purpose only

Figure 8.2.4 Reservoir Operation Simulation Result

8.3 RESERVOIR INUNDATION

8.3.1 LAND USE AT RESPECTIVE RESERVOIR ELEVATIONS

Based on the 1:10,000 scale maps that were prepared in this Phase II Study, latest conditions of land use at respective reservoir elevations were examined. Areas were classified into eight categories as shown below:

Table 8.3.1 Land Use Classification

No.	Item	Particular
1.	Urban area	Areas being used for permanent settlements such as villages, towns, public gardens etc.
2.	Rice paddy	Areas permanently being used for rice cultivation
3.	Agricultural land	Agricultural land being used for other than rice cultivation (except for grazing)
4.	Other cleared agriculture	Area for grazing, or unfertile or degraded land on which no trees or scrubs grow
5.	Mixed deciduous forest	Forest where the deciduous tree species represent more than 50% of the stand.
6.	Ray, scrub, forest regeneration	Ray: Areas where the forest has been cut and burnt for temporary cultivation of rice and other crops, Scrub: Areas which are covered with scrub and stunted trees, Forest regeneration: Previously forested areas in which the crown density has been reduced to less than 20% because of logging, shifting cultivation or other heavy disturbance
7.	Sand	Sand terrain
8.	Water	Rivers and ponds

Land cover maps for the whole reservoir area below EL.320 m, as well as EL.360 m, are represented in Figure 8.3.1. In addition, land cover in the Thaviang Sub-District only below EL.316 m, EL.320 m, El.324 m, and EL.328 m is also illustrated in Figure 8.3.2.

Land use classifications below respective elevations in the Thaviang Sub-District were calculated as shown in Figure 8.3.3. As seen, each classified area noticeably increases in case the elevation surpasses EL.320 m in general.

Elevations of villages in the Thaviang Sub-District were also checked on the same 1:10,000 scale maps.

Table 8.3.2 Elevations of Villages in Thaviang Sub-District

No.	Village	EL. (m)	No.	Village	EL. (m)
1	B. Pou	316	6	B. Phonyeng	326-330
2	B. Naphang	324	7	B. Naxong	330
3	B. Hatsamkone	326	8	B. Naxay	338
4	B. Phiangta	321	9	B. Viengthong	343
5	B. Dong	326-330			

Except B. Pou of which relocation in near future is scheduled by a national plan irrespective to the Project, the lowest village in elevation was B. Phiangta at EL.321 m.

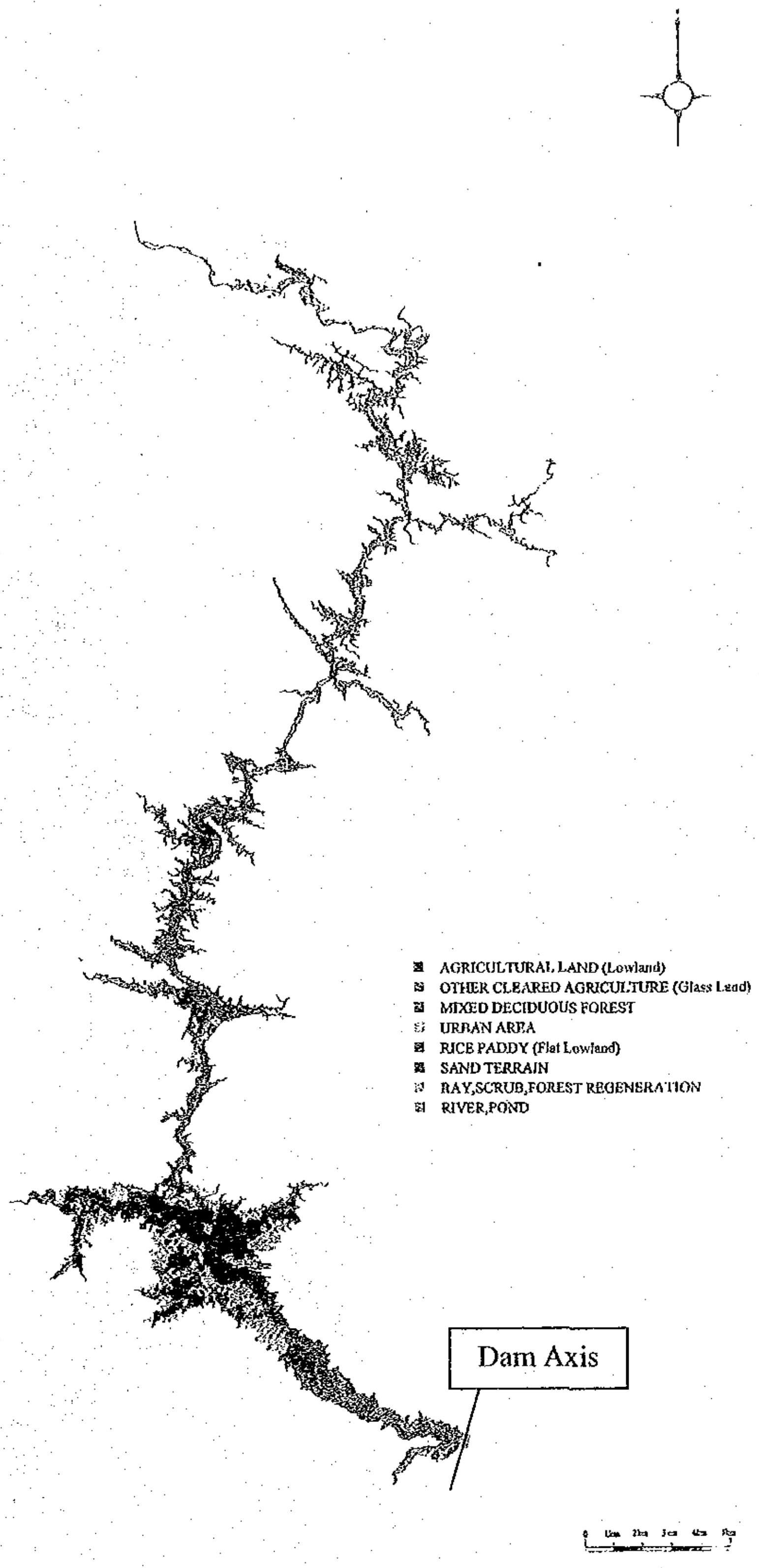


Figure 8.3.1 Land Use in Reservoir (1/2: EL.320 m)

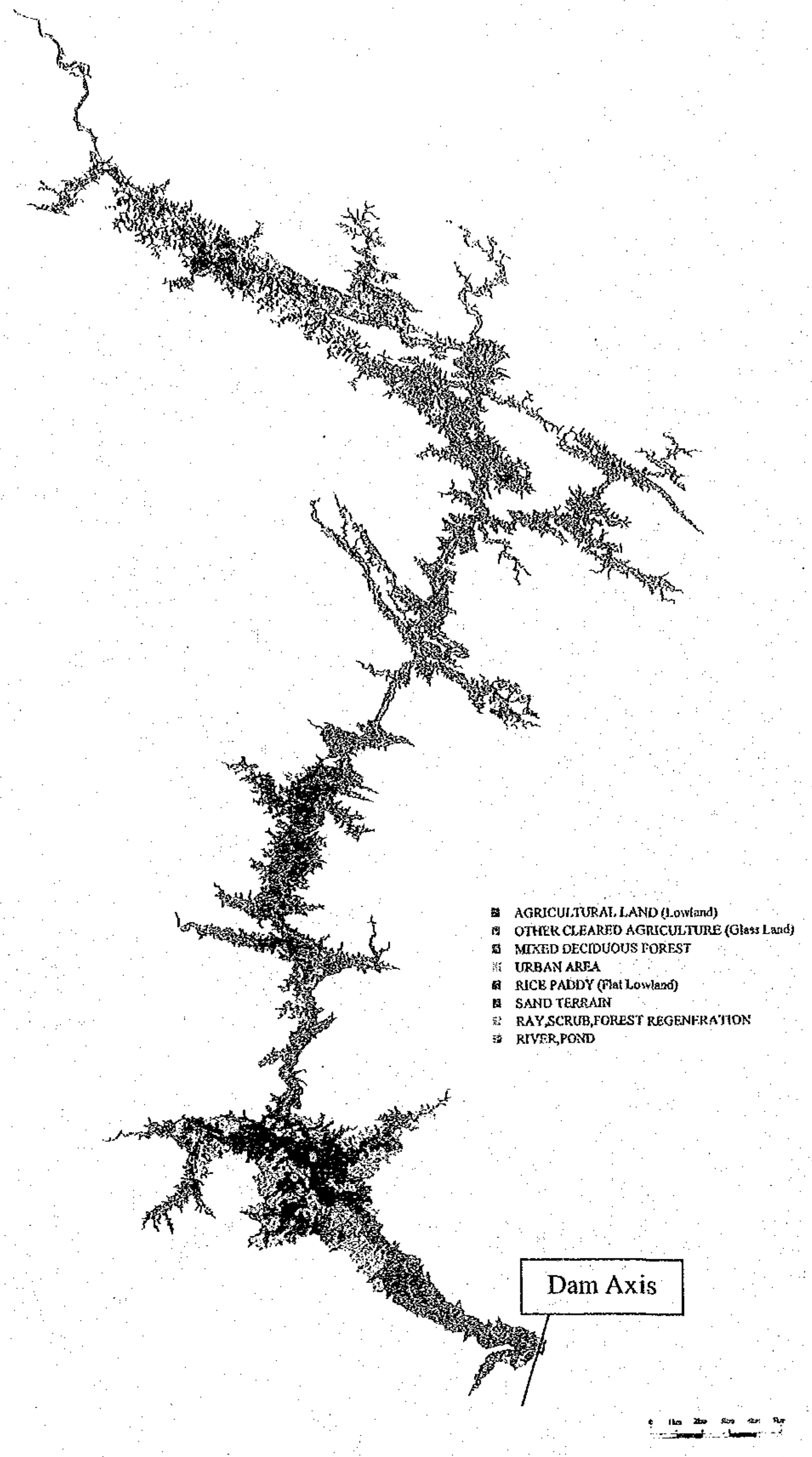


Figure 8.3.1 Land Use in Reservoir (2/2: EL.360 m)

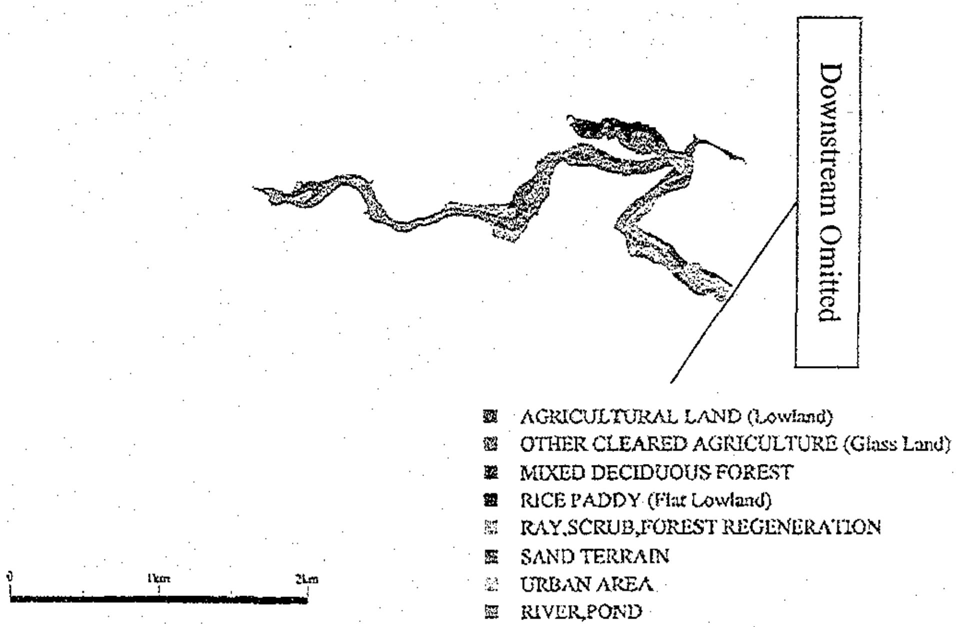


Figure 8.3.2 Land Use in Thaviang Sub-District (1/4: EL.316m)

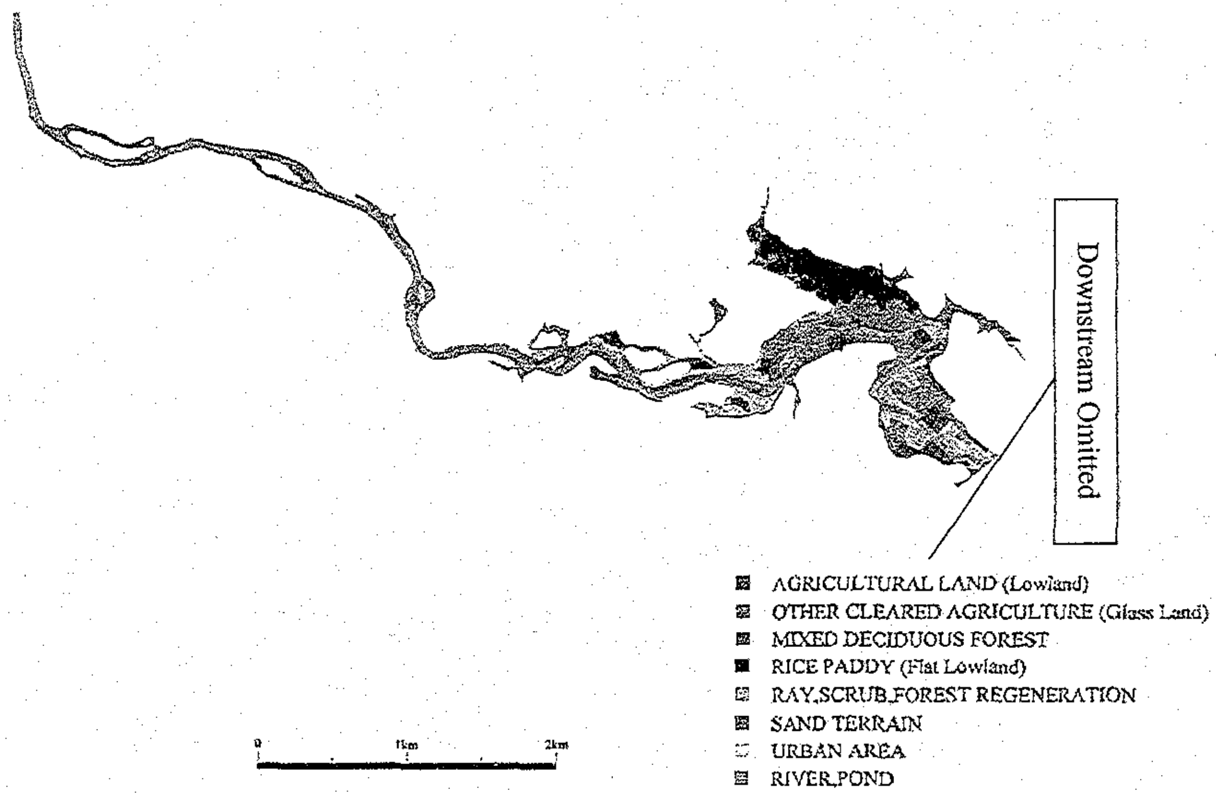


Figure 8.3.2 Land Use in Thaviang Sub-District (2/4: EL.320m)

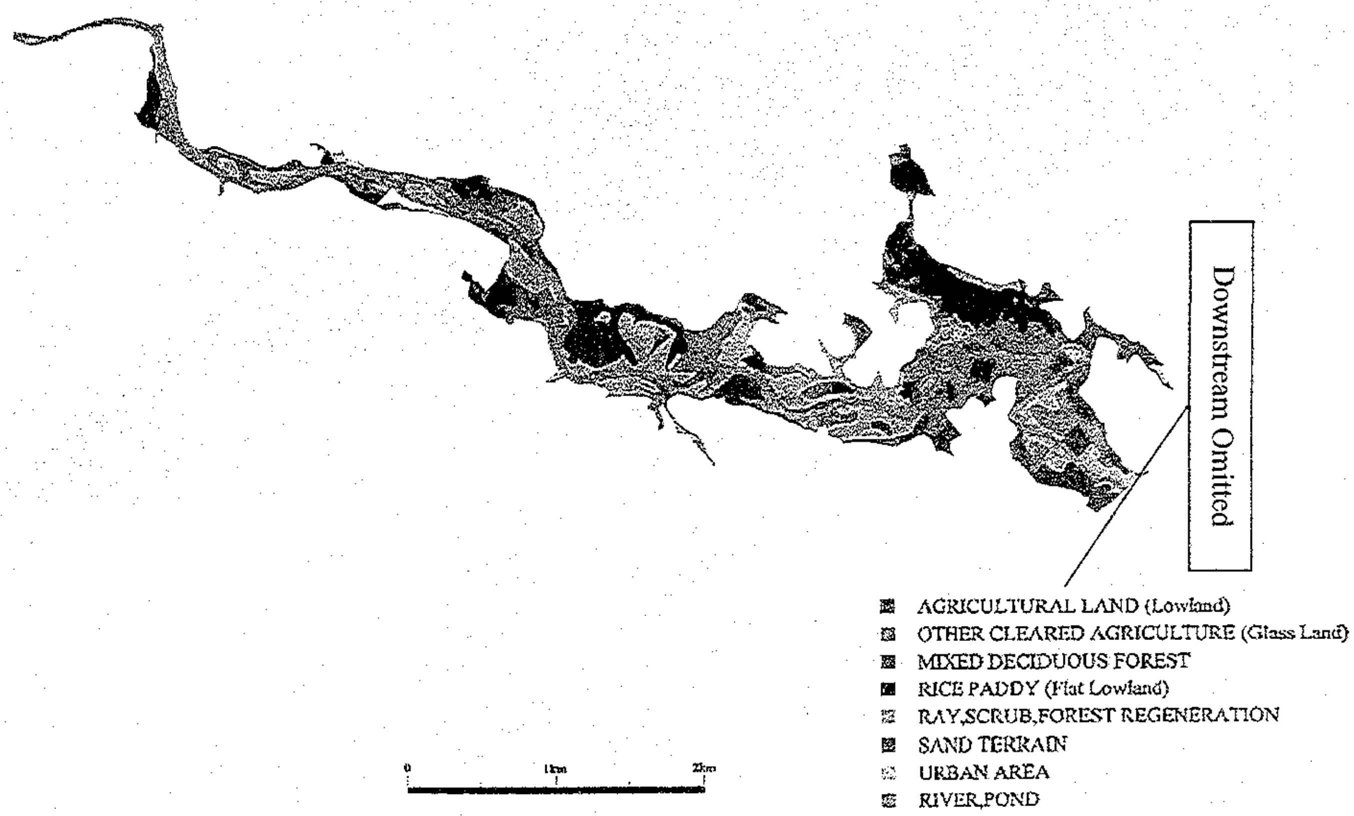


Figure 8.3.2 Land Use in Thaviang Sub-District (3/4: EL.324m)

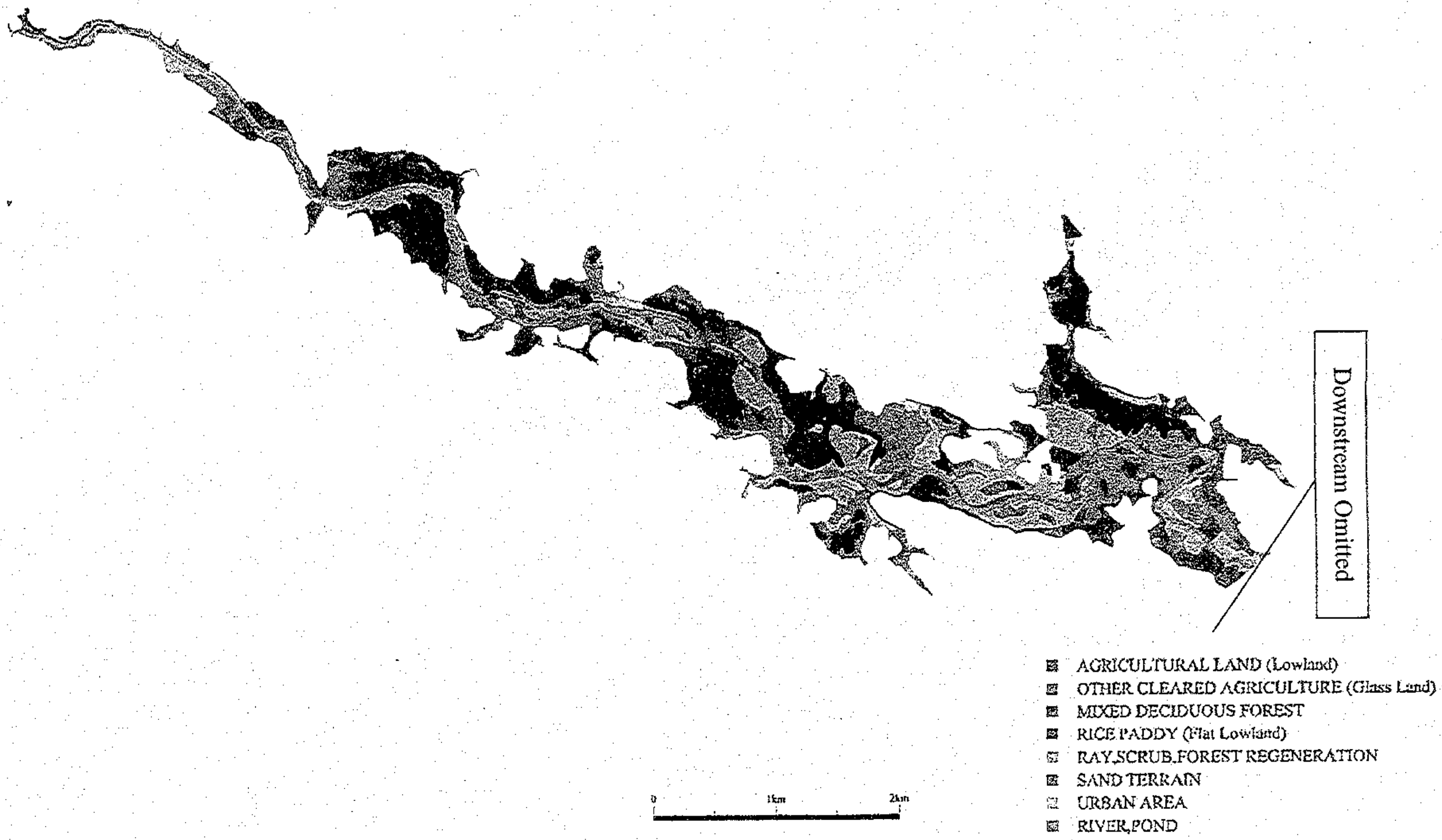


Figure 8.3.2 Land Use in Thaviang Sub-District (4/4: EL.328m)

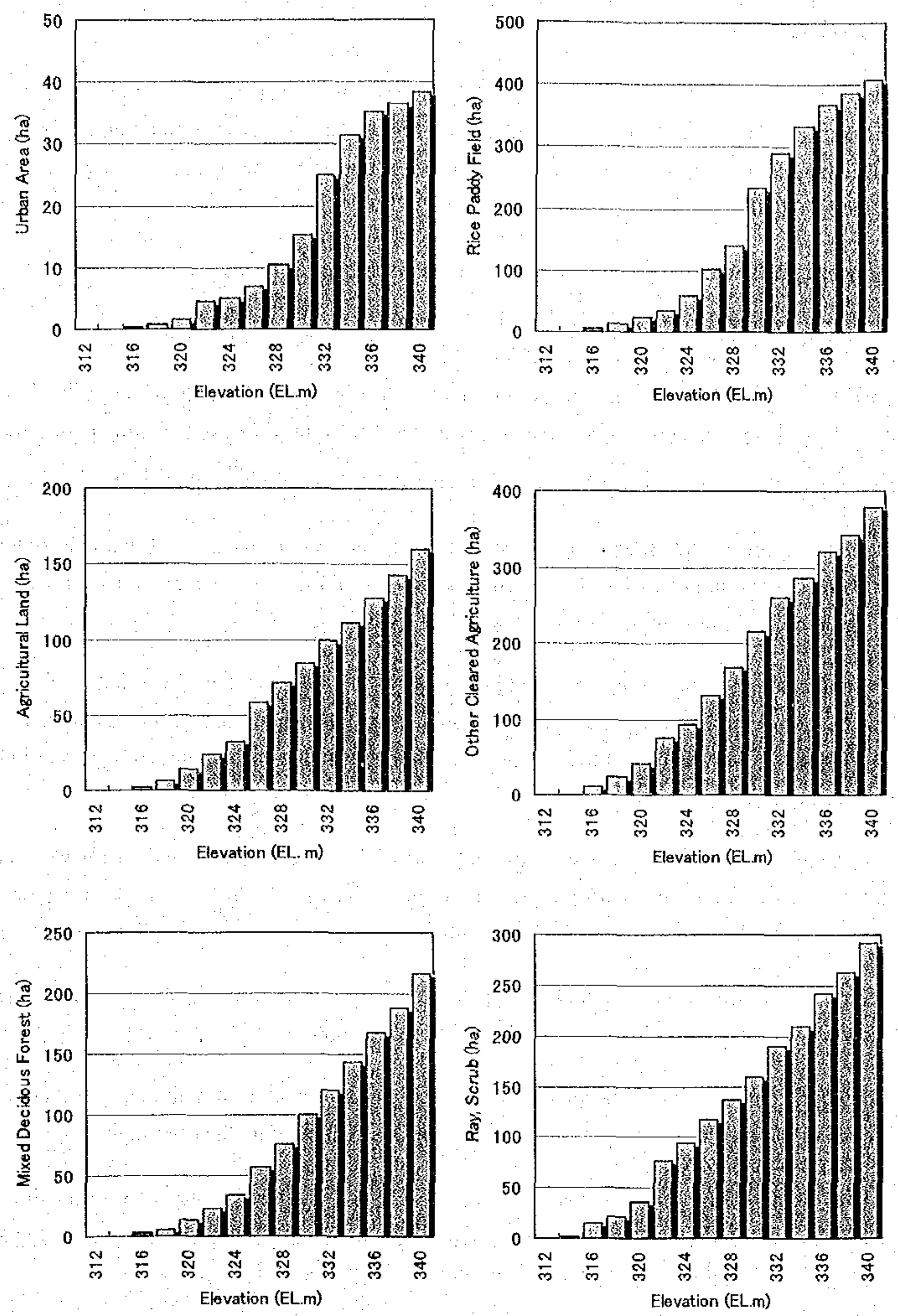


Figure 8.3.3 Classified Land Use below Respective Elevations

8.3.2 BACKWATER EFFECT

The backwater effect at the upstream reach of the reservoir was evaluated for the FSL.320 m case, based on the result of non-uniform flow computation for the stretch between the Nam Ngiep-1 dam and the Thaviang Sub-District. Cross sections for the non-uniform flow computations were obtained based on the 1:10,000 scale maps.

Water surface profiles were calculated for several flood discharges as follows:

Case 1: Before dam construction,
Case 2: After dam construction and no sediment,
Case 3: After dam construction and 5 m thick sediment below FSL.320 m, and
Case 4: After dam construction and removal of sediment upto EL.310 m along river course,

Based on result of the calculation as illustrated in Figure 8.3.4, the following observation were made:

(i) Water surface profiles at B.Dong and its upstream reaches do not change substantially before or after dam construction (Case1 and 2), even considering sediment (Case 3), therefore a backwater effect would not occur there due to reservoir water level.

(ii) For B.Phiangta in the case of no sediment and flood discharge at the dam site less than 2,000 m^3/s (Case 2), there would exist a backwater effect about 1m higher than FSL. If the discharge is more than 2,000 m^3/s, the water level before and after dam construction is similar, so there would exist no backwater effect.

(iii) For B.Phiangta in case of 5 m thick sediment below FSL.320 m (Case 3), the backwater effect about 2m high would occur. However, if the sediment above EL.310m is removed along the river course (Case 4), the water level would drop similarly as with the case of no sediment (Case 2).

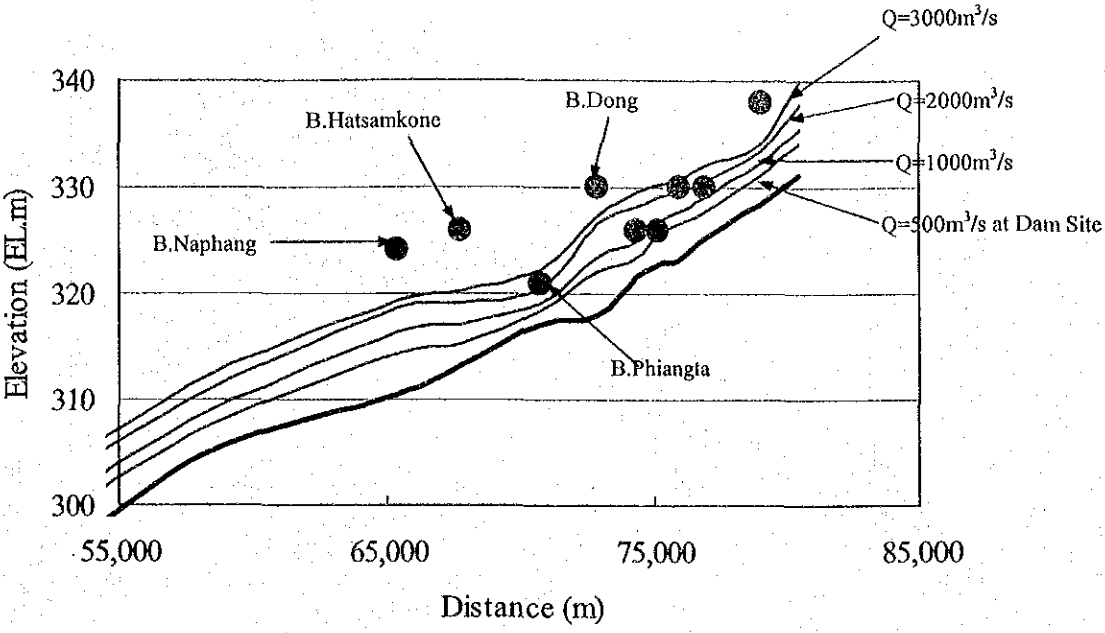
(iv) Based on rough estimation, 5 m thick sediment in reservoir corresponds to 50-year sedimentation. Meanwhile, removal of sediment above EL.310 m would amount to 2 mil. m^3.

(v) Serious flood damage, due to backwater effect in case of sediment accumulation, would not occur if the top of sedimentation in reservoir is kept below EL.310 m along the river course.

(vi) Dredging of 40,000 m^3 per year would maintain this figure, and even annual cost for the dredging is added to the O&M cost, that would not pull down the B/C ratio substantially. Thus the dredging option would be more economical than lowering FSL with the same effect.

(vii) For B.Naphang and B.Hatsamkone, both villages are above the water surface profiles after dam construction even considering the existence of sediment (Case 3).

Case 1: Without Dam



Case 2: FSL320 / Without Sediment

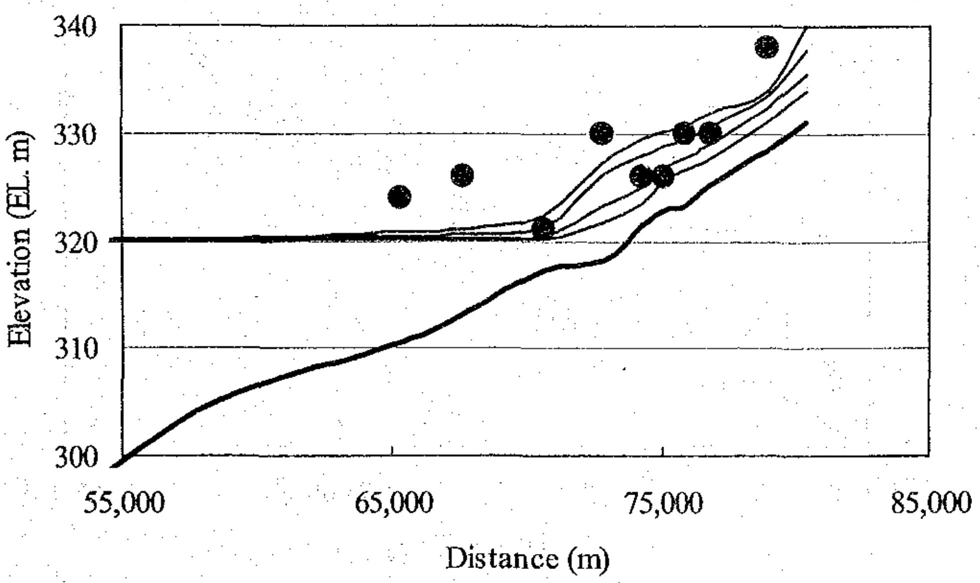
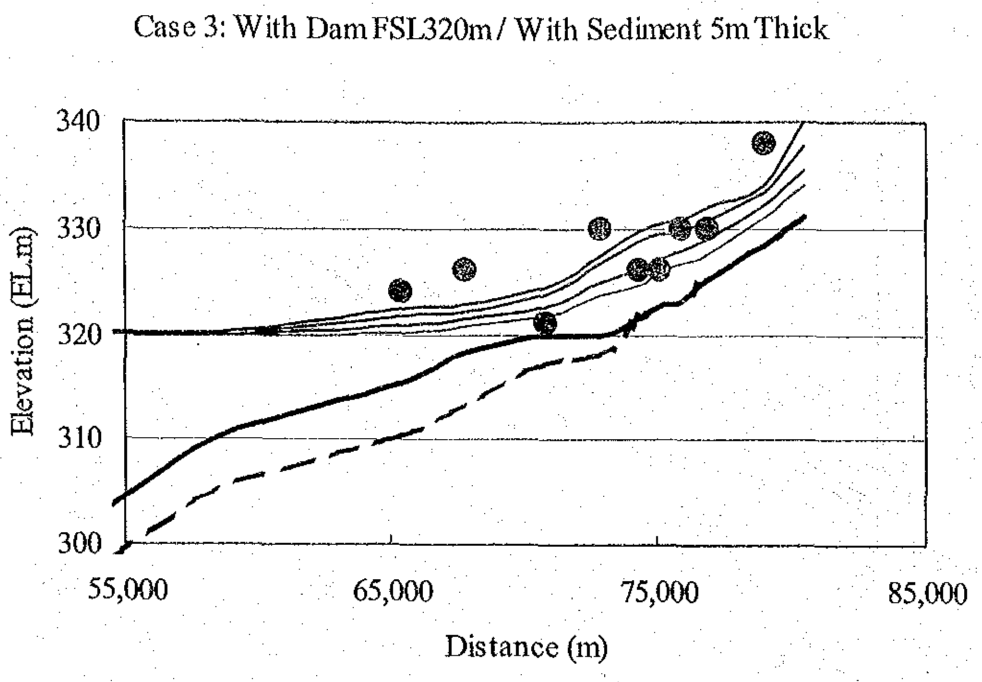


Figure 8.3.4 Backwater Effect (1/2)



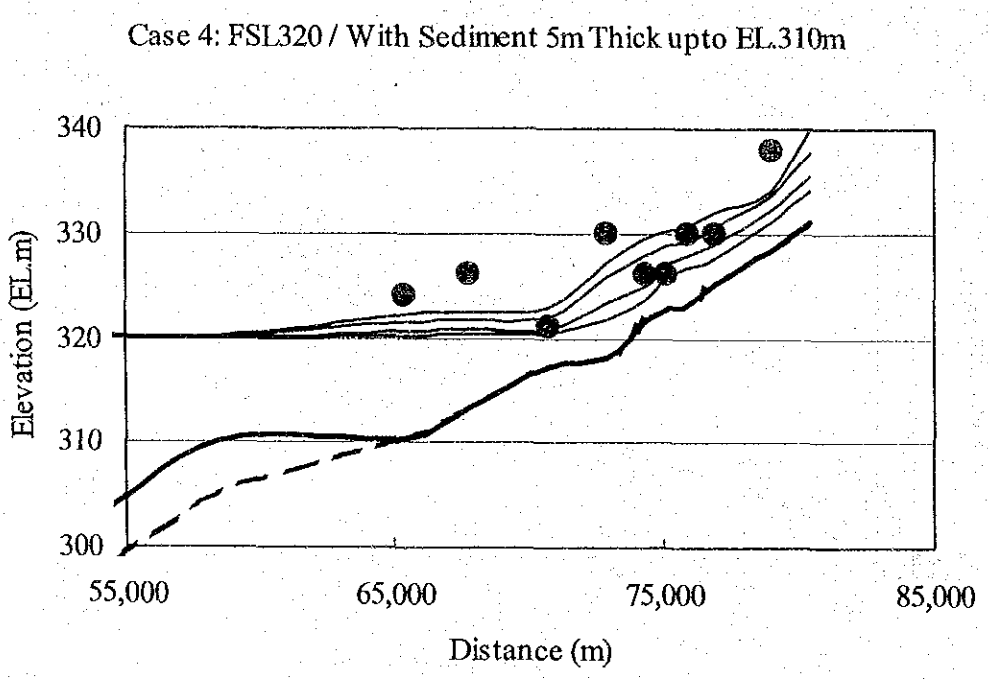


Figure 8.3.4 Backwater Effect (2/2)

8.4 OPTIMIZATION OF DEVELOPMENT SCALE

8.4.1 APPROACH TO OPTIMIZATION OF DEVELOPMENT SCALE

The criteria used in this study to decide the optimum development scale were:

(i) To achieve the highest benefit/cost (B/C) ratio

(ii) To avoid inundation of villages and paddy fields of the Thaviang Sub-District, except B. Pou of which relocation in the near future is scheduled because of an unrelated national plan.

Costs used for this B/C evaluation were estimated with unit prices assumed referring to recent international competitive bidding (ICB) data of similar project in Lao PDR and other Asian Countries. Work quantities for respective scale were computed by a simplified method. Annual O&M cost was assumed as 1 % of the investment cost.

Benefit was estimated assuming an average energy selling-rate at 5.4 USc/kWh (avoided cost for intermediate load by EGAT, 2001) for the peak energy production, discounted by the transmission line loss.

Discount rate of 10% was applied. No escalation for inflation on investment, O&M costs and energy selling rate were considered.

8.4.2 OPTIMUM DEVELOPMENT SCALE

The results of the B/C examination are presented in Figure 8.4.1. The 16-hour peak operation case brings higher B/C ratios by about 0.1 compared with the 12-hour peak operation case. As foreseen in the Phase I Study, the higher FSL brings the higher B/C. Meanwhile, in order to avoid inundation of villages (except B.Pou) in the Thaviang Sub-District, it is suggested the FSL be set below or equal to EL.320 m. The highest B/C under FSL.320 m is attained in case of MOL.296 m.

Thus, the operational characteristics resulting from the study for the optimum scheme are as follows:

Table 8.4.1 Optimum Development Scheme for Power Export

No.	Item	Optimum Scale
1.	FSL	EL.320m
2.	MOL	EL.296m
3.	Max. Plant Discharge	230 m^3/s
4.	Installed Capacity	260 MW
5.	Average Annual Peak Energy	1,173 GWh/year
6.	Average Annual Total Energy	1,327 GWh/year

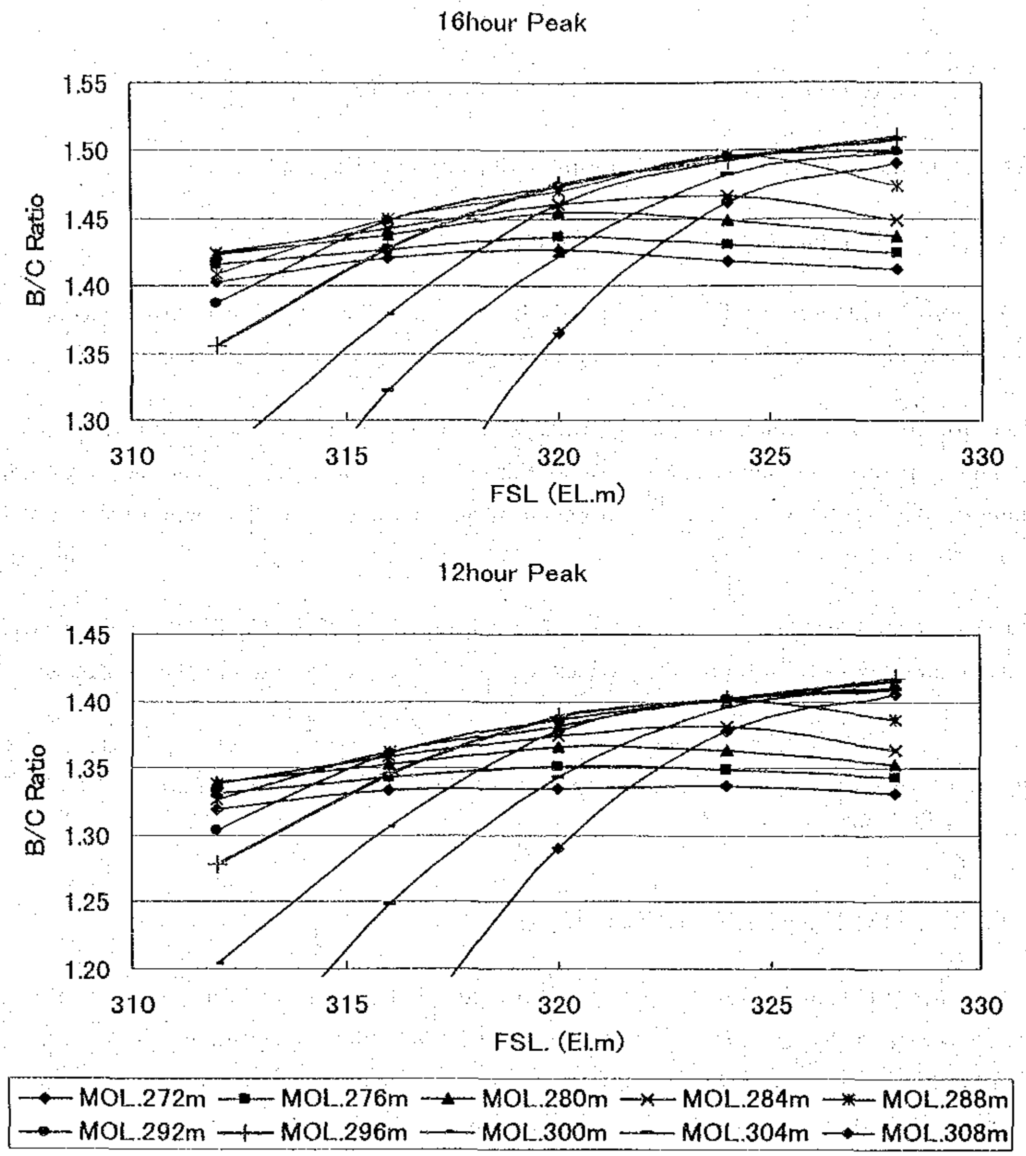


Figure 8.4.1 Benefit/Cost Ratio for Each Project Scale

8.5 POWER SUPPLY FOR DOMESTIC USE

8.5.1 GENERAL

According to the power policy in Laos, IPP plants for power export are expected to allocate some of their capacity (at least 5% of the output) for domestic use. For example, Theun Hinboun 210 MW and Houay Ho 150 MW have domestic off-take arrangements equivalent to 5% of output of the plants.

As there already exists a 22 kV transmission line in the vicinity along National Road 4 from Pakxan to Bolikham, the allocated power from the Project would be utilized by connecting to EDL's grid system, not for an isolated off-grid. For connecting to the EDL's grid system, another single line circuit of 115 kV power transmission line between the power ststaion and Paxsan substation will be constructed.

It is possible that if the Nabong collector substation is completed and ready for step down from 230 kV to 115 kV, such domestic off-take arrangement might be conducted at the said substation. However, as there exists some uncertainty on this matter regarding timing, providing an additional independent power unit is considered a safe option. Allocating power from the export units to the EDL's grid system, not by way of the Nabong substation, is technically difficult due to stability reason of the both EGAT's and EDL's grid systems.

The following shows the study results for the cases of providing independent unit at the main dam powerhouse as well as at the re-regulating weir. According to the result, B/C ratio decreases if such unit is provided at the main dam powerhouse, but the ratio slightly increases if the unit is equipped at the re-regulating weir. In case of the latter option, maximum capacity of the unit will be 16.8%, about 6% of the total capacity for the main dam powerhouse.

8.5.2 POWER PLANT AT MAIN DAM

The project economics were evaluated on the basis that the IPP provides additional plant at the main dam for domestic use, with the following conditions:

(i) The allocation of generating capacity of 5%, 10%, 15% and 20% to domestic use were analyzed in this study.

(ii) The maximum plant discharge (sum of both export and domestic use) for each case was so determined that the annual peak energy from the export units for 95% dependability of the total analyzed period should not fall below 80% of the annual average peak energy from the export units.

(iii) Benefit from the domestic unit was estimated based on 4.1 USc/kWh (avoided cost for base load by EGAT) discounted by the transmission line loss.

(iv) The domestic use plant would be accommodated in the same powerhouse building for the export units. A small penstock would branch off from one of penstocks for the export units.

(v) Reservoir operation analysis was carried out with the aforementioned simulation model under the condition that the FSL and MOL were set at EL.320 m and EL.296 m, respectively. The 16-hour peak operation was assumed for the power units for export, meanwhile the power unit for domestic use was assumed to take the base load and to be operated continuously for 24 hours throughout each year with no breaks. Combined efficiency of generator and turbine for the domestic plant was assumed to be 0.84.

Based on the above conditions, B/C examinations were carried out, and the results are shown on Table 8.5.1.

The highest B/C is attained for the case that no independent unit for domestic power supply would be provided. The B/C ratio decreases as the ratio of capacity for domestic power supply increases.

Table 8.5.1 Benefit/Cost Ratio for Alternative Domestic Power Supply

Ratio of Capacity for Domestic Use	0%	5%	10%	15%	20%
Annual Energy Production					
Maximum Plant Discharge in Total (m^3/s)	230	222	214	206	200
for Export	230	211	193	175	160
for Domestic Use	0	11	21	31	40
Maximum Power Output in Total (MW)	252	243	234	225	218
for Export	252	231	211	192	175
for Domestic Use	0	12	23	33	43
Annual Average Peak Energy in Total (GWh)	1,173	1,171	1,167	1,164	1,164
for Export	1,173	1,074	980	893	814
for Domestic Use	0	97	187	271	350
Annual Average Off-Peak. Energy in Total (GWh)	154	144	135	126	116
for Export	154	144	135	126	116
for Domestic Use	0	0	0	0	0
Annual Average Total Energy (GWh)	1,327	1,315	1,302	1,290	1,280
Present Value of Benefit (mil. US$)	619.9	606.5	592.9	580.6	570.6
Present Value of Cost (mil. US$)	420.2	424.6	423.6	421.6	420.3
Benefit / Cost	1.48	1.43	1.40	1.38	1.36

Remarks; Maximum power output mentioned above is against average combined efficiency.

8.5.3 POWER PLANT AT RE-REGULATING WEIR

As an alternative scheme, providing a domestic power generating unit at the re-regulating weir was considered to utilize the residual head. In this case, the full supply level of the re-regulating pond would be determined at EL.181 m so as to utilize maximum head and to match with the tailwater level of the Nam Ngiep power plant at the dam.

The minimum operation level of the pond was selected as EL.176 m whereas the full supply level would be EL.181 m so as to have a capacity more than the required 4.5 mil. m^3 for re-regulation. The rated water level was set at 179.3 m i.e. two-thirds of drawdown.

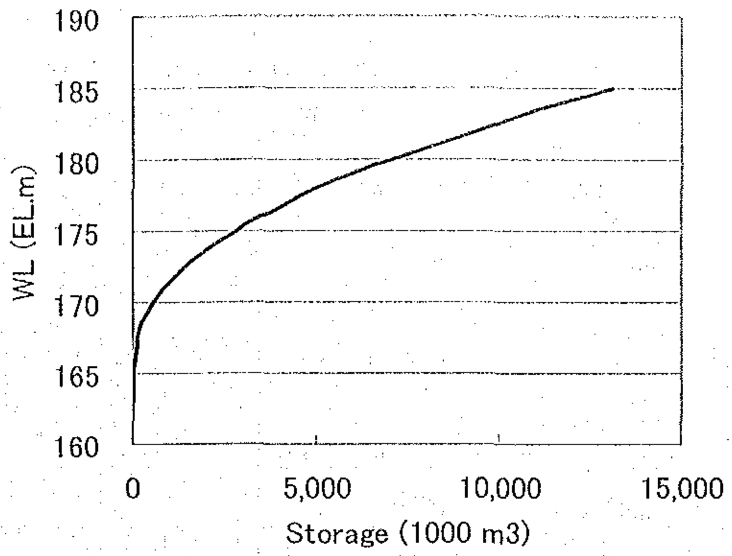


Figure 8.5.1 Storage Capacity Curve of Re-Regulating Pond

The maximum plant discharge at the design head was estimated at 160 m^3/s. The tailwater level corresponding to this discharge was obtained at EL.167.3 m by the results of non-uniform flow computation for the stretch between the weir and the stream gauging station at B. Hatkham.

In general, the variable discharge characteristics for the tubular turbine are considered to be about 20% of the rated discharge, and variable head characteristics are considered approximately 50% of maximum head for an adjustable runner blade. The discharge and head in case of both the full supply level and the minimum operation level satisfy these criteria for the above-mentioned dimensions as follows:

Table 8.5.2 Variation of Discharge and Head

Item	FSL: EL.181m	MOL: EL.176m
Q (m^3/s)	138	138
Q/Qmax	0.86	0.86
TWL (EL.m)	167.1	167.1
Head (m)	13.9	8.9
H/Hd	1.16	0.74

Annual energy generated by this plant was computed based on the outflow from the main dam obtained with the aforementioned simulation model under the condition that the FSL and MOL were set at EL.320 m and EL.296 m, respectively. The power unit for domestic use was assumed to take the base load and be operated continuously for 24 hours. Combined efficiency of generator and turbine for the domestic plant was assumed to be 0.87. As a result, annual average energy was estimated at 108 GWh.

B/C ratio of the Project with this additional unit is slightly higher than the without-case, as shown in the following table.

Table 8.5.3 Benefit/Cost Ratio for Alternative Domestic Power Supply

With or Without Power Plant at Re-Reg. Weir	Without	With
Annual Energy Production		
Maximum Plant Discharge in Total (m3/s)	230	(230)
for Export	230	230
for Domestic Use	0	160
Maximum Power Output in Total (MW)	252	268
for Export	252	252
for Domestic Use	0	16
Annual Average Peak Energy in Total (GWh)	1,173	1,281
for Export	1,173	1,173
for Domestic Use	0	108
Annual Average Off-Peak. Energy in Total (GWh)	154	154
for Export	154	154
for Domestic Use	0	0
Annual Average Total Energy (GWh)	1,327	1,435
Present Value of Benefit (mil. US$)	619.9	663.3
Present Value of Cost (mil. US$)	420.2	444.9
Benefit / Cost	1.48	1.49

Remarks; Maximum power output mentioned above is against average combined efficiency.

8.5.4 OPTIMUM FORMULATION

The B/C ratio analysis aforementioned indicates that domestic off-take arrangements for this Project can be achieved most economically if the independent power unit is provided at the re-regulating weir.

The operational characteristic for the scheme are as follows:

Table 8.5.4 Optimum Development Scheme for Domestic Use

No.	Item	Optimum Scale
1.	FSL	EL.181m
2.	MOL	EL.176m
3.	Max. Plant Discharge	160 m^3/s
4.	Installed Capacity	16.8 MW
6.	Average Annual Total Energy	108 GWh/year

9. PRELIMINARY DESIGN

9.1 GENERAL

The dam development based on FSL.320 m has been selected as the optimum development option. This section describes the layout and preliminary design concepts. The major features of the selected scheme are as shown in Table 9.1.1.

Table 9.1.1 Salient Features of Scheme

Place	Particular	Unit	FSL.320m Alternative
Reservoir	Catchment area at dam site	km^2	3,700
	Annual basin rainfall	mm	1,873
	Annual mean runoff	m^3/s	147.2
	Annual mean runoff	mill. m^3	4,642
	Average run-off coefficient	-	0.67
	Probable max. flood, PMF	m^3/s	14,220
	Mean annual sediment flow	$t/km^2/yr$	500
	Reservoir area at FSL	km^2	66.94
	Gross reservoir capacity	10^6m^3	2,241.2
	Min. operation level (MOL)	EL.m	296
	Draw-down	m	24
	Effective storage volume	10^6m^3	1,191.8
Main Dam	Dam type	-	CFRD
	Dam crest level	EL.m	325
	Parapet wall top level	EL.m	325.7
	Plinth bottom level	EL.m	174
	Dam height	m	151
	Dam crest width	m	10
	Dam crest length (in total)	m	513
	Upstream embankment slope	-	1:1.4
	Downstream embankment slope	-	1:1.4
	Face slab thickness	m	0.3 to 0.8
	Plinth apron width	m	4 to 12
	Curtain grout row numbers	nos	1 or 2
	Curtain grout depth	m	25 to 100
	Consolidation grout row numbers	nos	2
	Grouting tunnel length	m	30 and 100
	Dam embankment volume	10^6m^3	7.3
River Diversion	Design flood capacity for diversion	m^3/s	2,420 (Q=25yr)
	Tunnel lane numbers	nos	2
	Tunnel diameter	m	9.2
	Tunnel length	m	1,176 and 1,079
	Tunnel inlet level	EL.m	191.5
	Tunnel outlet level	EL.m	180.5
	Inlet tower top level (tunnel No.2)	EL.m	231
	Upstream cofferdam crest level	EL.m	228.5
	Upstream cofferdam height	m	46.5

Place	Particular	Unit	FSL.320m Alternative
Spillway	Design flood capacity for FWL	m^3/s	14,220 (Q=PMF)
	Design flood capacity for FSL	m^3/s	5,210 (Q=1,000yr)
	Design flood capacity for stilling basin	m^3/s	3,290 (Q=100yr)
	Spillway pier top level	EL.m	325
	Forebay sill level	EL.m	300
	Overflow crest level	EL.m	305
	Spillway gate numbers	nos	4
	Spillway gate width	m	12.25
	Spillway gate height	m	16.5
	Overflow weir length	m	33
	Chute width	m	58
	Chute length (horizontal)	m	250
	Slope of chute	-	1:1.99
	Stilling basin bottom level	EL.m	168
	Stilling basin width	m	58
	Stilling basin length	m	114
	Spillway excavation volume	10^6m^3	5.8
	Spillway concrete volume	10^3m^3	135
Bottom Outlet	Discharge capacity of outlet valve	m^3/s	200 (FSL.320m)
	Diameter of outlet valve	m	2.5
	Access gallery length	m	410
	Plug length in diversion tunnel	m	45
Power Intake	Design discharge	m^3/s	230
	Inlet forebay sill level	EL.m	262
	Inlet sill level	EL.m	263
	Inlet width	m	6.0 x 2
	Inlet height	m	5.25 x 4
	Intake shaft height	m	53
Headrace Tunnel	Lane numbers	nos	1
	Tunnel diameter	m	9
	Tunnel length	m	504
	Tunnel slope gradient	-	1:500
Surge Tank	Surge tank type	-	Restricted orifice
	Main tank shaft diameter	m	12
	Main tank shaft top level	EL.m	333
	Main tank shaft bottom level	EL.m	288
	Main tank shaft height	m	45
Penstock	Lane numbers	nos	1 (tunnel) to 2 (open)
	Penstock tunnel length	m	50
	Open penstock length (horizontal)	m	243
	Penstock diameter	m	8.0 to 4.0
	Steel penstock weight	ton	2,100
Power Station (Main Dam)	Design flood discharge for yard	m^3/s	3,290 (Q=100yr)
	Powerhouse yard level	EL.m	195
	Powerhouse type	-	Surface type
	Powerhouse length	m	70
	Powerhouse width	m	50
	Powerhouse height	m	46
	Turbine center level	EL.m	177.5
	Tail water level	EL.m	181.4
	Rated head	m	127.7
	Type of turbine	-	Vertical Francis
	Number of unit	No.	2
	Plant capacity	MW	260
	Peak operation hour	hour	16
	Annual energy	GWh	1,327
	Switchgear type	-	GIS

Place	Particular	Unit	FSL.320m Alternative
Re-regulating Pond	Full Supply Level (FSL)	EL.m	181
	Pond area for FSL	km^2	1.1
	Gross storage capacity	10^6m^3	8.2
	Min. operation level (MOL)	EL.m	176
	Effective storage volume	10^6m^3	4.7
Re-regulating Weir	Weir type	-	CFRD with spillway
	Weir crest level	EL.m	184
	Design flood capacity for FSL	m^3/s	3,290 (Q=100yr)
	Check gate numbers	nos	5
	Check gate width	m	12
	Check gate height	m	9.5
	Overflow crest elevation	EL.m	172
	Stilling basin bottom level	EL.m	163
	Design flood capacity for diversion	m^3/s	1,590 (Q=5yr)
	Diversion channel width	m	50
Power Station (Re-regulating Weir)	Design flood discharge for yard	m^3/s	3,290 (Q=100yr)
	Powerhouse yard level	EL.m	184
	Powerhouse type	-	Surface type
	Powerhouse length	m	49
	Powerhouse width	m	26
	Powerhouse height	m	30
	Turbine center level	EL.m	161
	Tail water level	EL.m	167.3
	Rated head	m	12.0
	Design discharge	m^3/s	160
	Type of turbine	-	Tubular
	Number of unit	No.	2
	Plant capacity	MW	16.8
	Peak operation hour	hour	24
	Annual energy	GWh	108
	Switchgear type	-	Outdoor switchyard
Transmission Line	Capacity for main P/S	kV	230
	Distance from main P/S to Nabong	km	125.2
	Capacity for re-reg. P/S	kV	115
	Distance from re-reg. P/S to Pakxan	km	40
Permanent Access Road	B.Nonsomboun to B.Hatkham	km	20.9
	B.Hatkam to dam site	km	12.1

9.2 ACCESS ROAD

9.2.1 SELECTION OF TRANSPORTATION ROUTE

The major materials to be transported from outside the dam site include cement and reinforcement bars for concrete structures, gate and penstock structures, and a transformer and waterwheel for power generating facility. Prospective rock quarry sites have been identified near/upstream of the proposed dam site, and core drilling investigations were carried out at these two locations. It is judged that the aggregate materials for concrete works will be produced locally.

Meanwhile, the construction equipment and facilities to be transported from the outside is mainly composed of large size construction equipment, aggregate crushing plant and concrete batching plant.

From a market survey of the construction materials and equipment available in Lao PDR, it is obvious that most of the materials and equipment will be procured from abroad. Cement and reinforcement bars will be procured from Thailand, while construction equipment, transformer, gate and penstock etc. will be mainly procured from developed countries.

The most likely transportation route for the imported construction materials and equipment will involve unloading at Bangkok port in Thailand then transportation by land from Bangkok to Vientiane. The friendship bridge that crosses the Mekong River will be utilized when passing between Thailand and Vientiane. The national road No.13 running from north to south passes Vientiane, and connects to a provincial road which branches off at Pakxan, the capital of Bolikhamxay Province. A road to the dam site branches off from the provincial road at B. Nonsomboun.

Although there is some timber transport via the Mekong River between Thailand and Lao PDR, no other water-borne transportation route has not been identified. In addition, there is no unloading facility near Pakxan City along the Mekong River or on the Nam Ngiep River.

It is therefore recommended that most of the construction materials and equipment to be transported from outside be transported by land to the dam site.

9.2.2 IMPROVEMENT TO EXISTING ROADS

The transportation route from the friendship bridge near Vientiane to B.Hatkham is shown in DWG.25 and 26. Based on the survey results of the existing roads, the hauling distance and road conditions for each section are shown below:

Table 9.2.1 Existing Road Conditions between Friendship Bridge and B.Hatkham

No.	Existing Road	Distance (km)	Road Condition
1	Friendship Bridge - Pakxan	161.7	Asphalt pavement, 6.0m wide, Two (2) PC bridges (Cap.: 80ton)
2	Pakxan - B.Nonsomboun	19.9	First 3km section from Pakxan is asphalt pavement, the remainder is laterite pavement, 6.0m wide, One (1) metal bridge (Cap.: 20ton, Width: 4m, Length: 25m)
3	B.Nonsomboun - B.Thahua	18.3	Non-pavement, 3.5m wide, Six (6) fords are crossed
4	B.Thahua - B.Hatkham	2.6	Non-pavement, 1.5-2.0m wide, Five (5) fords are crossed
Total		202.5	-

During the construction period, the maximum transport weight, including vehicles, is estimated to be about 80 ton, this being the transformer transport. Therefore, the following improvements on the existing road should be made in consideration of the loading limit and minimum road width required.

(i) Construction of Eight (8) box culverts

(ii) Construction of Three (3) bridges

(iii) Widening and pavement of the existing road between B.Nonsomboun and B.Hatkham

9.2.3 TEMPORARY AND PERMANENT ACCESS ROADS

A temporary road between B.Hatkham and the dam site along the left bank of the Nam Ngiep river was constructed for the geological investigation during this Study. This road could be improved and utilized for temporary access to the dam site during the construction. However, as it is anticipated that steep topography along this road might invite a rather high maintenance cost if used in the long term, the permanent access road to the dam site should be provided mainly along the right bank of the river.

The general layout of these temporary and permanent access roads is shown on DWG.02. Profiles and typical sections of these roads are shown on DWG.03.

The permanent access road, 12.1 km in length, begins at B.Hatkham on the left bank of the river, then passes by the SPC site office and branches into two at the re-regulating weir; one for right and the other for left banks. The former road passes over the re-regulating weir bridge and two dykes, and ends at the powerhouse yard and the intake gate shaft yard. Meanwhile, the latter road is the maintenance road for two dykes on the left bank. The carriageway of 6.0 m width will be paved with asphalt penetration macadam.

From the second dyke on the left bank, the temporary road extends to the dam site, a distance of 9.9 km. Another temporary road between the dam site and the prospective aggregate quarry site for concrete at B.Sopyouk will be also constructed. The road is laid along the left bank of the reservoir area for a distance of 23.7 km. General layout of this road is shown on DWG.04.

9.3 MAIN DAM

9.3.1 GENERAL ARRANGEMENT

The dam layout is composed mainly of two lanes of river diversion tunnels, a main concrete face rockfill dam (CFRD), a spillway with gated overflow portion, an intake structure and power waterway, a surface type powerhouse and outlet facilities. Proposed layout for the main dam is shown on Figure 9.3.1(DWG.05).

As the most appropriate dam type, the CFRD was selected taking account the site topography and geology, availability of construction material, and technical as well as cost advantages, compared with the earth core rockfill dam (ECRD) or roller compacted concrete dam (RCC).

In order to avoid excessive excavation at the spillway, the dam axis was selected about 1.1 km downstream from the confluence with the Nam Katha River. The spillway is laid out on the left abutment, while a power intake, an intake gate shaft, a headrace tunnel, a surge tank, penstock, a surface type powerhouse, and an open switchyard are laid out on the right abutment. Two lanes of river diversion tunnels, with a bottom outlet, are also laid out on the right abutment.

Geometrical characteristics of the main dam body are as follows:

- Crest Level: EL.325 m
- Parapet Wall Top Level: EL.325.7 m
- Plinth Bottom Level: EL.174 m
- Dam Height: 151 m (EL.325 m-EL.174 m)
- Crest Width: 10 m
- Crest Length: 513 m
- D/S Slope: 1:1.4
- U/S Slope: 1:1.4
- Dam Volume: 7.3 mil.m^3

Design of CFRD is carried out based on authorized standard such as ANCOLD (Australian National Committee on Large Dams) Guidelines on Concrete-Faced Rockfill Dams 1991, etc.

9.3.2 DAM AXIS

In the Phase I Study, the gorge of the Nam Ngiep River about 0.8 km downstream from the confluence with the Nam Katha River was identified as the promising dam site.

The layout for this dam axis was reviewed in the Phase II Study. As a result, it was revealed that excavation for the approach bay of the spillway would reach about 180 m height in this layout due to the topographic conditions, as seen in Figure 9.3.2.

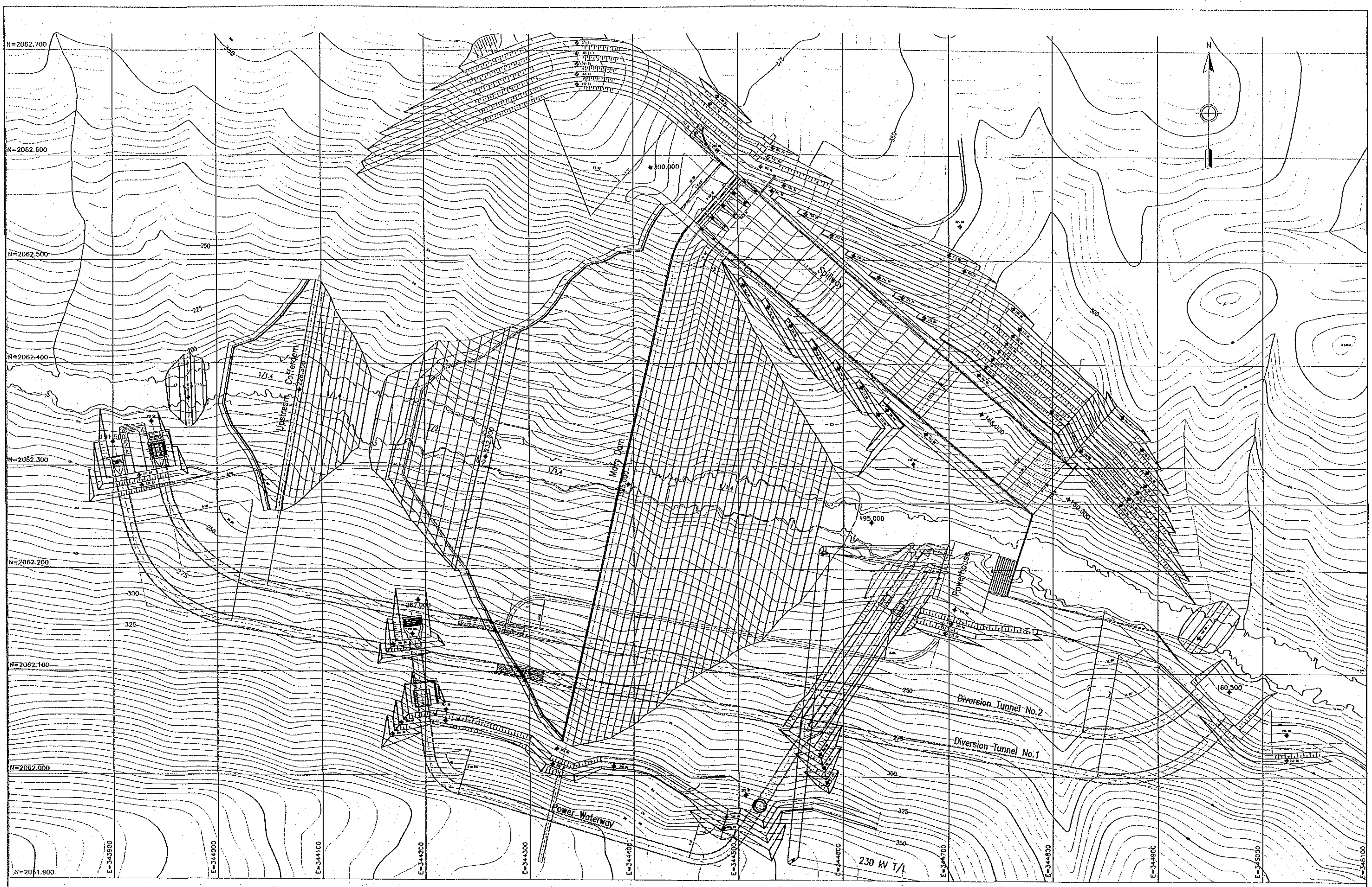
As such a high excavation should be avoided, especially for conglomerate rock that is liable to be split by tension in case of exposure, an alternative dam axis was considered further downstream from that location. As the topographic condition differs considerably between the right and left banks at this location, positioning of the spillway on both the right and left banks was examined as shown in Figure 9.3.2.

A comparison of the above-mentioned alternatives is summarized below.

Table 9.3.1 Comparison of Alternative Dam Axis

Item	Unit	Upstream Axis	Downstream Axis, R/Bank Spillway	Downstream Axis, L/Bank Spillway
Dam Embankment Volume	mil. m^3	7.0	7.2	7.3
Spillway Excavation	mil. m^3	7.1	8.5	5.8
Total of Direct Construction Cost	mil. US$	301.6	313.9	291.8
Excavation Height of Approach Bay	m	180	135	90

After comparing the above-mentioned alternatives, it was found that the downstream dam axis with the spillway positioned on the left bank would be superior to the other alternatives for reasons of economical construction cost as well as reduced height of excavation of the spillway. Accordingly, this layout was selected as the most appropriate alternative.

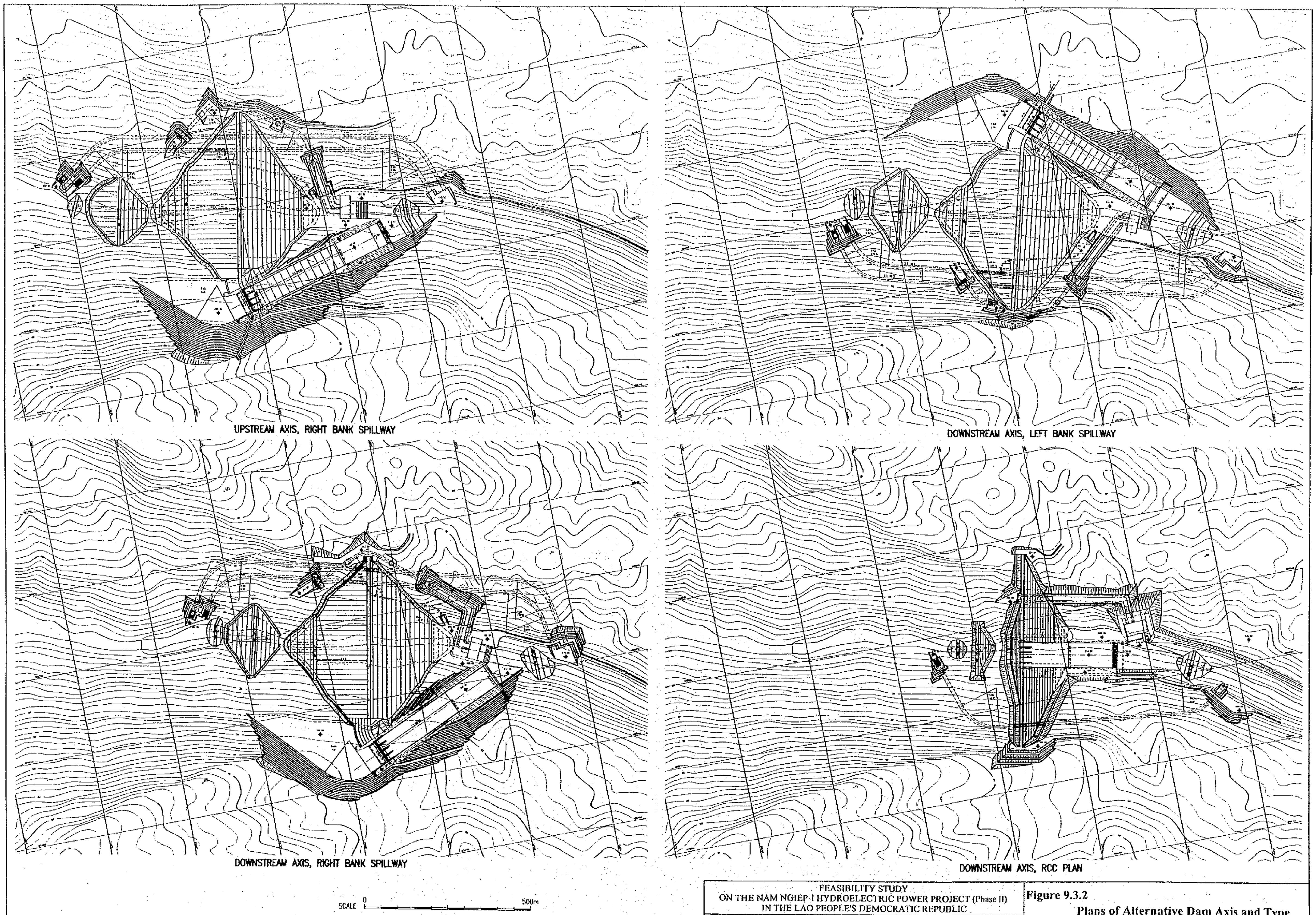


FEASIBILITY STUDY
ON THE NAM NGIEP-1 HYDROELECTRIC POWER PROJECT (Phase II)
IN THE LAO PEOPLE'S DEMOCRATIC REPUBLIC

JAPAN INTERNATIONAL COOPERATION AGENCY

Figure 9.3.1

Main Dam, General Plan



FEASIBILITY STUDY ON THE NAM NGIEP-1 HYDROELECTRIC POWER PROJECT (Phase II) IN THE LAO PEOPLE'S DEMOCRATIC REPUBLIC	**Figure 9.3.2** **Plans of Alternative Dam Axis and Type**
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9.3.3 DAM TYPE

From the topographic and geological constraints, the dam types conceivable for the Nam Ngiep-1 site were, (i) a concrete faced rockfill dam (CFRD), (ii) an earth core rockfill dam (ECRD), and (iii) a roller compacted concrete dam (RCC).

Of the above, ECRD, which requires a lot of soil for the embankment, was discarded as an alternative because sufficient soil material was not yet confirmed to exist near the dam site. The placement of core material would also have to be interrupted during the rainy season.

For the RCC option, a layout study was made as shown in Figure 9.3.1. The following geometrical characteristics of the dam body were assumed so that the dam body would be founded on hard rock (C_H) and satisfy the required stability.

- Crest Elevation : EL.325 m
- Crest Width : 7.0 m
- D/S Slope : Vertical (EL.325 m-314 m), 1:0.8 (below EL.314 m)
- U/S Slope : Vertical (EL.325 m-220 m), 1:0.4 (below EL.220 m)
- Height of Dam : 160 m (Dam bottom EL.165 m)

Cost comparison was made between CFRD and RCC as below.

Table 9.3.2 Comparison of CFRD and RCC

Item	Unit	RCC	CFRD
Dam Body Volume	mil. m^3	2.8	7.3
Unit Price for Dam Body	US\$/m^3	48.6	3.3
Total of Direct Construction Cost	mil. US$	332.7	291.8

CFRD was found be superior to RCC in terms of economical aspect and was therefore selected as the most appropriate design.

9.3.4 DETAILS OF MAIN DAM BODY

(1) Crest

A parapet wall of 4.7 m height is provided at the upstream face of the dam aiming to act as a wave wall, thus reducing the total freeboard required, as well as providing a safety barrier (0.7 m height) for personnel on the crest.

The top elevation of the parapet wall is determined at EL.325.7 m to satisfy the following criteria, considering wave height due to wind and seismic force, and allowance against gated fill-type dam.

- 5.00 m, or more, above FSL.320 m (>EL.320 m+5 m=325 m)

- 1.50 m, or more, above FWL.324.2 m in case of PMF (>EL.324.2 m+1.5 m=325.7 m)

Dam crest is set at EL.325 m, and its total length is 513 m. Crest width is set at 10 m wide, considering that the crest will provide a permanent crossing and a two-lane roadway. Details are shown on DWG.07.

A camber will be required so as to ensure that post-construction settlement of the dam does not encroach into the freeboard allowance. Generally, a camber of from 0.5 to 1.0% of embankment height is regarded as sufficient with well-compacted rockfill. In this case, a camber of some 1.0m at the highest dam embankment point will be necessitated. The drawings attached show elevations excluding such camber.

(2) Dam Slope

It is generally recognized that an embankment of hard rockfill with face slopes at the natural angle of repose (1:1.3) is inherently stable. For this Project, as some weaker embankment rockfills such as sandstone are utilized, flattening both upstream and downstream slopes to 1:1.4 is adopted, as shown on DWG.07.

(3) Dam Embankment Foundation

Principally, the river deposits, talus alluvium and highly weathered rock will be removed below the dam embankment. The dam body will be founded on the C_L class moderately weathered rock except for the plinth portion.

(4) Plinth

The plinth will be constructed to provide a watertight seal between the concrete face and the foundation rock. The plinth also serves as a grout cap for curtain and consolidation grouting.

In this Project, the plinth will be founded on C_M class firm rock, which is characterized with sufficient strength and low compressibility. Plinth bottom level will be EL. 174 m at the highest dam embankment.

With regard to erodibility, the C_M rock in this Project is categorized into two classes according to its geological structure; that is, "moderately erodible" for flexure zone at lower portion of the right bank and "non-erodible" for non-flexure zone at other portions.

Meanwhile, acceptable hydraulic gradients of 1:18 for "non-erodible" rock and 1:12 for "moderately erodible" rock are adopted as common practice for CFRD design. Thus, apron width of the plinth at the lowest bottom is set at 8 m on the left bank, and at 12 m on the right bank. Details are shown on DWG.07.

Width and thickness of the plinth apron will vary, according to the foundation elevation, as shown below.

Table 9.3.3 Applied Apron Width of Plinth

Elevation (m)	Apron Width at Left Bank (m)	Apron Width at Right Bank (m)	Apron Thickness (m)
Below 220	8.0	12.0	0.80
220 – 250	6.0	8.0	0.65
Above 250	4.0	6.0 - 4.0	0.50

The apron thickness above is taken as the same degree of thickness of the concrete face, as they will both be subject to the same reservoir head. In the plinth, a single layer of reinforcement amounting to 0.3% of the apron cross-section will be placed as a minimum. The plinth will be anchored to foundation rock with anchor bars to resist construction loads and uplift pressures from grouting.

Flexible joints will be provided at the perimetric joint between the plinth and the concrete face slab to accommodate the slab movement. Joint filler as well as both primary and secondary waterstops will be provided. Waterstop will be also provided at transverse joint of the plinth.

The rock foundation will be covered by reinforced shotcrete for 5 m and 2 m from the downstream and upstream sides of the plinth respectively, so as to increase the seepage path and thus decrease the hydraulic gradient.

(5) Foundation Treatment

Grouting from surface of the plinth apron includes grout curtain and consolidation grouting. The purpose of grout curtain is to form a deep zone of low permeability. Meanwhile, consolidation grouting is designed so as to prevent leakage immediately below the plinth where the hydraulic gradient is the highest.

Principally, three rows of grout holes, that is, one curtain grouting row and two consolidation rows, one on each side of the main curtain, are proposed. At the right bank zone where flexure structure is observed, however, the plinth will be widened and the curtain grouting will be increased to two rows.

The criterion to define the depth to which the curtain grouting should be taken is not standardized. In this project, following the empirical practice used in similar cases, (H/3 + 25 where H: dam height above the holes in meter) is applied for the left bank non-flexure zone. Another 25 m is added in depth for the right bank, that is (H/3 + 50). Thus, the deepest curtain grout at the right bank bottom reaches 100 m depth. Curtain grout depth arrangement is shown on Figure 9.3.3(DWG.06).

The split-spacing method will be used to locate the holes; mostly, the primary holes will be spaced at 6 m intervals; secondary holes, at mid spacing between primaries, and tertiary holes at mid-spacing between secondaries, could be drilled and grouted based on the grout take.

Depth of consolidation grouting will most probably be at 10 m for flexure zone and 5 m for non-flexure zone below the plinth Grout holes may be arranged at 3 m intervals and single stage grouting will be performed.

A grouting tunnel will be constructed of 100 m length at the right bank and 30 m length at the left bank for the grouting works.

(6) Face Slab

Minimum thickness of 0.3 m is adopted considering constructivity. Furthermore, the hydraulic gradient across the face is limited to 200 according to the past practice. Thus, the face slab thickness is designed by applying the equation (0.3 + 0.003 x H) in meter. The thickness varies between 0.3 m at the top and 0.8 m at the deepest bottom.

Reinforcement will be provided to secure a steel area equal to 0.5%, at minimum, of the design concrete area in each direction. Single waterstop will be provided at the vertical joints of the slab.

(7) Zoning of Dam Embankment

Zones of the dam embankment are categorized into three; that is, transition zone, main rockfill zone and sealing zone, as shown on DWG.07.

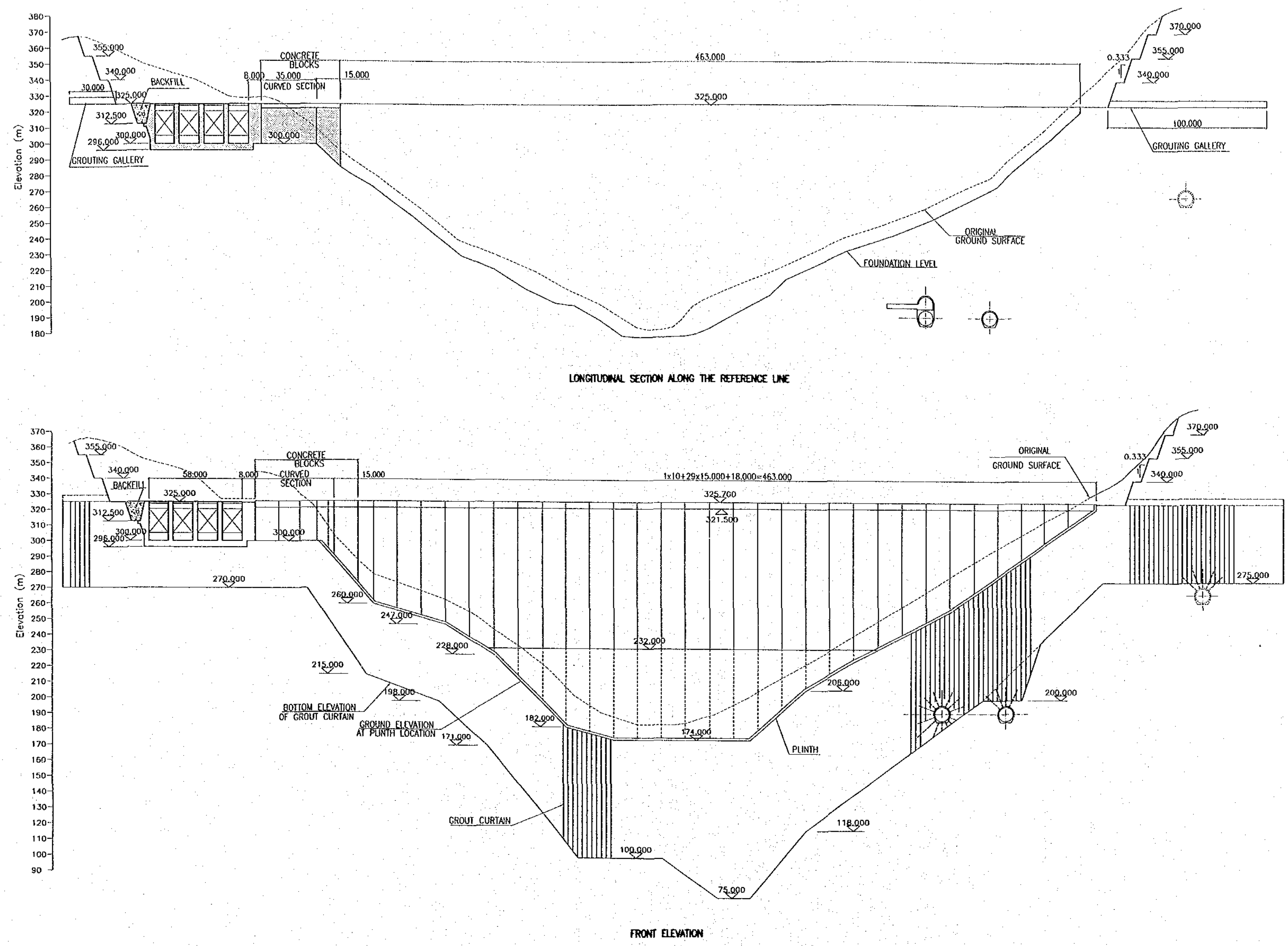
The transition zone is a thin layer between the concrete face and the rockfill, and divided into two zones in the up- and downstream direction.

The upstream transition zone is designed to support the face slab with angular particles, such as processed (crushed) small rock, with a width of 5 m ("2" in DWG.07), which is convenient for placing and compaction of the material by vibratory rollers in layer thickness of 0.5 m. For its gradation, road base type specification of the material at the maximum of 75 mm in size will be adopted so as to bring a competent base for concrete as well as semi-pervious grading to limit leakage.

Immediately behind the upstream transition zone, the downstream transition zone of a layer with selected rockfill of 5 m wide will be placed to fulfil the role of a filter between the upstream transition zone and the main rockfill zone ("3A"). This zone will be constructed with selected rock which is intermediate sizes between its upstream and downstream zones, and to be placed in the same width and the same layer thickness at the upstream zone.

The main rockfill zone is basically pervious and capable of carrying the imposed reservoir load, and is also divided into two zones: the upstream shell and the downstream shell. The upstream two-thirds of the zone will be designed as the upstream shell and the reminder will be designed as the downstream shell to transfer water load to the foundation level moderately.

Conglomerate (medium strength) and sandstone (low to medium strength) may be used as embankment material. Better conglomerate will be used for the upstream shell zone to minimize settlement and hence displacement and cracking of the concrete surface slab ("3B").



FEASIBILITY STUDY
ON THE NAM NGIEP-1 HYDROELECTRIC POWER PROJECT (Phase II)
IN THE LAO PEOPLE'S DEMOCRATIC REPUBLIC

JAPAN INTERNATIONAL COOPERATION AGENCY

Figure 9.3.3
Main Dam, Longitudinal Section and Front Evelation

The material will be mostly placed in 1 m thick layers, other than the thinner 0.5 m layer adjacent to the transition zone. Meanwhile, sand stone will be used for the less critical downstream shell zone in which layer thickness may be increased to 1.5 to 2 m thick ("3C"). The maximum size of rock will be equal to the layer thickness.

A well-graded material from gravel to silt sizes will be placed as a precaution against emergency cases of perimeter-joint leakage through the slab ("1A"). Available random fill will cover the sealing material ("1B").

9.4 RIVER DIVERSION

9.4.1 GENERAL ARRANGEMENT

During the construction, the river water will be diverted via two diversion tunnels driven through the right abutment. Separate main cofferdams will be provided at the upstream and downstream sides of the main dam. The tunnels will be driven through geological formation mainly of mudstone, but with some sandstone.

After completion of the river diversion function, one of the tunnels (Diversion Tunnel No.1) will be permanently plugged, while an outlet will be installed in the other tunnel (No.2). So as to prevent clogging at the inlet due to silting, a vertical shaft and trash beam structure will be provided at the No.2 tunnel. Layout of these tunnels are shown on DWG.11 to 14.

Geometrical characteristics for the river diversion are as follows:

- No. of Lanes:	2 nos.
- Tunnel Diameter:	9.2 m
- Lining Thickness:	70 cm
- Tunnel Length:	1,176 m (No.1) and 1,079 m (No.2)
- Height of cofferdam:	46.5 m (Upstream coffer)

9.4.2 DESIGN FLOOD FREQUENCY

The criterion to define design flood frequency of diversion for CFRD to which the flood discharge should be discharged is not standardized.

The following is the design flood frequency that has been adopted in planning of CFRD projects in Laos.

Table 9.4.1 Design Flood Frequency for Diversion for CFRD in Laos

Project	Adopted Design Flood Frequency (years)
Hoay Ho	20
Nam Ngum 2	25
Nam Ngum 3	100

As the powerhouse is positioned at the dam toe, its foundation and substructure might be subject to a higher risk of flooding if a smaller return period is adopted. As this may hamper the overall progress of the Project, additional safety margin for the river diversion will be adopted.

Based on the above, the 25-year recurrence probable flood of 2,420 m^3/s is adopted for this Project. Less probable flood, such as 5-year flood of 1,590 m^3/s, 10-year flood of 1,930 m^3/s or 20-year flood of 2,300 m^3/s is still conceivable as alternative design, but not adopted at this time.

9.4.3 OPTIMUM DIVERSION PLAN

(1) Alternative Plan for Diversion

As an alternative to the construction of a separate main cofferdam at the upstream side of the main dam, staged construction of the main dam to facilitate river diversion by itself is conceivable. In this case, the CFRD for the main dam should be raised during one dry season to diversion flood level.

As the geological formation has a flexure structure at the bottom of the right bank, it is judged that grouting work at this section will be critical and difficult, with the possibility of unknown problems. Completing the CFRD up to diversion flood level in one dry season might be difficult and therefore the staged construction option has been discarded.

(2) Optimum Diversion Dimensions

At least two tunnels are required for discharging a rather large design flood, as well as installing the bottom outlet facilities in the one of the tunnels while the other tunnels continues to divert the river flow.

Optimum diversion dimensions are obtained through comparison of the sum of construction cost for diversion tunnels and cofferdams. Table 9.4.2 illustrates results of the comparison, which shows the lowest cost for the diversion tunnels diameter of 9.2 m with upstream cofferdam elevation of EL. 228.5 m.

Table 9.4.2 Comparison of Diversion Tunnel Diameter

No.	Item	Unit	Alternative Diameter				
1.	Diameter of Diversion Tunnel	m	8.8	9.0	9.2	9.4	9.6
2.	Height of Upstream Cofferdam	1,000 US$	53.2	49.7	46.5	43.6	40.9
3.	Construction Cost	1,000 US$	21,555	21,364	21,300	21,346	21,469

Note: The cost above includes construction cost for tunnels, inlet/outlet concrete, plug concrete and upstream cofferdam.

9.4.4 DETAILS OF DIVERSION STRUCTURES

The diversion will be operated in both the free-surface flow for low discharge and pressure flow for high discharges. The rating curve for relationship between the reservoir water level and the discharge capacity of diversion in case of two lane tunnels of 9.2 m diameter is shown on Figure 9.4.1.

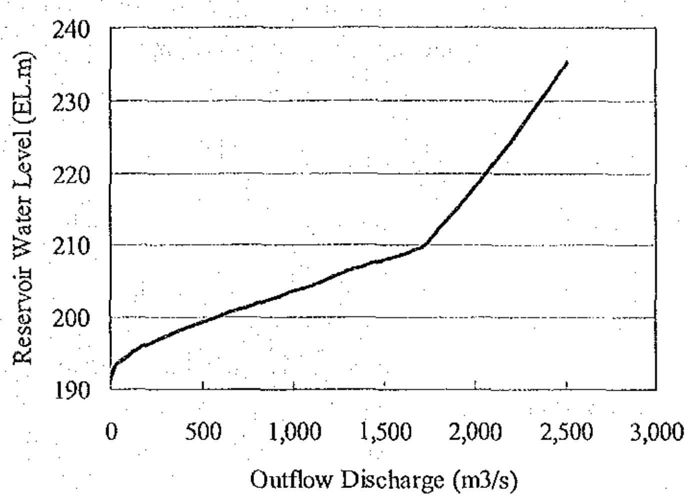


Figure 9.4.1 Rating Curve for River Diversion Discharge

Although the reservoir volume up to the crest of the upstream cofferdam is small, a small percentage of reduction in outflow could still occur due to the routing effect of the reservoir. For the case of a 9.2 m diameter diversion tunnel, the maximum outflow for the 25-year recurrence probable flood is reduced to 2,260 m^3/s. Maximum reservoir water level that accords to this outflow is EL. 226.5 m.

The upstream cofferdam will be 46.5 m high CFRD. Crest level at EL. 228.5 m is set so as to enable discharge the 25-year recurrence probable flood with 2 m freeboard. Meanwhile, crest level of the downstream cofferdam at El.188 m was assessed from the backwater calculation for the 25-year flood plus 1 m.

The designed inlet and outlet sill levels of the tunnel are EL. 191.5 m and EL. 180.5 m, respectively. Length of the diversion tunnels are 1,176 m (Tunnel No.1) and 1,079 m (No.2). The tunnels have a circular cross section of 9.2 m internal diameter and the distance between the centerlines of the two tunnels is set at 40 m.

The tunnel will be lined with 0.7 m thick concrete. The lining will be reinforced, particularly at the invert lining that will be subject to erosion during free-flow conditions. Backfill grouting at the crown and consolidation grouting around the whole perimeter of the tunnel will be carried out along its entire length. Furthermore, two (2) sets of diversion gates will be provided at the inlet structure of the diversion tunnels.

9.5 SPILLWAY

9.5.1 GENERAL ARRANGEMENT

The spillway is a gated overflow structure located on the left abutment of the main dam. The crest consists of four bays provided with radial gates which discharges the floods. Floodwater is conveyed downstream through an open square concrete chuteway and dissipated by the flat-apron type stilling basin. Layout of the spillway structure is shown on DWG.15 to 18.

Geometrical characteristics for the spillway are as follows:

- Spillway Pier Top Level:	EL.325 m
- Forebay Sill Level:	EL.300 m
- Overflow Crest Level:	EL.305 m
- Gated Weir:	4 nos., 12.25m wide x 16.5m high
- FWL in case of PMF:	EL. 324.2 m
- Chute Width:	58 m
- Stilling Basin Bottom Level:	EL.168 m
- Length of Stilling Basin:	114 m
- Excavation Volume:	5.8 mil. m^3

9.5.2 DESIGN FLOOD FREQUENCY

Based on the common practice of dam design, the spillway was designed to discharge a PMF (14,220 m^3/s) safely, with incorporating the routing effect of the reservoir.

According to the backwater effect examination detailed in Chapter 8, it is found necessary to keep the reservoir water level during floods equal to or below FSL 320 m, so as to avoid any substantial flood damage at the Thaviang Sub-District. Thus, the spillway gates were designed to discharge 5,210 m^3/s (1,000-year recurrence flood) at FSL with full opening.

The stilling basin of the spillway was designed for the 100-year recurrence flood with peak discharge of 3,290 m^3/s.

9.5.3 ALTERNATIVE PLAN FOR SPILLWAY

The spillway for the Project was designed as a gated overflow type so that FSL might be set as high as possible without causing negative impact due to inundation in the case of flooding at the areas of upstream reach of the reservoir. For this reason, other spillway type such as non-gated weir or combination of gated/non-gated have been discarded from this study.

9.5.4 DETAILS OF SPILLWAY STRUCTURES

The forebay to be provided in front of the spillway weir functions to lead flood discharge smoothly to the overflow weir of the spillway. The bed excavation level is set at EL. 300m, 5m lower than the gated ogee crest level.

The overflow section is designed to have four (4) 12.25 m wide and 16.5 m high radial gates. This overflow weir discharges the peak of the 1,000-year recurrence flood of 5,210 m^3/s at FSL 320 m at full gate opening. At PMF, maximum outflow from the spillway during the routing of PMF becomes 7,860 m3/s and reservoir level corresponding to this discharge is FWL.324.2 m. Dimensions of this section are determined so that the discharge per unit width might not surpass 200 $m^3/sec/m$, and also that the gate height might not surpass 20 m. One (1) set of stoplogs are also provided with a gantry crane.

The spillway chute conveys the discharge released from the reservoir smoothly. The chute width is decided to be 58 m. The training wall height is set at 7.6 m so as to discharge PMF outflow of 7,860 m^3/s without overtopping.

The stilling basin is a standard apron type designed against the 100-year recurrence probable flood outflow of 3,290 m^3/s. The length of the stilling basin is set at 114 m, and the bottom level at EL.168 m.

9.6 BOTTOM OUTLET

9.6.1 GENERAL ARRANGEMENT

Bottom outlet facilities are provided for retardation of reservoir-rise during impoundment, releasing riparian flow to the downstream reach, emergency draw-down, and so on.

An outlet is provided in the diversion tunnel No.2, which will close after the completion of the main dam. A valve chamber is provided in the main plug portion of the tunnel, which will be constructed at the middle of the tunnel stretch. Access to this valve chamber will be made through a gallery arranged at the right bank with an entrance located at the powerhouse yard.

Geometrical characteristics for the bottom outlet are as follows:

- Discharge Capacity: 200 m^3/s for FSL
- Outlet Valve: 2.5 m dia.
- Steel Conduits: 3.5 m dia., 45 m length
- Access Gallery Length: 410 m
- Plug Length: 45 m