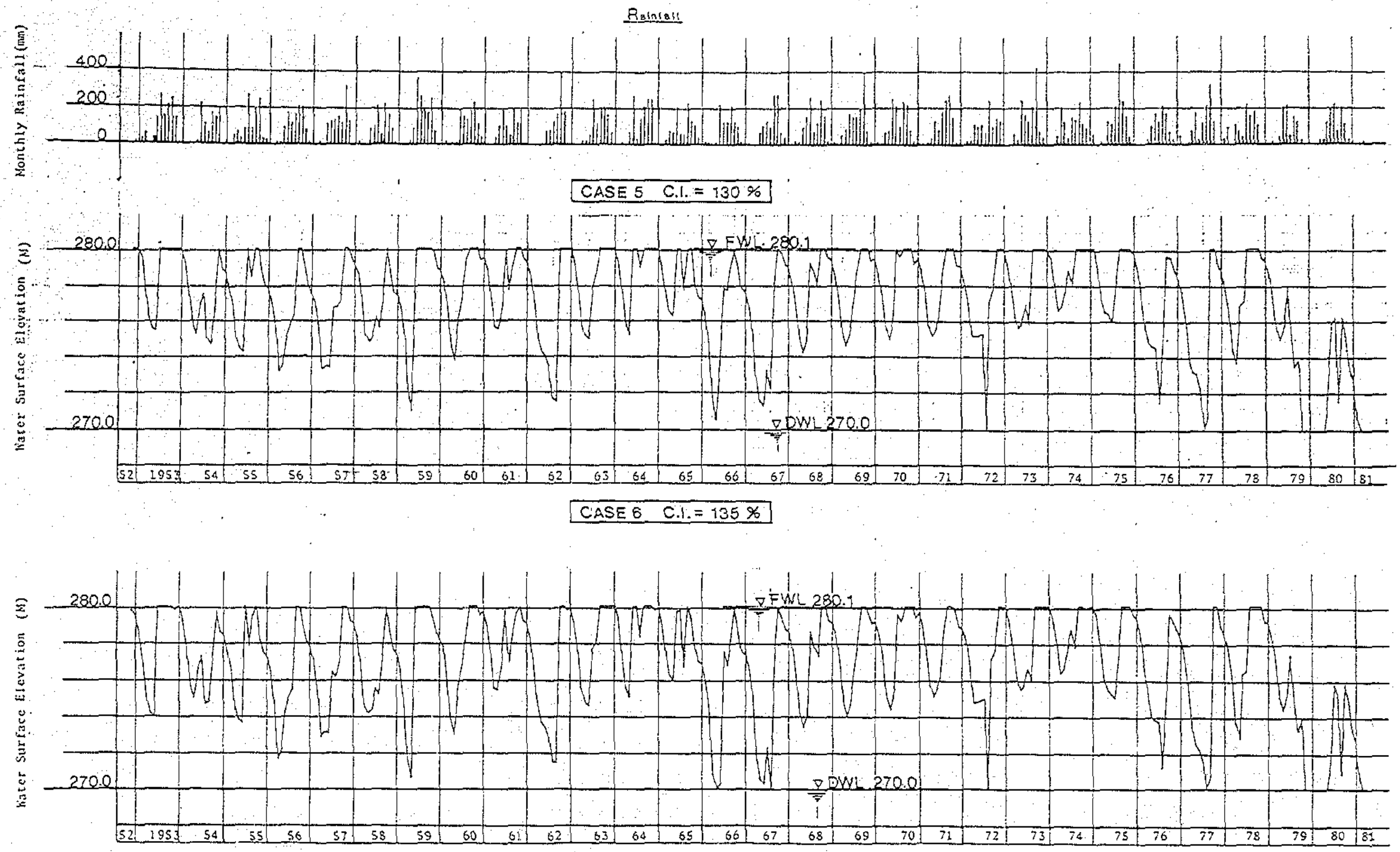


FIG. 4.2-2 WATER BALANCE SIMULATION (1)

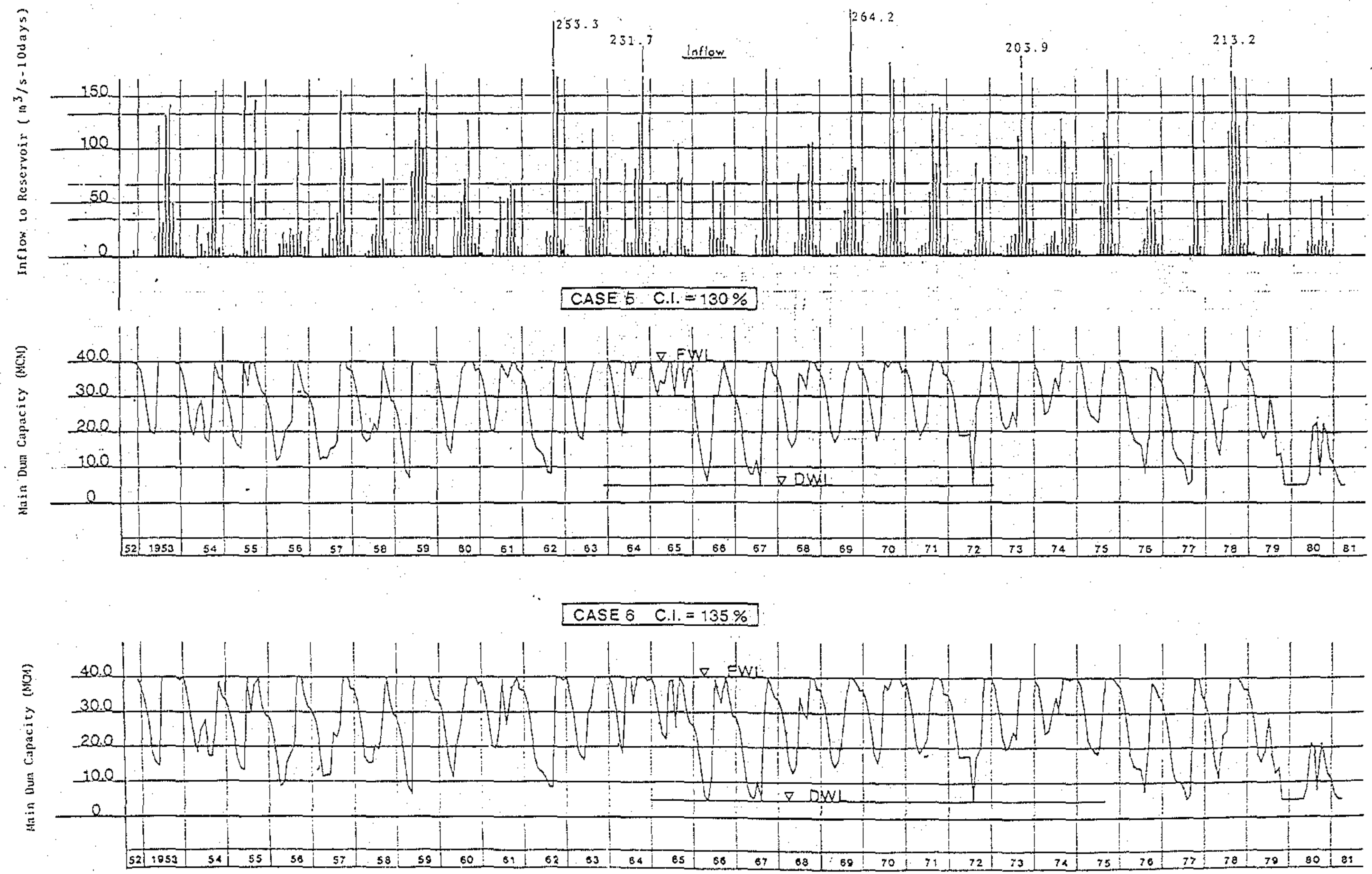


FIG. 4.2-3 WATER BALANCE SIMULATION (2)

4.2.6. Flood Control Plan

There are two methods of flood control. One is the river improvement method and the other is the reservoir storage method. Features of each method are briefly described below.

River Improvement Method

The river improvement method aims at smooth river flow by providing dikes, rivetments, dredging and flood ways. This method requires a great deal of land acquisition and reconstruction of bridge, weir, gate and other river structures.

Reservoir Storage Method

The reservoir storage method can be divided in to two examples. One provides a dam or group of dams at the upper reaches of a river system. The other builds a retarding basin in the middle reaches of a river. It is very difficult in most cases to construct a retarding basin since extensive land is required.

On the other hand, the reservoir storage method commonly utilizes stored water for several purposes to serve regional development.

In the Project Area, irrigation is the primary concern. However, flood control is also taken into account.

According to the hearings to villagers at the downstream along the Mae Chang, any flood damage have never been experienced. After the dam construction, flood controle by the dam can be expected.

4.2.7. Drainage Plan

1) Drainage Modulus from the Irrigated Paddy Field

The paddy fields in the Project Area drain excess water by natural gravity flow depending upon the topographic slope. The drained water flows into the low-lying fields, thus inundation will occur and crops will suffer damage.

Considering the above matters, the drainage modulus in the irrigated paddy fields was computed by applying the following method with a five-year return period and two consecutive rainfall observed at the Mae Tha station.

$$D = A\ (R\ (n,\ \max)\ t - n\ (Dc + Cu))$$

Where,
- D: Water depth on paddy field at n-day after the first rainfall
- R(n, max)t: Maximum rainfall during n-day with t-year return period (mm/day)
  R(1, max) 5 = 95 R(2, max) 5 = 117
- Dc: Drainage capacity (mm/day)
- Cu: Consumptive use of paddy (mm/day) = 49

On the basis of the above method, the drainage modulus is obtained at 4.88 lit./sec/ha. (Refer to Appendix 4.2-4)

To apply this modulus for each drainage system, the reduction rate should be considered under the Area effected by specific rainfall distribution i.e., spot rainfall in the tropical zone.

2) Drainage from Hilly Area

The peak discharge from the hilly area was estimated by the Rational method.

The drainage modulus on the hilly area was obtained at 24.3 lit./sec/ha, which value is also modified depending upon the drainage area by using the reduction rate. (Refer to Appendix 4.2-4.)

4.2.8. On-farm Development Plan

The provision of on-farm facilities such as main farm ditches, supplementary farm ditches, field ditches, division boxes, end checkes and farm road is essential work for irrigated agriculture with farm mechanization, and farmers' eagerness for agriculture will act as the prime mover. The careful control and measurement of water are required to carry out proper water management.

It is recommended to implement the on-farm development by the RID quickly to realize the purpose of the agricultural development. However, taking the recent Thai Government policy on the agricultural development into consideration implemendation of on-farm development should be carried out by farmers themselves under assistance of the Project office and Regional office of RID after completion of the construction of main and lateral canals.

As discussed in Appendix 4.2-5, the on-farm facilities are projected as such that an irrigation block (chak) covers a 250 rai (40 ha) of cultivated land and each facility is planned depending on the following criteria.

	Quantity	
Description	Per Chak	Per Rai
Main Farm Ditch	300 m	1.2 m
Supplementary Farm Ditch	4,100 m	16.4 m
Field Ditch	4,000 m	16.0 m
Division Box	4 sets	0.016 sets
End Check	5 sets	0.020 sets

4.2.9. Hydropower Development Plan

The purpose of this plan is to develop electric power generation by a hydrpower generation plant proposed at immediately downstream the planned Storage Dam A. The designed installed capacity is 164 KW (two units of 82 KW) that will generate about 1.18 GWH of energy.

The proposed Storage Dam A, which will be operated to meet irrigation requirements and can serve power generation by its discharge and water head available.

The power station was so designed that a firm peak can be secured to obtain a maximum output of 338 KW (two units of 119 KW) for a net head of 17 meters and a discharge of 1.65 cubic meters per second. Twin Francis horizontal shaft, turbine-generator units are proposed for this plant.

The two lines and 600 millimeter dia. penstocks as major structures of the hydropower plant is branched off from the outlet pipe with 2.0 meter dia.

The scale of the hydropower stations are as follows:

1) Output

Maximum Output	238 KW (119 KW x 2)
Rated Output	164 KW (82 KW x 2)
Minimum Output	45 KW
Annual Generation	1.18 GWH

2) Discharge and Head

Maximum Discharge	1.65 m^3/sec
Maximum Head	17.0 m
Rated Net Head	13.0 m

3) Hydraulic Turbine

Type:	Horizontal Shaft Francis
No. of Units:	2 sets
Rated Output:	82 KW each
Rated Speed at 50 HZ:	1,000 rpm
Rated Net Head:	13.0 m
Rated Flow:	0.75 m^3/s each
Net Head Range:	9.0 m min. - 17.0 m max.
Flow Range:	0.50 m^3/s - 1.65 m^3/s

4) Synchronous Generator

Type:	Horizontal shaft synchoronous, hydraulic turbines driven at both shaft ends
No. of Units:	1 set
Output	278 KVA
Rated Voltage:	416 V
Power Factor	80%
Frequency:	50 Hz
No. of Poles	6 p
No. of Phases	3 Phase
Rated Speed	1,000 rpm
Rating	Continuous
Class of Insulation:	Class B

As for a decision of the hydropower plan, average discharge available for hydropower was studied based on the average flow analyzed from the data in a period from 1952 to 1981 and flow duration curve as shown in Appendix 4.2-6. From the above-mentioned flow duration curve, more than 0.50 m^3/sec of flow can be utilized for hydropower generation in a period of 80 per cent of a year and 1.50 m^3/sec of flow is in a period of 55 per cent. On the other hand, fluctuations of the water level in the reservoir was studied as shown in Appendix 4.2-6, resulting in a maximum output of 238 KW (119 KW x 2) with a net head of 17 m, and a rated output of 164 KW (82 KW x 2) with a net head of 13 m and a minimum output of 45 KW with a net head of 9 m. Furthermore, the annual generation was estimated at 1.18 GWH in average. Proposed facilities of the hydropower plant are shown in Drawings No.16.

Taking into consideration the objectives of the Mae Chang Irrigation Project, the hydropower generation will be developed in the near future as Phase II development after completion of the irrigation project.

As a reference for future development, the construction cost of the hydropower plant was estimated as follows.

Description	Cost (฿'000)
(1) Civil Works	2,000
(2) Hydropower Facilities	35,000
Horizontal Shaft Francis Turbine (2 units)	5,000
Accessories of Turbine	7,500
Horizontal Shaft Synchronous Generator	3,500
Accessories of Generator	8,000
Distribution Line for 1 km	1,000
Transportation on Boat and in Land and Installation Cost	10,000
(3) Communication Facilities and other Appurtenances	5,000
(4) Engineering and Administrative Charge (30% of (1) to (3))	13,000
Total	55,000

4.3. Agricultural Development Plan

4.3.1. Agricultural Production

Agricultural production will be rapidly promoted after completion of the Project. With the progress of the Project, processing of products, agricultural input materials and transportation will require improvement, and increments for existing systems and facilities. The required labor depends upon farm household members and as a result, the farm economy will prosper due to the increment of opportunity and income from the non-agricultural sectors. The major agricultural products for each case are shown as follows:

(Unit: ton)

Item	Case 5 Present	Case 5 Proposed	Case 6 Present	Case 6 Proposed
Paddy	8,653 (100)	14,094 (163)	7,424 (100)	12,644 (170)
Soybean	133 (100)	2,670 (2,008)	125 (100)	2,277 (1,822)
Groundnut	652 (100)	2,775 (426)	606 (100)	3,340 (551)

1) Selection of Crops

Changwat Lampang in which the Project Area is located is included in the agricultural economic Zone 9. It produces cotton, tobacco, soybean and cattle. Major agricultural production from Amphoe Mae Tha and Ko Kha are composed of rice, sugar, groundnut, corn, tobacco and soybean.

According to the land classification, the Project Area is classified into paddy fields and upland fields. The paddy fields are proposed to be planted with paddy as usual. In the dry season, they are planted with soybean and groundnut taking into consideration promotion of soil fertility, reduction of irrigation water and economy. In addition to such crops, tobacco and garlic are introduced. Concerning the upland fields, major crops consist of soybean and groundnut. Sugarcane will be planted to some extent.

2) Proposed Cropping Pattern

Paddy fields in the wet season are planted with paddy rice which is composed of 60 percent non glutinous rice and 40 per cent glutinous rice. In the dry season they are planted with soybeans, groundnuts, garlic and tobacco.

Upland crops are proposed to be soybeans, groundnuts and sugarcane. The former two crops are to be cropped in both seasons and the latter to be one cropping in three years and to be harvested three times. The proposed cropping period is shown as follows:

Crops	Nursery Period	Planting Period	Harvested Period
Paddy	25 days	Late in Jul. - Early in Sept.	Late in Oct. - Middle of Dec.
Soybeans			
Wet season		August	Early in Nov. - Early in Dec.
Dry season		Middle of Dec. - Middle of Jan.	Late in Mar. - Late in Apr.
Groundnuts			
Wet season		August	Late in Nov. - Late in Dec.
Dry season		Middle of Dec. - Middle of Jan.	Middle of Apr. - Middle of May
Tobacco		Middle of Nov. - Middle of Dec.	Early Apr. - Early in May
Garlic	50 - 55 days	Middle of Nov. - Middle of Dec.	Late in Mar. - Middle of Apr.

3) Requirements of Input Materials and Labor

Input Material

Applicable amount per rai of agricultural inputs such as seed, fertilizer and pesticide was determined by discussions with RID on the basis of a cultivation guideline prepared by the agricultural extension office and reports by the agricultural experiment station. (Details are shown in Table 4.3-1.2.)

Material	Case 5	Case 6
Seed (ton)	2,593	2,574
Seedling ('000 pcs)	3,250	3,250
Fertilizer (ton)	2,865	2,698
Pesticide (ton)	177	160
Rice Straw (ton)	4,500	4,500

Labor

The number of farm households within the Project Area is estimated at 6,072 in Case 5 and 5,562 in Case 6. The number of persons per household is 4.9 persons out of which 2.9 persons are assumed to be labor.

In Case 5, annual labor requirements, are 1,607,621 man.days and the available labor force is 5,088,945 persons. Peak requirements appear in December at 350,371 man.days with a labor force of 457,829 person.day. Therefore, a deficiency of labor will not occur. Deficiency will not occur in the usage of draft animals.

In Case 6, a labor force of 4,661,514 is available compared to the annual labor requirements of 1,510,162 man.days. The peak demand of 323,498 man.days appears in December but a more than sufficient labor force of 419,375 persons can be supplied. This fact shows that labor deficiency will not occur. As for the usage of draft animals, no deficiency occurs. (See Table 4.3-3 - 4.3-6.)

Judging from the above mentioned labor balance, introduction of agricultural machinery into the Area is not proposed.

4) Agricultural Production

Case 5

The proposed net irrigable area is 50,600 rai of which 40,500 rai (80%) is cultivated with paddy and the remaining 10,100 rai (20%) with upland crops.

Paddy - 60 per cent non glutinous rice, 40 per cent glutinous rice - is planted in the rainy season. Upland crops, i.e., soybean, groundnut, sugarcane, garlic and tobacco are planted in paddy and upland fields in the dry season. The cropping intensity is 130 per cent.

Case 6

The proposed net irrigable area is 45,900 rai of which 36,400 rai (79 per cent) is cultivated with paddy and the remaining 9,500 rai (21 per cent) with upland crops.

Paddy is planted in the rainy season with the same composition as Case 5.

The target yield per rai for each proposed crop was determined by discussions with RID staff referring agricultural statistics and tested results carried out by the research center.

Each crop production in both cases is summarized in Table 4.3-9, 11.

5) Animal Husbandry

Fodder crops are not included in the proposed cropping. To promote beef cattle production, by-products of crops such as rice straw and soybean residue are utilized by mixing them with wild grasses.

Figure 4.3-1 Cropping Calendar Case 5 (130%)

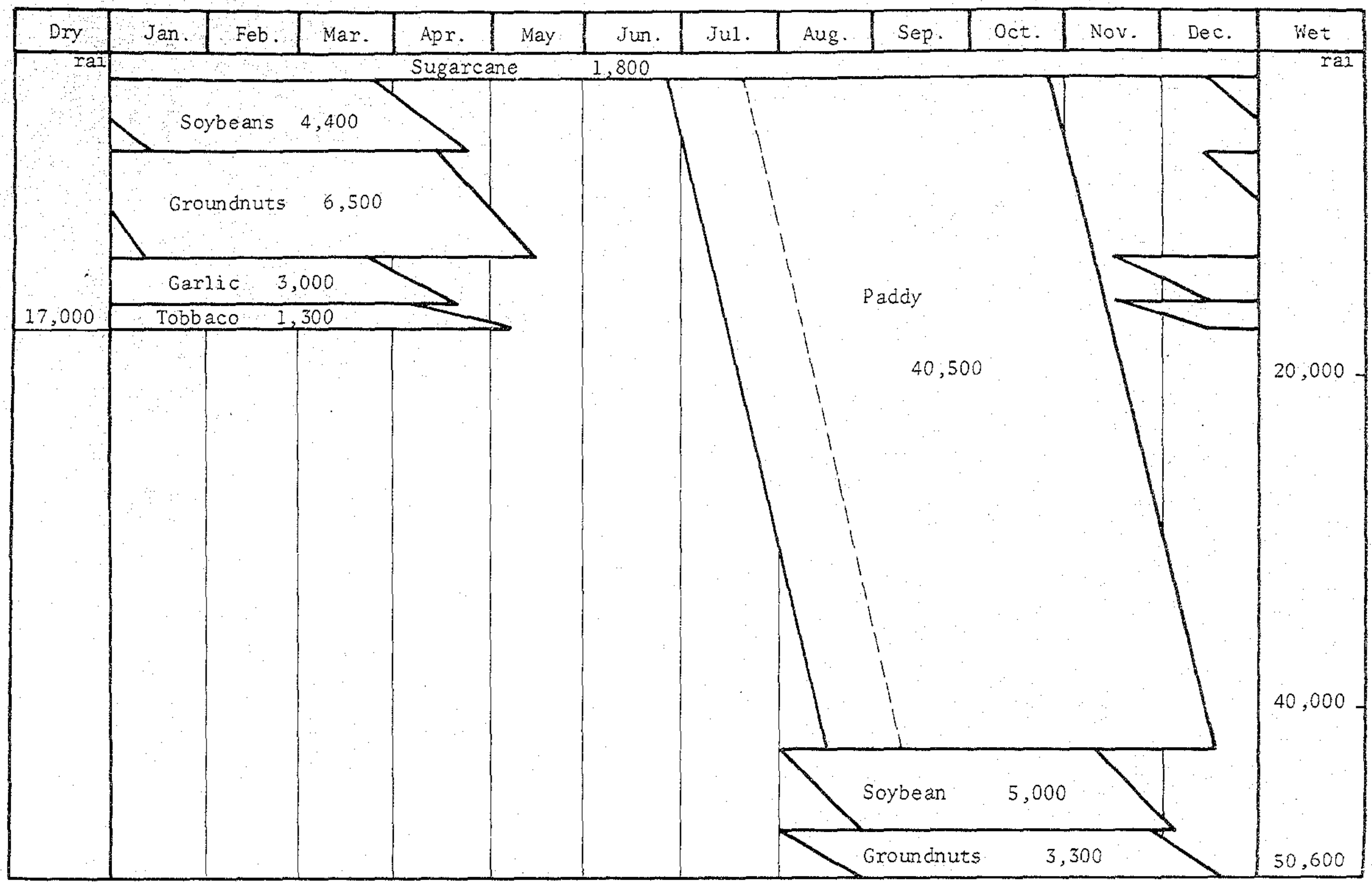


Figure 4.3-2 Cropping Calendar Case 6 (135%)

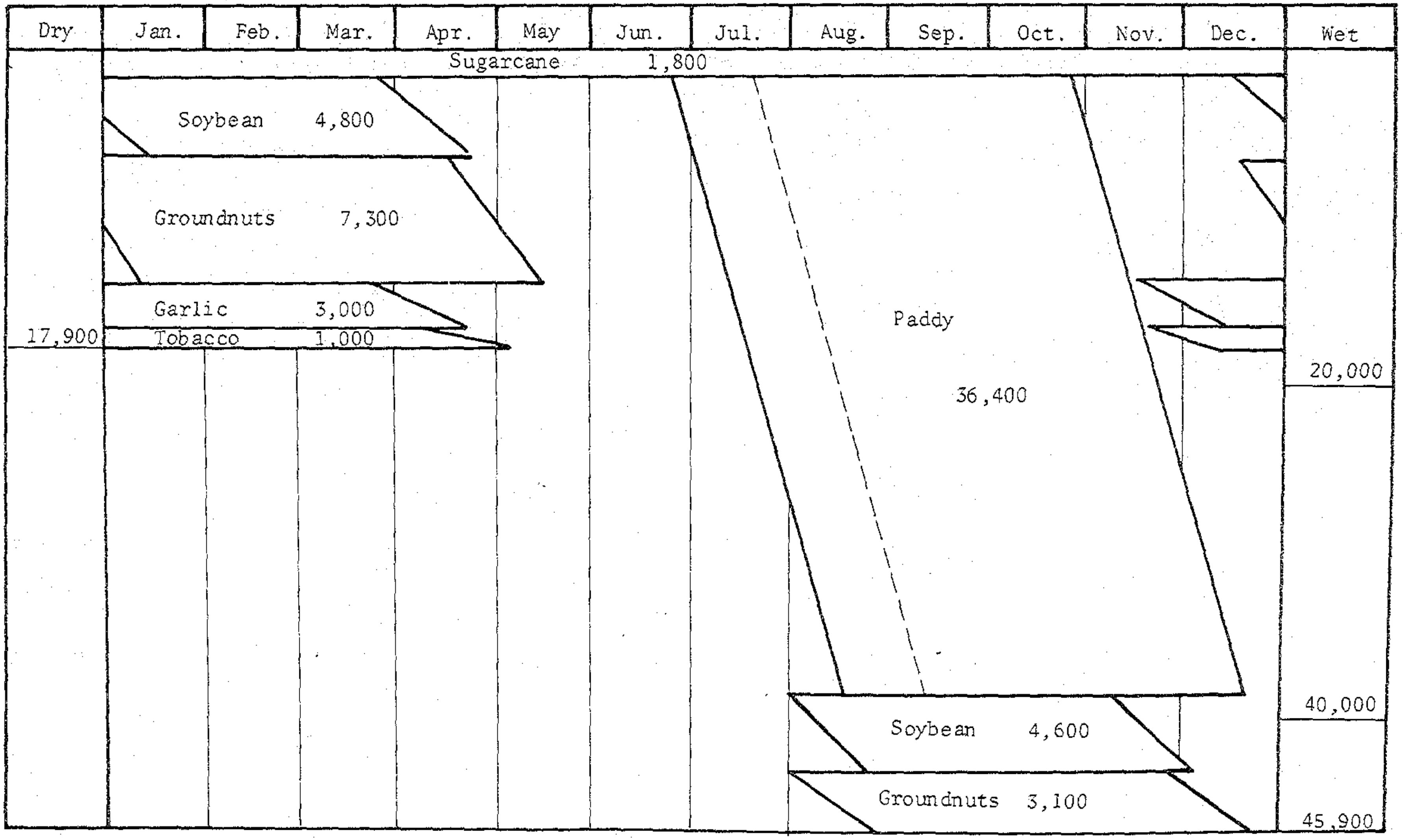


Table 4.3-1 Proposed Agricultural Input Material (per rai)

Item	Paddy	Groundnuts	Soybeans	Tobbaco	Garlic	Sugarcane
Seed	8 kg	12 kg (without shell)	7 kg	2,500 unit	95 kg	1,000 kg (stem.)
Fertilizer	Co.F(16:20:0) 25 kg A.S.(20) 20 kg	Co.F(20:20:0) 30 kg	Co.F(14:24:12) 25 kg	Co.F(4-16-24-4) 80 kg (Mg) Co.F(13-0-46) 20 kg	Co.F(13:13:21) 100 kg	Co.F(13-13-21) 100 kg
Pesticide	Furadan 4 kg Kinsuma 80 g Kasumin 200 cc	Methomyl 0.05% 100 cc Methomyl 100 cc	Methomyl 0.05% 100 cc Lannate 50 g	Furadan 6 kg Carmitron 60 cc	Toku Thion 60 cc Azinmag 45 g	-
Paddy Straw	-	-	-	-	Mulching 1.5 t	-
Labor (man/day)	24.5	35.7	19.8	42.0	25.7	22.1 (32.4) (16.9)1/

Note: 1/ 32.4 new planting
16.9 ratoon

Table 4.3-2 Proposed Agricultural Input by Crop and Case

Item	Case	Paddy	Groundnuts	Soybeans	Tobacco	Garlic	Sugarcane	Total
					(1,000 unit)			
Seed (t)	5	324	118	66	3,250	285	1,800	2,593 3,250 1,000 unit
	6	291	143	55	2,500	285	1,800	2,574 3,250 1,000 unit
Fertilizer (t)	5	1,823	294	235	33	300	180	2,865
	6	1,638	357	198	25	300	180	2,698
Pesticide (t)	5	173	2	1.4	0.2	0.3	-	176.9
	6	156	2.4	1.2	0.2	0.3	-	160.1
Paddy Straw (t)	5	-	-	-	-	4,500	-	4,500
	6	-	-	-	-	4,500	-	4,500
Labor (man/day)	5	993,292	228,414	176,632	54,549	112,511	42,223	1,607,621
	6	892,736	276,010	150,588	41,961	112,511	42,223	1,516,029
Animal	5	124,232	32,346	22,833	5,521	10,684	3,715	199,331
Labor (animal/day)	6	892,736	39,279	19,215	4,247	10,684	3,715	188,794

Table 4.3-3 Proposed Labor Requirement Case 5 (130%)

(Unit: animal/day)

Crops	Area rai	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Total
		26 1/	24	26	26	26	26	22	21	20	20	26	26	289
Paddy	40,500						6,295	74,705	184,958	185,328	52,303	262,903	226,800	993,292
Sugarcane	1,800													
Ratoon	1,200	2,194	5,074	5,349	3,264	1,015	878	878	823	411	411	823	823	21,943
New Plant	600	1,097	2,537	2,674	2,159	9,290	439	439	411	206	206	411	411	20,280
Soybeans	9,400													
W.S. 2/	5,000							2,377	18,457	9,943	12,229	26,400	20,000	89,406
D.S.	4,400	13,678	11,666	18,002	23,634							4,667	15,579	87,226
Groundnuts	9,800													
W.S.	3,300	754						2,187	8,478	3,696	5,657	18,631	32,510	71,913
D.S.	6,500	18,794	15,303	10,400	50,956	47,989							13,059	156,501
Garlic	3,000	8,571	7,200	17,897	28,457							23,787	26,599	112,511
Tobacco	1,300	9,301	13.163	5,794								11,701	14,590	54,549
Total	65,800	54,389	54,943	60,116	108,470	58,294	7,612	80,586	213,127	199,584	70,806	349,323	350,371	1,607,621
Ava. L. 3/		457,829	422,611	457,829	457,829	457,829	457,829	387,394	369,785	352,176	352,176	457,829	457,829	5,088,945

Note) 1/ working days 2/ W.S. : Wet Season 3/ Available Labor
D.S. : Dry Season

Table 4.3-4 Proposed Animal Power Requirement Case 5 (130%)

(Unit : man/day)

Crops	Area rai	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Total
		26 1/	24	26	26	26	26	22	21	20	20	26	26	289
Paddy	40,500						1,944	30,456	46,471	4,629	926	19,440	20,366	124,232
Sugarcane	1,800													
Ratoon	1,200	165	494	521	313	44	41	41	27	27	27	27	27	1,754
New plant	600	82	247	261	683	576	21	21	14	14	14	14	14	1,961
Soybeans	9,400													
W.S. 2/	5,000							2,377	6,971	114	114	1,257	1,257	12,090
D.S.	4,400	101	101	805	1,408							3,993	4,335	10,743
Groundnuts	9,800													
W.S.	3,300							1,946	4,118	75	75	1,584	3,093	10,891
D.S.	6,500	4,130	149	149	4,606	4,606							7,815	21,455
Garlic	3,000	69	69	1,440	2,880							3,079	3,147	10,684
Tobacco	1,300	386	921	565								2,181	1,468	5,521
(1) Total	65,800	4,933	1,981	3,741	9,890	5,226	2,006	34,841	57,601	4,859	1,156	31,575	41,522	199,331
(2) Ava. L.F. 3/		130,156	120,144	130,156	130,156	130,156	130,156	110,132	105,126	100,120	100,120	130,156	130,156	1,446,734

Note: 1/ working days 2/ W.S. : Wet Season D.S. : Dry Season 3/ Available Labor Force

Table 4.3-5 Proposed Labor Requirement Case 6 (135%)

(Unit: man/day)

Crops	Area	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Total
	rai	26 1/	24	26	26	26	26	22	21	20	20	26	26	289
Paddy	36,400						5,658	67,142	166,234	166,566	47,008	236,288	203,840	892,736
Sugarcane	1,800													
Ratoon	1,200	2,194	5,074	5,349	3,264	1,015	878	878	823	411	411	823	823	21,943
New plant	600	1,097	2,537	2,674	2,159	9,290	439	439	411	206	206	411	411	20,280
Soybeans	7,900													
W.S. 2/	4,600							2,187	16,980	9,148	11,250	24,288	18,400	82,253
D.S.	4,800	14,921	12,727	19,639	25,783							5,091	16,995	95,156
Groundnuts	11,900													
W.S.	3,100	708						2,055	7,964	3,472	5,314	17,502	30,540	67,555
D.S.	7,300	21,107	17,186	11,680	57,232	53,895							14,667	175,767
Garlic	3,000	8,571	7,200	17,897	28,457							23,787	26,599	112,511
Tobacco	1,000	7,154	10,126	4,457								9,001	11,223	41,961
(1) Total	62,000	55,752	54,850	61,696	116,895	64,200	6,975	72,701	192,412	179,803	64,189	317,191	323,498	1,510,162
(2) Ava. L.F. 3/		419,375	387,115	419,375	419,375	419,375	419,375	354,856	338,726	322,596	322,596	419,375	419,375	4,661,514

Note) 1/ working days 2/ W.S. : Wet Season D.S. : Dry Season 3/ Available Labor Force

Table 4.3-6 Proposed Animal Power Requirement Case 6 (135%)

(Unit: animal/day)

Crops	Area rai	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Total
		26 1/	24	26	26	26	26	22	21	20	20	26	26	289
Paddy	36,400						1,747	27,373	41,766	4,160	832	17,472	18,304	111,654
Sugarcane	1,800													
Ratoon	1,200	165	494	521	313	44	41	41	27	27	27	27	27	1,754
New plant	600	82	247	261	683	576	21	21	14	14	14	14	14	1,961
Soybean	7,900													
W.S. 2/	4,600							2,187	6,413	105	105	1,156	1,156	11,122
D.S.	4,800	110	110	878	1,536							4,356	4,729	11,719
Groundnuts	11,900													
W.S.	3,100							1,828	3,869	71	71	1,488	2,905	10,232
D.S.	7,300	4,639	167	167	5,173	5,173							8,777	24,096
Garlic	3,000	69	69	1,440	2,880							3,079	3,147	10,684
Tobacco	1,000	297	709	434								1,678	1,129	4,247
(1) Total	62,000	5,362	1,796	3,701	10,585	5,793	1,809	31,450	52,089	4,377	1,049	29,270	40,188	187,469
(2) Ava. L.F. 3/		130,156	120,144	130,156	130,156	130,156	130,156	110,132	105,126	100,120	100,120	130,156	130,156	1,446,734

Note) 1/ working days 2/ W.S. : Wet Season
D.S. : Dry Season 3/ Available Labor Force

Table 4.3-7 Target Yield per rai

Crops	Condition	Yield (kg)
Paddy		
Non glutinous Rice	air dry	660
Glutinous Rice	air dry	580
Soybean		
Wet Season	air dry	270
Dry Season	air dry	300
Groundnuts		
Wet Season	air dry with shell	250
Dry Season	air dry with shell	300
Sugarcane	fresh	8,000
Garlic	air dry	700
Tobacco	fresh	2,600

Table 4.3-8 Cropping Area Case 5 (130%)

	rai	ha
Wet Season		
Paddy	40,500	6,480
	N.G. 24,300	
	G. 16,200	
Upland	10,100	1,615
Sugarcane	1,800	290
Soybean	5,000	795
Groundnuts	3,300	530
Sub-total	50,600	8,095
Dry Season		
Soybean	4,400	700
Groundnuts	6,500	1,050
Sugarcane	(1,800)	(290)
Garlic	3,000	480
Tobacco	1,300	200
Sub-total	15,200	2,430
Total	65,800	10,525

Table 4.3-9 Present and Proposed Crop Production Case 5 (130%)

Crops		Present			Proposed without Project			Proposed with Project		
		Area (rai)	Yield/rai (kg)	Production (t)	Area (rai)	Yield/rai (kg)	Production (t)	Area (rai)	Yield/rai (kg)	Production (t)
Paddy	G.	31,350	276	8,653	31,350	290	9,092	16,200	580	9,396
	N.G.	-	-	-	-	-	-	24,300	660	16,038
Sugarcane		3,200	2,436	7,795	3,200	2,560	8,192	1,800	8,000	14,400
Soybean	W	-	-	-	-	-	-	5,000	270	1,350
	D	830	160	133	830	170	141	4,400	300	1,320
Groundnuts	W	1,200	160	192	1,200	170	204	3,300	250	825
	D	2,300	200	460	2,300	210	483	6,500	300	1,950
Maize	W	460	306	141	460	320	147	-	-	-
Upland Rice	W	1,700	268	456	1,700	280	476	-	-	-
Mangbean etc.	W	240	210	50	240	220	53	-	-	-
Garlic	D	-	350	-	-	-	-	3,000	700	2,100
Tobacco	D	850	2,000	1,700	850	2,100	1,785	1,300	2,600	3,380
Others	D	240	117	28	240	123	30	-	-	-
Total		42,370	-	19,608	42,370	-	20,603	65,800	-	50,759

G. : Glutinous rice W.: Wet season
N.G.: Non glutinous rice D.: Dry season

Table 4.3-10 Cropping Area Case 6 (135%)

	rai	ha
Wet Season		
Paddy	36,400	5,819
(N.G.	21,800	
(G.	14,600	
Upland		1,530
Sugarcane	1,800	290
Soybean	4,600	740
Groundnuts	3,100	500
Sub-total	45,900	7,349
Dry Season		
Soybean	4,800	770
Groundnuts	7,300	1,160
Sugarcane	(1,800)	(290)
Garlic	3,000	480
Tobacco	1,000	160
Sub-total	16,050	2,570
Total	62,000	9,919

Table 4.3-11 Present and Proposed Crop Production Case 6 (135%)

Crops		Present			Proposed without Project			Proposed with Project		
		Area (rai)	Yield/rai (kg)	Production (t)	Area (rai)	Yield/rai (kg)	Production (t)	Area (rai)	Yiedl/rai (kg)	Production (t)
Paddy	G	26,900	276	7,424	26,900	290	7,801	14,600	580	8,468
	N.G.	-	-			-		21,800	660	14,388
Sugarcane		3,000	2,436	7,308	3,000	2,560	7,680	1,800	8,000	14,400
Soybean	W		-			-		4,600	270	1,242
	D	780	160	125	780	170	133	4,800	300	1,440
Groundnuts	W	1,100	160	176	1,100	170	187	3,100	250	775
	D	2,150	200	430	2,150	210	452	7,300	300	2,190
Maize	W	430	306	132	430	320	138	-	-	-
Upland Rice	W	1,600	268	429	1,600	280	448	-	-	-
Mangbean etc.	W	220	210	46	220	220	48	-	-	-
Garlic	D	-	-	-	-	-	-	3,000	700	2,100
Tobacco	D	800	2,000	1,600	800	2,100	1,680	1,000	2,600	2,600
Others	D	220	117	26	220	123	27	-	-	-
Total		37,200	-	17,696	37,200	-	18,594	62,000	-	47,573
										47603

G. : Glutinous rice W.: Wet season
N.G.: Non glutinous rice D.: Dry season

Livestock farming is planned by the same method applied to the program for increasing beef cattle as shown in Appendix 4.3-1.

	Annual Breeding (Number)	Annual Delivery (Number)
Case 5	6,545	1,182
Case 6	5,832	1,053

4.3.2. Plan for Marketing and Processing of Farm Products

1) Marketing of Agricultural Products

When the Project reaches full development in 1996 agricultural production is expected to increase as follows:

(Unit: ton)

Item	"Without" Project Case-5	"Without" Project Case-6	"With" Project Case-5	"With" Project Case-6
Paddy				
Glutinous	9,092	7,081	9,396	8,468
Non-Glutinous	-	-	16,038	14,388
Upland Rice	476	448	-	-
Sugarcane	8,192	7,680	14,400	14,400
Groundnuts	687	639	2,775	3,340
Maize	147	138	-	-
Mungbean, etc.	53	48	-	-
Soybeans	141	133	2,670	2,277
Tobacco	1,785	1,680	3,380	2,600
Garlic	-	-	2,100	2,100
Others	30	27	-	-
Total	20,603	18,594	50,759	47,573

This means that total agricultural production will increase approximately 2.5 times. According to merchants and middlemen intervived during the field survey such an increase would not create any difficulties for the present marketing channels. All surplus agricultural production will be marketed as follows:

i) Paddy - it is foreseen that demand for the crop for domestic consumption and for export will continue to be much greater than production. Also, higher yields resulting from implementation of the Project will make the marketing of this crop more profitable for the farmers. Major purchasers of this crop are cooperatives, middlemen and millers.

ii) Sugarcane - There are two sugar refineries located in Lampang Province. All sugarcane grown in the Project Area is contracted to be purchased before planting. Therefore, there is no problem in the marketing of this crop.

iii) Groundnuts and Soybeans: Primary processors such as drying, sorting and shelling companies contacted during the field surveys were very enthusiastic about the increases in production of these crops that the Project would bring about. There are appropximately 30 such companies operating in Lampang Province and almost all are operating at less than full capacity. Farmers should face not difficulties in marketing these crop.

iv) Tobacco: This crop is also contracted to be purchased before planting. Farmers are required to contact a curing station which in turn supplies all necessary inputs for proper cultivation. Therefore, no problem is foreseen in the marketing of this crop.

v) Garlic: This crop is mainly sold at local markets or to middlemen in Lampang City. It is a very popular ingredient in Thai food and the demand is always high. In times of shortage in production it is freely imported, therefore, no difficulties are foreseen in the marketing of this crop.

2) Processing of Agricultural Products

i) Paddy - Rice mills inside and nearby the Project Area normally operate a maximum of 12 hours per day during peak periods. Therefore, processing the increase in paddy production should not create any problems even after full development.

ii) Sugarcane - The Lampang Sugar Factory is capable of processing the present output and is now considering expansion of its processing capacity.

iii) Groundnuts and Soybeans: Primary processing before forwarding to brokers can be carried out in Lampang Province.

iv) Tobacco - Curing stations are located nearby areas which are most suitable for growing tobacco. After curing the product is sold to redrying milk located in Chiang Mai.

4.3.3. Farm Management Plan

A farm management plan has been prepared based on interviews conducted and observations made during the field surveys and the socio-economic survey carried out by the Royal Irrigation Department inside the Project Area.

1) Land Holding Size

Farm sizes "With" Project are planned as follows; small scale-5 rai (0.8 ha), medium scale-8 rai (1.28 ha) and large scale-12 rai (1.92 ha).

2) Cropping System

Cropping systems and production amounts of each proposed farm-size are shown in Tables 4.3-8 and 4.3-9. The main crop to be cultivated will be paddy. Upland crops will consist of soybeans, groundnuts, garlic and tobacco.

3) Farm mechanization and Farm Labor Balance

There is an abundant supply of farm labour in the Project Area. Also, it is expected that all land preparation in the future will continue to be done by animal power. Therefore, the introduction of agricultural machinery is not proposed.

Table 4.3-12. "Without" and "With" Project Cropping Area by Farm-Size

(Unit: rai)

Farm-size	Paddy	Soybeans		Groundnuts		Garlic	Tobacco	Others 2/	Total
	W	W	D 1/	W	D	D	D		
"Without"									
8	6.0	-	-	0.5	-	-	-	1.5	8.0
(Average)									
"With" Project (Case 5)									
5	4.0	0.4		0.6	-	0.5	1.0	-	6.5
8	6.5	0.6	0.8	0.9	1.1	0.6	-	-	10.4
12	10.0	1.0	1.4	1.0	2.0	0.2	-	-	15.6

1/ W: Wet season 2/ others: Sugarcane, Mungbean, Maize, etc.
D: Dry season

Source: Present, Report on Farmers' Socio-Economic Survey inside Mae Chang Reservoir Project Boundary RID 1980/81

Table 4.3-13. "Without" and "With" Project Agricultural Production by Farm-Size

(Unit: kg)

Farm-Size (rai)	Paddy	Soybeans	Groundnuts	Garlic	Tobacco	Others
"Without"						
8	1,740	-	80	-	-	1,323
(Average)						
"With" Project (Case 5)						
5	2,512	108	150	350	2,600	-
8	4,082	302	555	420	-	-
12	6,280	690	850	140	-	-

4.3.4. Agricultural Supporting Institution

1) Extension Services

Extension services for on farm techniques have been remarkably promoted due to a strengthening of the agricultural extension office. In this Project, paddy production is estimated at three times of the present production after completion of the Project. At present, most of the farm households in the Area have little experience in dry season cultivation.

This Project is an epoch making event for the Area where the existing agriculture, characterized by rain-fed cultivation is reconverted to irrigated agriculture by well organized and managed systems. Here, all farm practises will be carried out in accordance with these newly established systems. The target yield will be performed by combining new farm techniques with such irrigation systems.

The existing extension services have been made focusing upon paddy, therefore, soybean and groundnut cultivation in the dry season are new crops for farmers and even for the agricultural extension office. In order to smoothly introduce such crops, education and an increase of extension workers is strongly required. Fortunately, in the Mae Wang Area which is adjacent to the Project Area irrigation facilities were completed and a demonstration farm provided. Such irrigated agriculture in the Mae Wang Area will make a contribution to extension services and farming techniques.

2) Cooperative Activities

The agricultural cooperatives within the Project Area will require institutional strengthening if they are to fulfill the role which is expected of them. Implementation of the Project will create a need for an increase in the services that the cooperatives are now offering their members. In order for farmers to purchase the agricultural inputs necessary for intensive farming, loans in the form of agricultural credits must be made readily available to all members.

Also, the cooperatives will need to increase marketing activities such as purchasing and selling of farm product and processing of products such as paddy and groundnuts. Purchasing and selling of crops by the cooperatives for their members will also result in better and more stable farm gate prices.

The main cooperative activities which will require institutional strengthening are as follows;

i) Finance - increase in the availability of funds to be used for loans and purchasing of farm products.

ii) Marketing - increase in the purchase of farm products to ensure that members receive a fair price.

iii) Agricultural inputs - stabilizing the availability of farm inputs.

iv) Staff - increase in the number of workers to ensure that all activities can be carried out smoothly.

4.4. Proposed Facilities

4.4.1. Storage Dam A

1) Topography and Geology

a) Topography

The proposed dam sites are located about 20 km south-southeast from Lampang city, in the midstream of the Mae Chang river (upstream about 20 km from the confluence of the Mae Wang river) in the northern part of Thailand. Those areas which are hilly regions just in transition from mountain regions to plain extending from north to southern, are formed into a plain with about 20 km wide and located between mountain ranges 800 to 1,000m in elevation.

Several limestone hills are found in the plain and suggest that the base rock of the plain is limestone.

The Mae Chang river joining several streams flowing from those mountains ranges has comparative steep river bed slope at about 1/600 to 1/700 around the dam sites. The river eroding the plain flows southeast at the Dam Site A into the confluence of the Mae Wang river at Bank Sop Chang after turning east at Dam Site B.

The topography of the proposed dam sites and their vicinity is formed into valleys with 200 to 300 m wid and 40 to 50 m relative height resulting from erosion. The dam sites are proposed for these valleys.

The span-height ratios of each dam site are as follows;

Dam Site A : 1/20
Dam Site C : 1/23
Dam Site D : 1/17

b) Geology

The site is at around EL 300 m and forms a mild hilly topography composed of diluvial terrace deposit. The Mae Chang river meanders considerably with small valleys formed by the erosion of terrace deposits.

Along upper and lower streams of the Dam Site A, alluvial terraces are developed and no outcrops of rock are observed; only some shales and conglomerate are observed downstream and upstream the site as shown Appendix 4.4-1. The site is comparatively advantageous than in the vicinity due to the narrowest river course. The bedrock is formed of mudstone and conglomerate classified into the middle formation of Lampang Group.

Judging from outcrops and boring core at the site, rock surface elevation is 267.5 m at the left bank, 269.2 m at the right abutment, and 252.9 m at the river bed. The surface of the bedrock seems to be almost flat resulting from erosion in the stage of the terrace creation.

The bedrock of the dam site consists mainly of mudstone and interbedding sandstone and conglomerate with mudstone in thin layers. The mudstone is complex with a wide variety of sedimentary faces such as sandy or tuffaceous and partly includes gravels in color also. These bedrocks are not hard and the boring core is easily broken by soft hammering and sections of it were dissolved by the drilling water. General strikes and dips of the bedrocks are N60W - 80E, 15 - 20°S. Due to coverings the fault is unclear but the bearing capacity of bedrock is sufficient for a fill type dam foundation. Permeability of bedrock is generally small indicating a lugeon value of under 20.

Quaternary deposits are widely distributed on the bedrock making a flat plain as lower, middle and higher terrace deposits. The lower and the middle terrace deposits consist of silty sand or sandy silt sometimes including small pebble. It is 3 - 10 thickness and N value 10 - 30.

The higher terrace deposit survey did not disclose the permeability definitely. Judging from outcrop and result of grain analysis using penetration samples, the terrace deposits will show a permeability coefficient of nearly $n \times 10^{-4}$ and some loose sections will be in the order at $n \times 10^{-3}$.

Special attention should be paid to permeability because terrace deposits are distributed on the right band left abutments in the reservoir area also. Therefore further investigation is necessary to make a more detailed geological investigation, especially about permeability and variety of sedimentary faces.

Groundwater table is three to five meters higher than the river water level at the right and left abutment in the dry season.

2) Study of Dam Axis

The dam aixs of this dam site was decided under the consideration of the topographic map surveyed by RID and the following factors.

(i) To select a dam site on a narrow valley

The outline of topography at the dam site A is as follows. The dam site A is located at about 1.0 km NE from Ban Don Mun. The Mae Chang river changes its direction of flow from south to west at about 1.0 km upstream of the dam site, and changes back to a southerly direction at the dam site. Meanders of the Mae Chang river have been considerably developed by erosive action, and the dam site A is the narrowest place. The width of river bed is about 60 m and the longitudinal length is about 500 m between the summit elevation EL.285.00 m of both abutments.

(ii) To select a site which the slopes of both abutments are not steep

Steep abutment cause a differential settlement in the dam. This dam site forms gentle abutments having 1:4.5 to 1:9.0 slope.

(iii) To select the abutments having highest elevation

It is necessary to be high dam in order to obtain a reservoir capacity as large as possible. The summit elevation of both abutments is EL.285.00 m.

(iv) To select favourable topography for the situation of spillway

As the right abutment forms convex topography, it is possible to situate the spillway in a straight line.

3) Embankment Materials

a) Location of Borrow Area and Quarry

The final borrow area and quarry were selected by the Second field survey continued from the first field survey. The borrow area is located at about 2.0 km on the right bank downstream from the dam site. The quarry is located at about 7.0 km NE from the dam site. And it is possible to use materials excavated at the spillway and core trench.

b) Conservative Quantity

It will be able to obtain a more than 50,000 cu.m of impervious material at the borrow area and more than 2 times the quantity of embankment material at the quarry.

c) Quality

(1) Borrow area

A soil test has not been carried out about a impervious materials of the borrow area. However, this material is classified as GM and/or GC by field observation. Generally, a favourable characteristic of impervious material is as follows:

(i) To possess necessary imperviousness, density and shear strength

(ii) To possess an adaptable plasticity against dam deformation

(iii) To have resistance force against a seepage water through embankment

(iv) There has no swell and shrinkage that influence a dam stability

(v) To have a good property for a embankment construction

(vi) No soluble organic materials are included in the fill materials

According to the USBR, GM and GC materials are within the above items.

(2) Materials excavated at the spillway and core trench

These materials are mainly terrace deposits and are classified in GM, SM and SC from the results of grain size analysis. They have properties of impervious materials because the total content of silt and clay is 23 per cent to 35 per cent.

A plasticity index of these impervious materials is range of 7.6 to 13.4. These values show to be a low and/or medium plastic materials because of less than 15. Cracks will occur in the embankment on compacting these materials under the dry condition (less than optimum water content). On the other hand, it is actually impossible to excavate and obtain each materials independently at the spillway and core trench. It is better to avoid use of the important zone like the impervious zone because these excavated materials do not have homogeneous properties.

(3) Quarry

Material tests of rock of quarry were not carried out. The quality is hard limestone and suitable for pervious material (rock material) and concrete aggregate by the field observation. Generally, favourable rock material characteristics are as follows:

(i) To be hard and massive

(ii) To have durability against water and weathering

(iii) To possess a large shear strength

(iv) To be small settlement caused by compression

For reference, from the results of rock tests using by boring core (calcarious sandstone), specific gravity, absorption and soundness are each 2.68, 0.20 per cent and 2.50 per cent. A limestone which will be obtained from the quarry of dam site A has a better quality than the above core.

4) Dam Type

In general, the dam is mainly classified into two types, that is, the fill dam and concrete dam.

a) Suitability of Concrete Dam

A concrete dam is not suitable at the dam site A for the following reasons;

(i) The profile along the center of the dam is an inverted trapezoid and the L/H ratio is 20. It is too wide to construct a concrete dam.

(ii) According to the geology of dam site, the bedrock is formed of mudstone and conglomerate into the middle formation of Lampang Group. These bedrocks are not hard and the boring core is easily broken by soft hammering with sections of it were dissolved by the drilling water. The bearing capacity of bedrock is not sufficient for a concrete dam. And also the surface of bedrock of both abutments seems to be almost flat. That is to say, quaternary deposits on the bedrock form a layer which is too thick with a N-Value in the 10 to 20 range. The shear resistance force of this foundation is too small for a concrete dam.

(iii) Geological condition cause the trench excavation of a concrete dam to be more than a fill dam

b) Selection of a Fill Dam Type

A type of fill dam is mainly classified into 3 types, that is, the homogeneous, zone type and dams with impervious membranes of reinforced concrete and asphaltic concrete, etc. The zone type of earth fill dam was adopted by the following reasons;

(i) Dams with artificial impervious membranes are expensive and have difficulty in their against construction. And embankment material is rock.

(ii) A homogenous dam is a suitable type because the borrow area is in vicinity of the dam site. However, it is more economical to use materials excavated at the spillway and core trench as a part of the dam in order to decrease the construction cost. As a result, fill dams become zone type earth fill dams similar to homogeneous dams because the majority of the dam body is composed of the above two materials.

A zoning is as follows:

(i) The impervious zone should be arranged on the center of dam depend on topography. The size of the impervious zone was based on the water pressure, quality of fill materials, width of the foundation treatment at the core trench and ratio between materials excavated at the dam site and material in the borrow area.

(ii) The materials excavated at dam site should be arranged on both sides of the impervious zone as semi-pervious zone considering dam stability.

(iii) The pervious zone should be arranged on the both sides for slope protection.

The typical cross section of dam A is shown on the attached Drawing No.2.

5) Dam Dimensions

a) Dam Crest Elevation

Dam crest elevation of the dam A is considered to be 285.00 m as a maximum elevation on a topographical map in order to obtain a reservoir capacity as much as possible.

b) Freeboard

The fill dam should not be overtopped by the cause of the waves in a reservoir generated by winds and earthquakes. In case of the uncontrolled spillway (spillway without gate), the overflow water depth (Hd) can be decided by hydrological analysis of the spillway's function as planned. Therefore, the freeboard is defined as a difference between the dam crest elevation and the high water level (FWL + Hd).

According to the climatological data at Changwat lampang for 30 year period, the average monthly wind velocity is not more than 2.0 m/sec. And also the height of waves generated by wind will be very low because of this wind velocity. The height of waves generated by earthquake will also be low.

In this study, the freeboard was decided in taking into consideration examples of fill dams constructed in Thailand. The freeboard of fill dams in Thailand is designed at 2.0 to 3.0 m and the range for dam at 30 to 50 m height. Therefore, the freeboard of dam A is planned at 2.0 m.

c) High Water Level and Full Water Level

The dam crest elevation usually can be determined by the following equation.

Dam crest EL = FWL + Hd + Fb

Where,

FWL : Full water level

Hd : Overflow water depth on spillway

Fb : Freeboard

In the above equation, the dam crest elevation was already fixed by the topography and the freeboard was also decided. An overflow water depth on the spillway (Hd) is calculated by 2.80 m based on hydrological analysis. So that, high water level (FWL + Hd) becomes EL 283.00 m. And full water level gets EL 280.20 m.

d) Width of Dam Crest and Embankment Side Slopes

A width of dam crest adopts 8.0 m in accordance with the standards of fill dams in Thailand. Embankment side slopes apply 1:3.0 for the upstream slope and 1:2.5 for the downstream slope from the results of stability analysis.

As a conclusion, the principal features of the storage dam A are summarized below.

<u>Reservoir</u>

Catchment Area	:	403	sq.km
High Water Level	:	EL 283.00	m
Full Water Level	:	EL 280.20	m
Intake Water Level	:	EL 272.20	m

Gross Reservoir Capacity	:	40	MCM
Usable Water Depth	:	8.0	m

Dam

Dam Height	:	35	m
Dam Crest Elevation	:	EL 285.00	m
Min. Trench Elevation	:	EL 250.00	m
Crest Length	:	470	m
Dam Volume	:	682,000	cu.m
Slopes Upstream	:	1:3.0	
Downstream	:	1:2.5	

6) Stability Analysis

a) Method of Stability Analysis

Stability of the dam was studied by using the slice method to the slip circle surface taking into consideration the property of embankment materials and condition of dam foundation.

The dam shall be safe against sliding failure under the following conditions:

Condition 1: At the end of dam construction

Condition 2: Reservoir is at high water level and seepage is steady

Condition 3: Reservoir is at full water level and seepage is steady

Condition 4: Reservoir is at intermediate water level and seepage is steady

Condition 5: Reservoir at rapid drawdown from full water level to low water level

Notes: For conditions 4 and 5, stability analyses are conducted on the upstream slope only.

The safety factor against the sliding surface method is defined at the ratio of sliding moment resisting moment acting on the slip surface. The safety factor for these conditions is obtained by the following formula.

$$SF = \frac{\Sigma\,(c.L + (N-U-Ne) \times \tan \phi)}{(T + Te)}$$

Where,

SF: Safety factor

N : Normal force acting on slip circle of each slice

T : Tangential force acting on slip circle of each slice

U : Pore pressure acting on slip circle of each slice

Ne: Normal force of earthquake load acting on slip circle of each slice

Te: Tangential force of earthquake load acting on slip circle of each slice

ϕ : Angle of internal friction of materials on slip circle of each slice

C : Cohesion of materials on slip circle of each slice

L : Arc length of slip circle of each slice

The safety factor shall not be less than 1.2 in any conditions.

b) Design Values

Design values to be used for stability analysis of the dam were determined based on the results of soil test and referred to the other dams. The design values are as follows:

Design Values of the Storage Dam A

Materials or Name of Zone	Wet Density (rt) (t/m^3)	Saturated Density $(rsat)$ (t/m^3)	Cohesion (c) (t/m^2)	Internal Angle (ϕ) (°)
Impervious material (Zone 1)	2.10	2.19	4.0	15°-00'
Semi-pervious material (Zone 2)	2.10	2.19	4.0	15°-00'
Filter material (Zone 3)	2.00	2.10	0	35°-00'
Pervious material (Zone 4)	2.00	2.10	0	38°-00'

Note: * The design value of impervious material (Zone 1) was applied the one of semi-pervious material (Zone 2) because soil tests of impervious materials have been not performed.

* The design density of semi-pervious material (Zone 2) was applied at 95 per cent value of the maximum dry density by compaction test taking consideration into the property of materials and construction condition. The shear strength of impervious materials was obtained from a triaxial compressive strength test under the two kinds of conditions. And the design value of shear strength (cohesion and internal friction) was determined by the combination of minimum value of the two shear strength.

* The design values of filter and pervious materials were decided referred to the Mae Kuang Dam.

c) Results of Stability Analysis

The stability analysis of dam was performed by the computer under the above conditions. The results are as follows:

Conditions	Earthquake Force	Water Level	Slope	Safety Factor
1	k = 0.05	-	Upstream Downstream	1.391 1.395
2	k = 0.05	HWL 283.00	Upstream Downstream	1.486 1.396
3	k = 0.05	FWL 280.20	Upstream Downstream	1.419 1.395
4	k = 0.05	MWL 276.20 LWL 272.20	Upstream Upstream	1.327 1.270
5	k = 0.05	FWL to LWW	Upstream	1.248

The contour of safety factor of each conditions are shown in Fig. 4.4-27 in the Appendix 4.4-3.

7) Spillway

<u>Type and rout of spillway</u>

The most appropriate location of the proposed spillway will be on the left bank terrace in taking into consideration the topographical conditions on both the left and the right abutments. The right bank is so steep in topography that the total spillway structure would have to be longer.

The type of spillway to the proposed dam was determined as an uncontrolled spillway (spillway without gate) for the following reasons.

For providing gated type spillways:

° Sufficient knowledge is required on the specific features of the flood discharges in the past,

- a thorough study is necessary for operation and maintenance services of the facilities in due consideration of floods caused by mishandling of the facilities in the downstream, and

- the spillway facilities of any suitable scale can be constructed independently from the dam body in view of the topo-conditions.

For providing uncontrolled spillways;

- The construction cost is relatively in expensive and the operation and maintenance services can be easily made.

A comparative study of a dissipator to be provided downstream from the proposed uncontrolled spillway has been made between the hydraulic jump type and the deflector bucket type and the latter has been selected in due consideration of economy and topographical and geographical conditions at the site.

Dimensions of spillway

The designed flood discharge for the spillway of the main dam was decided by taking the maximum overflow discharge which would take place in uncontrolled storage of the maximum flood discharge with a peak flood of 1,600 m^3/s as mentioned before.

Such flood discharge, being in a functional relationship with the weir length, has been studied on its balance for various lengths of the weir.

To decide the capacity of the spillway the calculation has been carried out by using the following formula considering the relationship between the capacity of spillway and the surcharge storage routing.

$$1/1\ (I_1 + I_2) \cdot \Delta\ t + S_1 - 1/2\ 0_1 \quad t = S_2 + 1/2,\ 0_2 \cdot \Delta\ t$$

Where, I_1 ... inflow discharge at t_1 hour in cu.m/sec.

I_2 ... inflow discharge at t_2 hour in cu.m/sec.

0_1 ... outflow discharge at t_1 hour in cu.m/sec.

0_2 ... outflow discharge t_2 hour in cu.m/sec.

S_1 ... storage volume at t_1 hour in cu.m/sec.

S_2 ... storage volume at t_2 hour in cu.m.

$$\Delta t = t_2 - t_1\ ,\ (t_2 > t_1)$$

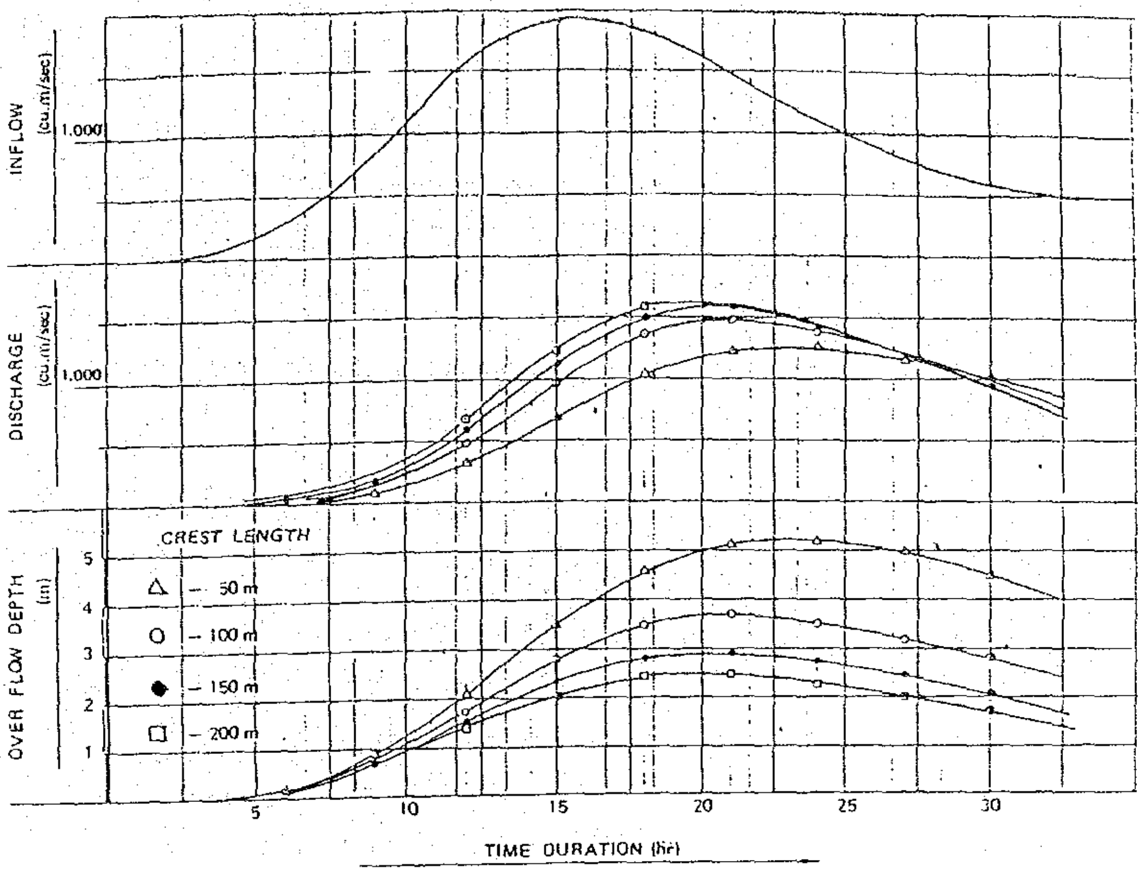
The following are the dimensions of the storage reservoir and the spill with various weir lengths obtained by the above equation for the design flood of 1,660 m^3/s. (See Appendix 4.4-4.)

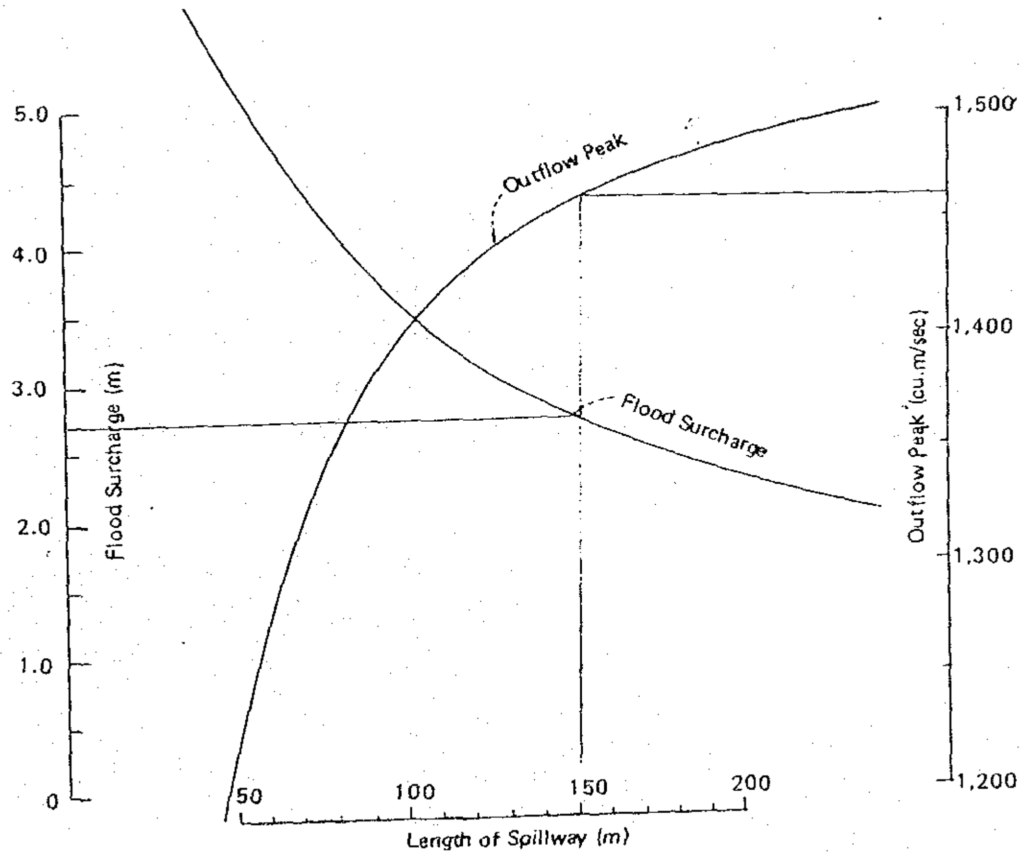
Crest Length (m)	Reservoir Capacity (MCM)	FWL (m)	Overflow Depth (m)	Maximum Spillway Discharge (cu.m/s)
50	30	277.86	5.14	1,222
100	37	279.46	3.54	1,398
150	41	280.23	2.77	1,455
200	43	280.68	2.32	1,480

When a longer weir is provided much more water can be utilized in decreasing in the overflow depth; however, the weir length has been finally determined by 150 m in taking into account the topography of the spillway site and the size of embankment.

Designed flood discharge (1,000 year return period)	1,660 m^3/s
Max. spillway discharge	1,455 m^3/s
Designed flood water level	EL 283.00
Crest elevation	EL 280.20
Overflow depth	2.80 m

Fig. 4.4-1 Spillway Discharge with Each Weir Length





Structural layout

The spillway to be constructed at the right abutment will be provided with a 150m-long overflow weir, and the flood discharges, after flowing through a tail channel, will be introduced into a 70 m-wide chute, from which the water shall jump and fall by a deflector bucket provided at the terminal of the spillway. A water pool to receive falling water shall be constructed as dissipator immediately downstream of the spillway.

The spillway is constructed with reinforced concrete, and the tail channel shall be of trapezoidal cross-section with side slope at the slope of 20 per cent for easy construction of a cross-bridge and for concrete economy.

The bucket requires harder foundation rock and will be provided by 10 to 15 m excavations from the original ground surface; however, the boring survey for the site is essentially required for successful implementation of the works.

Hydraulic Dimensions

Entrance Channel and Control Structures

The approach channel should be provided at the elevation of EL 278.0 m for securing reasonable flow velocity by hydraulically stable water surface (less than V = 4 m/sec) overflow portions of weirs commonly apply either of Harold's curve or the second order parabola in form, and the proposed overflow portion shall take second order parabola to cope with hightening and enlargement of the structures in the future.

The discharge capacity of the overflow weir can be expressed by the following equation and can cover the maximum flood discharge of 1,464 m^3/sec.

$Q = C\ L\ H^{3/2}$

$= 1{,}476\ m^3/sec$

Where, Q ... discharge (m^3/sec)

C ... coefficient of discharge and adopted by 2.1

L ... effective length of weir and adopted by 150 m

H ... overflow depth at crest and adopted by 2.8 m

Spillway canal and chute

The spillway canal width shall be 70 m in the consideration of the effects to the overflow weir and the dissipator, and the present river width. The canal slope shall be 1/500 for securing a steady flow. Under such conditions, the critical flow will takes at the beginning of the rapid flow and the relevant water depth can be

$$dc = 0.467\ q^{2/3}$$

where, dc ...... Critical water depth (m)

q ...... Discharge per unit width (m^3/sec)

$= 3.56$ m

According to the above, hydraulic profile computation of the water surface has been made by the following method.

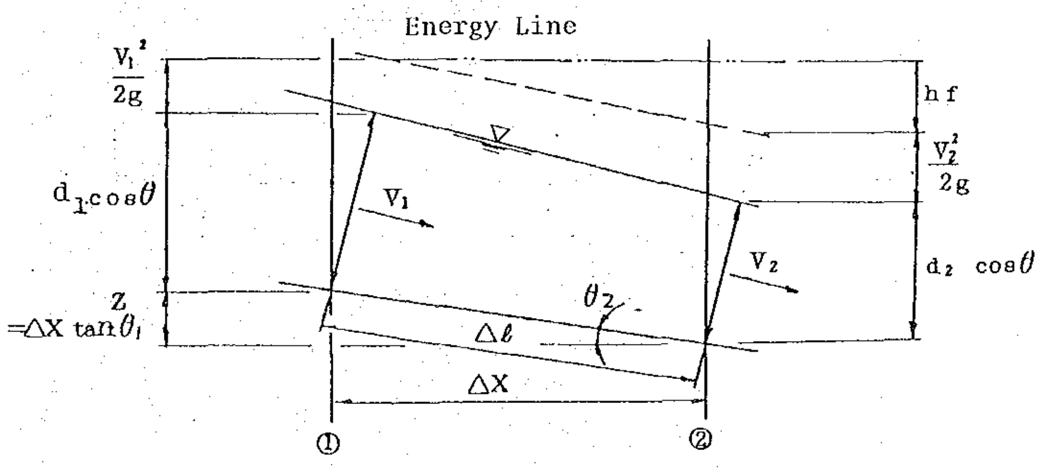


Fig. 4.4-2 Hydraulic Computation Chart

When applying the vernouill's Theory to the above illustration,

$$d_1 \ COS\theta + \frac{V_1^2}{2g} + Z = d_2 \ COS\theta + \frac{V_2^2}{2g} + hr$$

$$\text{where, } hf = \frac{n^2 \ Vm^2}{Rm^{4/3}} \times \Delta \ell$$

Vm. Rm: Average flow velocities for both profiles, average water depth

n: Roughness Coefficient

When the water elevation difference between the up- and down-stream of the portion is taken by h, the above equation will be.

$$h = d_1 \ COS\theta + Z - d_2 \ COS\theta$$

$$h = \frac{V_2^2}{2g} + hf - \frac{V_1^2}{2g}$$

The above equation shall be solved for the values of d_2 and V_2.

The results of calculation of water depth on the chute section are shown in the following table:

Elevation of Floor (EL m)	Water Depth (m)	Velocity (m/sec)	
277.80	3.54	5.81	STA 1 + 0.00 (chute)
275.72	2.11	9.92	STA 1 + 9.00
272.12	1.57	13.34	STA 1 + 18.00
268.52	1.32	15.86	STA 1 + 27.00
265.00	1.17	17.90	STA 1 + 35.82 (chute)

Terminal structure

Where the spillway discharge may be delivered and safely directed to the river without providing a dissipating or stilling device, the jet often projected beyond the structure by a deflector bucket and leaves the structure as a free discharging upturned jet and falls into the stream channel at a certain distance from the end of the spillway.

The horizontal range of the level of the lip is obtained by the equation;

$$x = 2\ K\ (d + hv)\ \sin 2\theta$$

Where,

θ = the angle of the edge of the lip with horizontal, and

k = a factor, equal to 1 for the theoretical jet (taken by 0.9 in this Case)

d = the depth of flow

hv = velocity head

When solving the above equation for the respective values of , the solutions can be found in a range from 24 m to 27 m in Case of = 30° and from 18 m to 20 m in Case of = 20°, and the falling point shall be taken in a range from 24 m to 27 m in Case of = 30° in consideration for the falling point to be located possibly for from the bucket.

8) Outlet

Outlet facilities are so designed as to satisfy demand of irrigation water as well as assumed floods during the construction of the dam in the dry season and the hydro-power plant to be constructed in the future.

The maximum discharge to be released for irrigation is less than 10 m^3/sec. As for the waterway during the construction of the dam in the dry season, 20 to 30 m^3/sec of capacity which correspond to five to 10 years flood in the dry season is required, so that it is convenient to set the conduit at a lower position. Elevation of the rock on the left abutment will be EL 260.0 m, and the rock at this portion will be higher than the others. Therefore, the alignment of conduit is proposed to follow a straight line in connecting the upstream axis of the stream with the downstream terrace on the left bank.

And in taking the elevation of conduit by 259.00 m at the level of the pipe center, it shall be connected with the terrace at the downstream left bank.

The terrace on the left bank will be suitable for power station.

The conduit is composed of a two meter diameter steel liner. The capacity is decided by the following formula;

$$Q = 9.56 \sqrt{h}$$

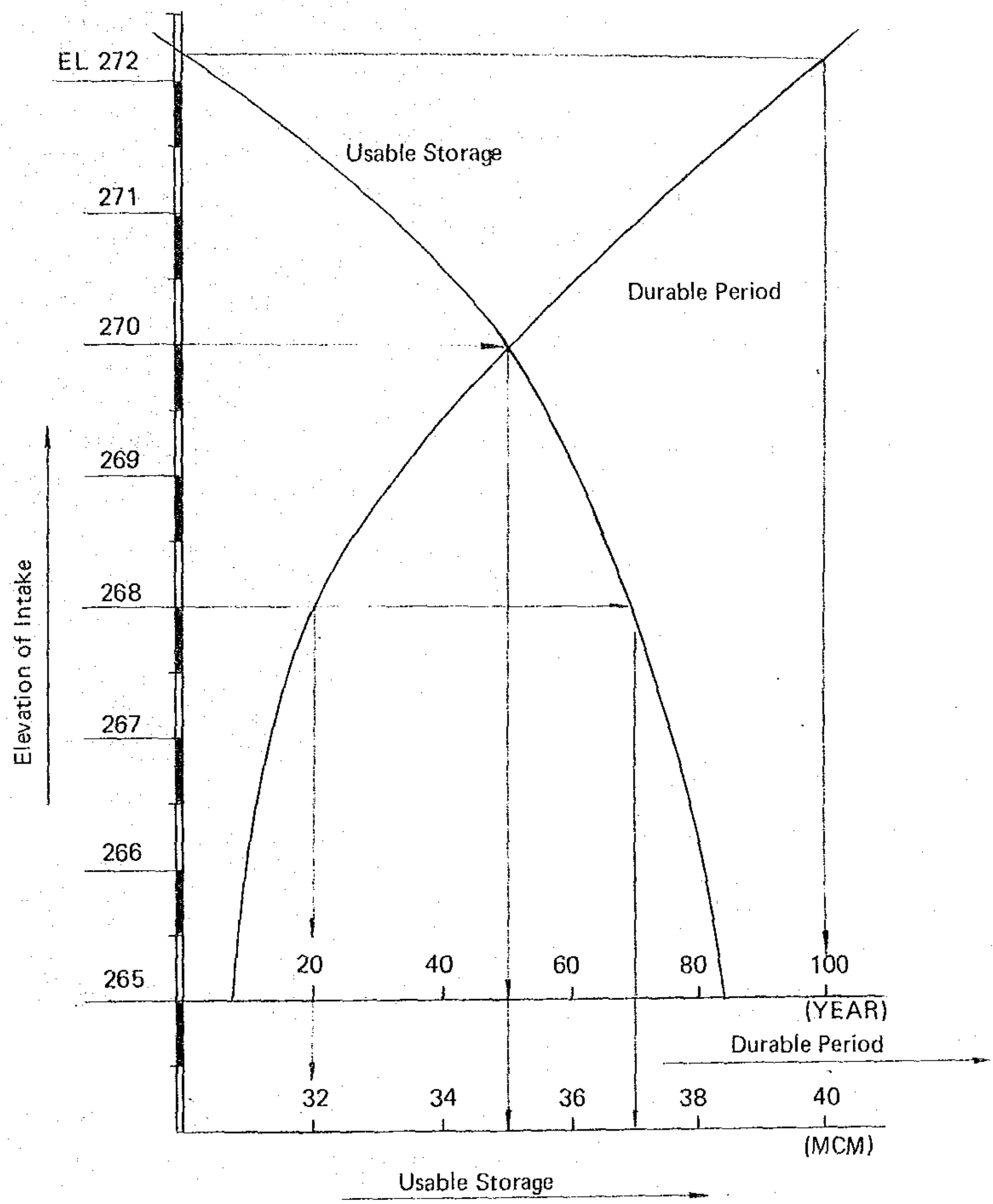
Where, Q: Discharge (m^3/sec)
H: Water Head (m)

When an upstream water depth (H) is five meters, the capacity of the conduit becomes 20 m^3/s.

The elevation of the top of the drop inlet is designed at EL 272.2 m, which is equivalent to the height of 100 year sediment.

The storage capacity available by the reservoir will be able to be increased by locating intake facilities as low as possible in the elevation in preventing them from being buried by sediments. The proposed reservoir, as illustrated in Fig.4.4-3, would have an

Fig. 4.4-3 Relation between Elevation of Intake and Usable Storage



effective storage capacity of 30 MCM at DWL 272.2 m by the 100-year sediment surface, while 35 MCM at EL 270.0 m by nearly 50-year sediment surface. When the intake facilities are provided at the elevation lower than 268.0 m, sedimentation would be accelerated whereas the storage capacity would not be increased so much.

In due consideration of these matters, the proposed intake facilities shall be constructed at two points at the elevation lower than EL 272.2 m, at EL 270.0 and EL 268.0 m.

The downstream outlet shall be provided with a regulating gate, guard gate and stilling basin. The dimensions of these facilities are selected, considering design discharge of conveyance canals. The tail of the gates is furnished with flange to convert easily to the future hydropower plant.

The major dimensions of the gates are as follows;

Regulating gate	ø1.3 m Jetflow gate
Guard gate	1.0 m x 1.0 m Slide gate

The jetflow gates will be employed as regulating gates because of the following reasons;

- to have well-watertightness, high abrasion resistance and little possibility of damage
- to have little fear at the valves being by clogged garbage which does not sink
- to allow dissipation by releasing water into the flow, although having no function as dissipator for the sake of its own

The stand-by gate, which will function only for shall be provided for stopping water and have slide gates from the viewpoint of economy.

The dissipation at the terminal shall be made by the projection into water in view of the low elevation of the conduit, because, if taking the method of a projection into air, the dissipation will be impossible due to jumping water to be affected by back water in the downstream.

Diameter determination of jetflow gate

The discharge amount of the proposed gates has been determined by adding some losses at the diversion in the downstream to 10.52 m^3/sec of irrigation water requirements to obtain 11.69 m^3/sec (=10.52/0.9).

The losses in taking, 2.0 m for pipe diameter and 150 m for the total length can be expressed by the following equation.

$$h = 1.4828 \times 1-^{-3} \times \frac{L \times Q^2}{D\ 5.33}$$

Where,

L = pipe length 9m)

Q = discharge (m^3/sec)

D = pipe diameter (m^3)

The total amount of losses will be found as $\Sigma\Delta h$ = 1.0 m in expecting 30 percent of the other miscellaneous losses.

The depth of the dissipation pool (Ht) can be expressed experimentally by the following equation:

$$Ht = 2.22\ Q^{0.4}$$
$$\doteqdot 5.9 \text{ m}$$

Consequently, the elevation of the water surface will be EL 264.9 m and therefore, the effective water head is as follows:

$$He = LWL\ (EL\ 273.3) - EL\ 264.9 - \Sigma\Delta h = 6.3 \text{ m}$$

The pipe diameter shall be determined in using He and Q by means of graph as d = 1.3 m.

Fig.4.4-4. Relation between He and Q

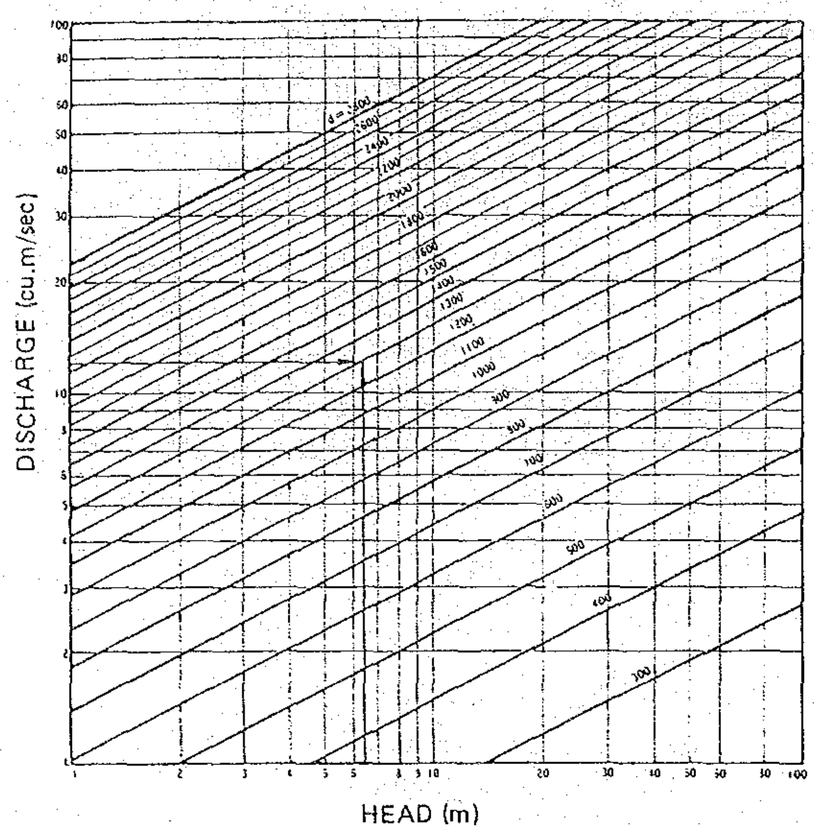
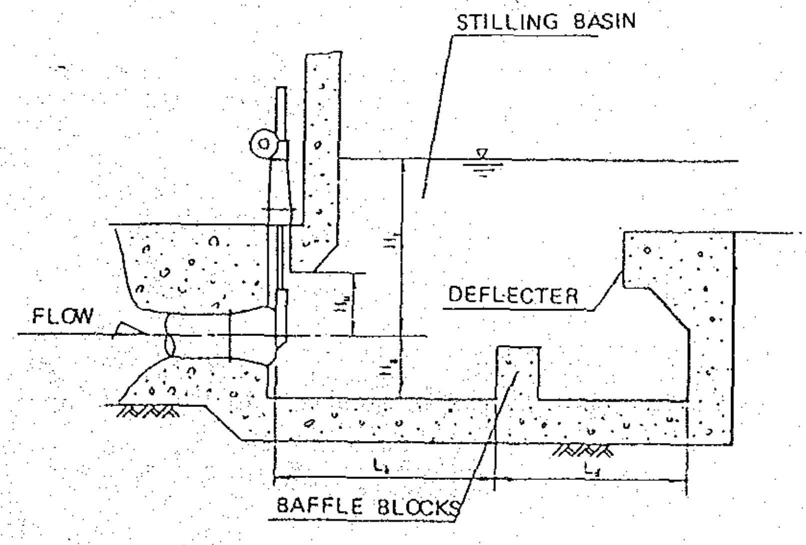


Fig.4.4-5. Profile of Stilling Basin



The respective dimensions can be obtained experimentally by the following equations.

$Lb = 0.333\ V^{1.25} a$

V: velocity at outlet $V = \sqrt{2gHe}$

$He = FWL - EL\ 264.9 - h$

$= 14.3$ m

$\therefore\ V = 16.7$

a: Gate opening degree at the max. water intake (65%)

$\therefore\ Lb = 7.3$ m

$Ld = 2.63\ Q^{0.4}$

Where,

Q: Intake amount 11.69 m^3/s

$\therefore\ Ld = 7.0$ m

Therefore, the length of the dissipating pool is: $L = Lb + Ld +$ 14.3 m

$Hg \geqq 1.5\ d$

Where,

d: Diameter of Gate 1.3 m

As a results Hg = 2.0 m can be obtained

$B \geqq 3d$

Where,

B: Width of the dissipating pool

Therefore, B = 4.0 m can be decided.

9) Foundation Treatment

a) General

All the proposed dam sites of A, B, C, and D have a foundation layer of semiconsolidated clayey soils and rocks, except for sand and gravels in the river bed, lower and middle terrace deposits. These structures are deemed satisfactory in terms of the bearability as foundations for fill-type dams. Unconsolidated layer like river deposits, low and middle terrace deposit are thick and less than eight meters that appears to provide a favorable condition for foundation treatment works.

In the Project Area, the structure requiring the foundation treatment is the silty sand and gravels of the diluvium higher terrace deposit, weathered shales, mudstone, calcarious sandstone of the Mesozoic and cavious limestones with heavy cracks.

The diluvial structure, consisting mainly of clayey soils and sometimes including sand and gravels, will be in semiconsolidation with a permeability coefficient in the level of 1×10^{-4}, and there will be some fear for piping occurring in the Case that the said structure would thinly develop from the upper to lower streams of the river especially at dam site A, B and D.

This structure comprises a slightly loosen sand and gravel layer which, with clayey soils mixed, is of about two-meter thickness. Under the conditions, the permeability coefficient seems to range from 1×10^{-3} to 5×10^{-4}, and the seepage control should be carried out with careful attention to seepage through this sand and gravel layer.

The permeability of shales, sandstones, etc. will be expressed by Lugeon in a range of 10 to 50, and judging from mode of occurrence the curtain grouting is considered most effective and economical as seepage control.

However, distribution of massive limestones has caused many long open cracks and permeability varies from depth with the cracks in a range from Lugeon 20 to 100. In such conditions, seepage control should be carried out with attention as careful as possible.

In due consideration of the topographical and geological conditions prevailing in the proposed dam sites, the following foundation treatment methods are in particular recommended in terms of importance of seepage control.

(i) Open method

For the Areas where the shallowly deposited sand and gravels and unsuitable base rocks are found, replacement of these materials should be made with the materials for seepage control.

(ii) Casting underground method

Large diameter borehole drilling or some other drilling should be made to form continuous walls in the ground by filling up the caves with concrete mortar.

(iii) Grouting method

After borehole drillings, grouting will be made to fill up openings in the soils and rocks of the layers with cement milk, mortar or any other suited grouting materials.

(iv) Blanket methods

There shall be such impervious materials widely spread over the ground surface as clayey materials, asphalt or any other appropriate materials for seepage control.

(v) Others

For large caves or wide faults found in some parts, tunnels or pits shall be dug to fill them back with concrete or mortar.

(vi) Selection of suitable method

The aforesaid practical methods were carefully studied and some suitable methods have been recommended to meet the specific conditions of the proposed dam sites.

b) Planning

Basalts are observed on the both abutments of this site in boulder on the right abutment and in massive hardrocks with heavy cracks on the left abutment. Permeability is high range more than 100 Lugeon.

Judging from rock condition, basalt will make excavation a little difficult but is considered to be advantageous.

Low and middle terrace river deposits shall also be excavated.

For river bed rocks, only the excavation of heavily weathered rocks will be sufficient for the purpose.

Although the transposition method by excavation of unsuitable and poor quality rocks and unconsolidated layer is deemed most economical and safe in technical terms.

For the proposed site A, the transposition method shall be advantageously employed within the extent mentioned above, while the grouting method will be most recommendable for the seepage control of the contact faces of both abutments with higher terraces and deep rock.

The fundamental pattern of the blanket and curtain grouting are shown as Drawing No.2 juging from rock condition and reservoir water level, grouting depth should reach 20 m deep from the foundation excavation surface in the way of ordinary grouting method.

4.4.2. Diversion Dam C

1) Topography and Geology

a) Topography

(Refer to Storage Dam A)

b) Geology

The Dam Site C forms hill rocks on both banks, and is distinguished in the very mild topography, with the natural ground surface assuming an irregular solution which is a characteristic feature of the massive limestone areas. The base rocks are the middle formation of the Lampang Group with its very hard, gray and massive limestones, and the surface of which is affected by solution with some ten centimeter to two meter (See Appendix 4.4-1). The gallery observation revealed that there were several solution traces as large as several 10 cm in size even 30 m deep from the rock surfaces, and rock classification is CM. As a whole, many cracks run at 50 cm to 100 cm intervals and the clay seams clog the cracks about 10 m below the rock surface. But it is sufficient in bearing capacity for the dam structure.

According to a previous investigation report by RID team, the bedrock usually shows high permeability ranging from 50 to 100 lugeon in a weathered zone within five meters from the rock surface. Permeability usually shows a tendency to exhibit a smaller range at lower parts in fresh rock. However, these values vary from one place to another or in depths and show more than 50 lugeon in some places. However, in the Case of the diversion dam, the high water level is 255 m elevation, and the water depth is less than 9 m in most of the area near the dam site. The bedrock is hard and solid, so it is possible to control leakage by high pressure grouting at the dam site according to requirement. The groundwater table can be expected to be a level slightly higher than the river water level at the point about 200 m apart from the dam site.

On the other hand, there are some sinkholes on the left and right bank upstream from the dam site as shown Appendix 4.4-1. On the right bank, there are three sinkholes near the Mae Chang river 500 m to 1,200 m in distance from the dam site. The sinkholes are about 250 m to 253 m in elevation and the depth is approximately 3 to 6 m from the surface. So it is considered that there is a possibility of the existence of another hidden sinkhole near the dam site. But it is impossible to search for a hidden sinkhole from the subsurface.

It is not so serious, because the groundwater table is higher than the Mae Chang river at the mountain side near the dam site.

However, it is necessary to observe the depth of the groundwater table by measurement throughout the wet season and the dry season near the dam site. On the left bank 800 m in a southerly direction from the dam site, four sinkholes are found, at an elevation of about 260 m. Their depth is more than 8 m. This means that the bottoms of the sinkholes are less than the high water level. The slope of the topography near the sinkholes inclines toward the Mae Chang river at a distance of more than 150 m from the back water. Although there is no definite conclusion concerning water leakage, there seems to be no particular problem. To make doubtly sure, however, it is better to trace the leakage route of the water using salt or another chemical.

On both banks, terrace deposits of more than 100 m wide extend in the upper portions of the base rocks towards the upper stream along the Mae Chang river. This terrace deposits, consist mainly of silty clay soils with some sand materials contained, is a well consolidated with N values raging from 20 to 30. The layer, having thickness ranging from three to five meters, is distributed up the point with elevation of 250 to 253 m around the dam site and plays a role of a natural blanket together with clogs sinkholes found in the area of less than EL 250 m.

The present river bed deposits are composed of the sand and gravel layers with medium/small sized round gravels with diameter of one to five centimeters and thickness of three to five meters.

2) Study of Dam Axis

The dam axis was restudied considering the following items and was moved approximately 80 m upstream in parallel to the original dam axis decided by RID.

i) The scale of the spillway is larger in comparison with the same scale, so it becomes more favorable situation for appurtenant structures to remove the dam axis to the upstream.

ii) The right abutment on the original dam axis has a steep slope, so it is not easy to carry out treatment of the foundation.

iii) The seepage path of both abutments become longer due to movement of the dam axis.

iv) The dam volume shows almost no change, even though the original dam axis was transferred to the upstream.

The shape of dam site is as follows:

Right abutment	slope 1:2.0
Left abutment	slope 1:2.0
Width of river bed	40 m
L/H ratio	
- Concrete dam	4
- Fill dam	20

3) Embankment Materials

a) Location of Borrow Area and Quarry

The borrow area is located at both terraces upstream from the dam site. The borrow areas were already investigated and tested by RID. A location of quarry has not been decided yet, but it is able to obtain rock (limestone) around the dam site.

b) Conservative Quantity

The terrace deposits at the borrow area have approximately 3.0 m and it is possible to obtain the necessary quantity judging from a dam scale. According to the RID's investigation, the conservative quality is approximately 350,000 cu.m. As a rock is used for riprap only, it is possible to obtain the necessary quantity around the dam site.

c) Quality

(1) Borrow area

The impervious materials are mainly classified into M Land CL by the unified soil classification from the results of soil tests. The contents of silt and clay 0.074 mm passing are 61 per cent to 95 per cent range. These materials have better property for the permeability, but these materials have a danger against cracks in case of compaction under dry condition. So, it is important to control soil water content and degree of compaction on the field.

(2) Quarry

Material tests of a rock of quarry were not carried out. The quality is a hard limestone and a favourable material as a riprap.

4) Dam Type

The following dam types can be technically considered from the viewpoint of the topography and geology of the dam site C.

Fill dam
Concrete gravity dam
Combination of fill dam and concrete gravity dam

A concrete gravity dam is of a simple structure because a spillway can be provided on the dam body but its construction cost is greater than for a fill dam. The plain terrace and a U valley compose of a special topographycal condition at the dam site C. The water-flowing valley is on the left bank and its depth from the level of the terrace is about 11 m. There is a plain terrace about 200 m in length on the right bank.

On considering a fill dam on this topography, there are two ideas for the location of a spillway. One is on the river bed, the other is on the terrace.

Situating a spillway at the terrace requires excavating a canal in front of a spillway in order to smoothly lead floods. A flood way is also necessary on the downstream. From the viewpoint of hydrology, it is most favourable to select the location of a spillway at the river bed. In this case, the spillway takes a type like a spillway of a concrete gravity dam. In conclusion, as a fill dam is constructed on the right terrace, a type of a dam is considered as a combination dam. A fill dam is a small dam (dam height = 10.5 m) and therefore a homogeneous type is most suitable. A type of a concrete dam adopts a concrete gravity dam.

5) Dam Dimensions

a) Dam Crest Elevation

$$\text{Dam crest EL} = \text{HWL} + \text{Fb} + 0.5$$

Where,

HWL: High water level

Fb : Freeboard

0.5 = In case of a spillway with gate, 0.5 is added in consideration of an accident of gate operation

This dam site has sufficient abutments on the topography against construction of a large dam. However, the height of dam is limited by the social problems accuring with submergence. For that reason, the water level of reservoir shall not be more than EL 255.00 m. The high water level in this dam site apply EL 285.00. The freeboard adopt 2.0 m as well as dam A. Dam crest elevation obtains EL 257.50 m from the above equation.

b) Width of Dam Crest

In case of a fill dam, the width of a dam crest adopts 6.0 m in accordance with the standard of fill dams in Thailand, and in the case of a concrete gravity dam, the width of a dam crest 5.0 m after taking into consideration installation of gates and the passing of vehicles.

c) Embankment Side Slopes

The embankment side slopes of a fill dam were decided taking into consideration examples of fill dams constructed in Thailand, and stability analysis. The upstream slope is 1:3.0 and the downstream is 1:2.5. The slope of a concrete gravity dam is decided by stability analysis based on a basic triangle. The upstream slope is straight and the downstream slope is 1:0.7.

As a conclusion, the principal features of the diversion dam C are summerized below.

Reservoir

Catchment area	254	sq.km
High water level	EL 255.00	m
Full water level	EL 254.00	m
Intake water level	EL 251.00	m
Usable reservoir capacity	7	MCM
Usable water depth	3	m

Concrete gravity dam

Dam height	22.5	m
Dam crest elevation	EL 257.50	m
Min. trench elevation	EL 235.00	m
Crest length	67.5	m
Dam volume	11,700	cu.m

Fill dam

Dam height	10. 5	m
Dam crest elevation	EL 257.50	m
Min. trench elevation	EL 247.00	m
Crest length	242.50	m
Dam volume	60,000	cu.m

6) Stability Analysis

a) Method of Stability Analysis

(1) Fill Dam

Stability analysis of the dam was performed under the condition of "at the end of dam construction" by using only the same method of the dam A because of the small size at this dam.

(2) Concrete Gravity Dam

Stability analysis of the concrete gravity dam was performed about the following items under the condition of full water level.

(i) Overturning

The vertical tension stress must not act on the upstream edge of any horizontal section. That is to say, The combined force of total loads acting on the dam body shall be within Middle Third.

(ii) Sliding

The shear friction resistance of the contact plane between dambody and bedrock must have a safety factor required for shear force. The safety factor shall not be less than 4.

Materials	Wet Density (rt) (t/m^3)	Saturated Density (rast) (t/m^3)	Cohesion (e) (t/m^2)	Internal Angle (ϕ) (°)
Impervious material	1.85	1.98	1.5	13°-30'
Filter	2.00	2.10	0	35°-00'

Notes: * The shear strength test of the impervious materials was carried out by using a direct shear machine. A shear strength obtained from a direct shear machine is bigger than the value of the triaxial compressive strength test due to the difference of machine function. The design values adopt 80 per cent of the shear strength obtained from the direct shear machine.

(2) Concrete Gravity Dam

Design values are as below:

Unit weight of concrete .... 2.3 t/m^3
Unit weight of sediment load .... 1.6 t/m^3

Coefficient of earth pressure of sediment load 0.5
Coefficient of uplift 1/3
Coefficient of earthquake 0.05
Shear strength of bedrock .... 10 kg/cm^2

Notes: Shear strength is assumed value

c) Results of Stability Analysis

(i) Fill dam

The stability analysis of a dam was performed by computer under the above condition. The results are as follows:

Condition	Earthquake Force	Water Level	Slope	Safety Factor
1	K = 0.05	-	Upstream Downstream	

The contour of safety factor is shown in Fig.4.4-28 in Appendix 4.4-3.

(ii) Concrete gravity dam

The combined force of total loads acting on the dambody is within Middle Third and safety factor against sliding has more than 4.

7) Spillway

Type

The high water level is designed at EL 255.0 m, taking into consideration the submerged area. On the other hand, the intake water level is kept at EL 251.0 m.

When a diversion dam is adopted, the water depth available for intake becomes as small as three meters which is the difference between the full water level of EL.254.0 m and the intake water level of EL 251.0 m, and so the necessary capacity cannot be secured;

To obtain a more effective storage capacity, a gate type is applicable for spillway design. the comparison between a gate type and a gateless type is shown as follows:

		Gateless Type		
	Gated Type	Case-1	Case-2	Case-3
° Over flow depth	7 m	2 m	3 m	4 m
° Crest length	50 m	300 m	180 m	120 m
° Full water level	EL 254.0	EL 253.0	EL 252.0	EL 251.0
° Effective storage capacity	7 MCM	4 MCM	2 MCM	0 MCM

The gateless spillway does not keep the effective storage capacity sufficiently, and it requires a long fixed weir; therefore, a gated type spillway is more suitable to this site. This spillway requires a 50 m fixed weir.

Due to provisions of the weir gate control is usually not necessary. Flood water less than 100 cu.m/sec flows at through the fixed weir naturally. Floods more than 100 cu.m/sec of flood occurred three times per year during the recent 10 years.

Site

The gates to be employed are the taintor type that is widely used in Thailand. The said gates should be constructed in the manner that a concrete gravity dam shall be provided on the river bed, where the spillway is to be installed, considering the spillway width of 56 m.

Such facilities are deemed to be advantageous in terms of both hydraulics and structure.

The fixed weir is positioned on a flat area on the left bank and intake facilities are on the right bank.

The bridge of the spillway is five meter in width. It can be used for operation, maintenance and passage.

Dimensions

The design flood discharge estimated for the proposed site is 1803 m^3/sec, and the flood flows will be stored by about 5.0 MCM available within a range from FWL 254.0 m to HWL 255.0 m by diversion dam. The peak of flood discharge of 1803 m^3/sec, however, appear 17 to 18 hours after the rainfall, whereas the water level in the reservoir reaches HWL 255.0 m five to six hours after water beings to flow into the reservoir. The designed flood discharge of the spillway, therefore, shall be determined by 1,803 m^3/sec, the peak inflow to the reservoir.

The major dimensions of the structures for the design flood discharge are shown as follows:

Design flood water level	EL 255.00 m
Full water supply level	EL 254.00 m

Gate Spillway

Width	12.5 m x height 7.1 m x 4 sets
Crest elevation	EL 247.40 m
Design spillway discharge	1,698 cu.m/s
Overflow depth	6.6 m

Fixed Weir

Width	50 m
Crest elevation	EL 254.00 m
Design spillway discharge	105 cu.m/s
Overflow depth	1.0 m

Hydraulic Computation

In taking the overflow depth of the flow by 1.0 m for the fixed weir, the full water level will be EL 254.0 m, equal to the crest height of the weir, because of the highest water level of EL 255.0 m. The said capacity, when taking an overflow coefficient by 2.1 and the length by 50 m, can be expressed as follows:

$$Q = CLH^{3/2}$$

Where,

C: overflow coefficient

L: length (m)

H: overflow depth (m)

$= 105\ m^3/sec$

Consequently, the design flood discharge for the gated spillway would be 1803 m^3/sec, and Q = 1,803 - 105 = 1,698 m^3/sec can be obtained. And the overflow depth can be expressed as follows, in taking the number of gates (12.5 m width) by four and the overflow coefficient by 2.0.

$$H^{3/2} = \frac{Q}{C\ B\ N}$$

$$= 16.98$$

$$= 6.6 \text{ m}$$

Where,

Q: discharge (m^3/sec)

C: overflow coefficient

B: gate width 9m)

N: number of gates

The necessary gate height will be 7.1 m including a 50 cm freeboard. The crest of gated spillway will elevate at 248.4 m which is the elevation 6.6 m lower than HWL 255.0 m.

8) Intake

(1) Site

Location of intake facilities is proposed at the upstream of the spillway on the right bank by the following reason. Most of the irrigation water is supplied to the right bank area.

Sediment materials deposited in front of intake facilities are to be flushed out by operating the spillway gate.

(2) Structures

Three-meter head is available from EL 251.0 m to EL 254.0. The irrigation water is conveyed to a box culvert through the inlet with screens and slide gates. After crossing the dam, it is delivered to the canal.

The slide gates are composed of three sets of a unit gate of 2.5 m x 2.5 m in dimensions.

9) Foundation Treatment

a) General

(Refer to Storage Dam A)

b) Planning

The three-to-five meter thick silty terrace deposits developed on the right and left bank as well as the river bed sand and gravels should be advantageously replaced with other suitable materials.

The bedrock of this site consists of hard and massive limestone and as the result of the test groutings, the cement used is estimated at as much as 97 kg/m used. Although heavily varying in rock conditions in cement consumption from a place to others, the proposed site will have many continuous heavy open cracks in the bed rock.

Judging from hardness and irregular cracks of the rocks, the Curtain Grouting Method to be applied along the dam axis can be recommended as most suitable to this site.

Usually diversion dam do not need very deep foundation treatment, but grouting treatment needs up to 15 m depth from rock surface is necessary in this site considering speciality of limestone and distribution of Lugeon value.

The curtain grouting method must be changed to materials such as mortar or others, according to condition of the grout hole.

Areas extending from the both left and right abutments to the mountain sides, have high permeable weathered rock, but leakage is trivial.

Part of concrete construction will require reinforcement of the bedrocks and seepage control works for the expected total concrete area due to heavy irregularities on the surface of rocks.

In due consideration of these conditions, the terrace deposits lying in the reservoir areas near the both abutments should remain intact as a natural blanket.

For diversion type dam which will keep the water level less than EL 255 m, stability of the dam based on the foundation rocks will be fully secured. On the other hand, however, the seepage control of the diversion-type dam should be considered and established prudently in proportionally considering the stored water pressure for high dam especially in this site condition. Consequently, the grouting should be carried out according to the fundamental grouting pattern shown as Drawing No.5.

The existing caves recognized in the limestone should be further surveyed to gain knowledge about their respective scales by tracing groundwater. Finely-tuned measurement of the groundwater table should be carried out around the dam sites.

4.4.3. Diversion Dam D

1) Topography and Geology

a) Topography

(Refer to Storage Dam A)

b) Geology

In the area, a vast terrace deposits plain extends from the proposed site towards the downstream and thus the site is less advantageous in terms of the topographic condition.

The bedrock consists of sandstone, shale, calcarious sandstone, limestone and conglomerate. (See Appendix 4.4-1). On the right bank, there is distributed alternately not so hard fine to medium sandstone and shale. The rock surface is undulating at about 245 m in elevation rising and falling every few meters. At the river bed and right river side, hard calcarious sandstone and partly banded limestone is distributed. At the left river side, on the diversion axis, there are distributed middle terrace deposits in alluvium age. The bedrock is covered by the terrace deposit, so there is no out cropping. The rock surface is approximately 235 m in elevation the same as the river bed point. But it is considered to increase in elevation moving toward the hill area.

On the left bank, there is massive limestone, sandstone and shale partly alternately interbedded with conglomerate. In most of the hilly area there is massive hard limestone. At the diversion axis, the dip and strike of the bedrock is N30°W, 38°E. Strike and dip observed around the site are various, and boring cores show some share zone, but are unclear on the surface. Permeability of the bedrock, usually indicates under 20 lugeon in fresh rock, in the weathered zone, sometimes shows more than 100 lugeon, about 4 to 7 m in thickness from the rock surface. The compressive strength of the bedrock indicated 150 kg/cm^2 to 400 kg/cm^2 by boring core. The groundwater table is higher than the river water, at the right and left bank also. The river bed is five meters in thickness and is composed of loose sand, gravels and silt.

On the left bank of the diversion axis, there are distributed alluvium terrace deposits, whose thickness and other geological conditions are unknown. From the above it seems that there is no major problem facing the diversion foundation.

It is necessary to acquire further geological formation along the new diversion axis.

2) Study of Dam Axis

The dam axis was decided considering the following items based on the topography map surveyed by RID.

i) To select a dam site on a narrow valley

The outline of topography at the dam site D is as follows. The dam site D is located at about 1.5 km downstream from the Ban Sop Po. The Mae Chang river changes its direction of flow from west to north about 1.5 km upstream of the dam site, and changes back to west at about 1.2 km downstream of the dam site. The right bank forms the Amphoe Mae Tha plain having a gentle slope and EL 260 m, and the left bank forms a mountain having summit elevation EL 320 m. This dam site is the narrowest place on topography.

ii) To take a right angle dam axis toward the river and a contour line

It is favorable for the stability of dam, intake and washing out the sediment load to select a right angle dam axis toward the river and a contour line.

iii) To select a favorable topography for the situation of a spillway

The shape of dam site is as follows;

Right abutment	slope 1:8.0
Left abutment	slope 1:2.5
Width of river bed	40 m
L/H ratio	8

3) Embankment Materials

a) Location of borrow area and quarry

The borrow area is located at the terrace about 300 m upstream from the dam site. A location for a quarry has not yet been decided, but it is able to obtain rock material (limestone) from the right mountain to the right of the dam site.

b) Conservative Quantity

(1) Borrow area

The impervious materials are mainly classified into SC, ML and CH by the unified soil classification from the results of tests. The contents of silt and clay (0.074 mm passing) are 21.0 per cent to 51.0 per cent range. These materials have better permeability, but also have of danger of cracking when compacted under dry conditions. So, it is important to control soil water content and the degree of compaction.

(2) Quarry

Material test an a rock of quarry were not carried out. The quality is a hard limestone and a favourable material as a riprap.

4) Dam Type

The topography of dam site D forms a terrace, about 140 m width. The river bed is located on the right bank, and the section is a trapezoid. This topography is similar to the dam site C.

Therefore, it is suitable to apply a combination dam as well as the dam C. The arrangement of the concrete dam and fill dam takes the same type as the diversion dam C. As the fill dam is a small dam (dam height = 7.0 m), a homogeneous type is most suitable.

5) Dam Dimensions

a) Dam Crest Elevation

The dam crest elevation is EL 252.50 m using the same idea as the diversion dam C.

b) Width of Dam Crest

Width of dam crest is the same as the diversion dam C.

c) Embankment side slopes

The embankment side slopes were decided by the same idea as diversion dam C and stability analysis. The upstream slope is 1:3.0 and the downstream is 1:2.5. The slope of concrete gravity dam was decided by stability analysis based on a basic triangle. The upstream slope is straight and the downstream slope is 1:0.7.

As a conclusion, the principal features of diversion dam D are summarized below.

Reservoir

Catchment area	293 sq.km
High water level	EL 250.00 m
Full water level	EL 249.00 m
Intake water level	EL 248.00 m
Usable reservoir capacity	2 MCM
Usable water depth	1 m

Concrete gravity dam

Dam height .... 21.5 m

Dam crest elevation .... EL 252.50 m

Crest length .... 70 m

Dam volume .... 13,400 cu.m

Fill dam

Dam height .... 7 m

Dam crest elevation .... EL 252.50 m

Crest length .... 155 m

Dam volume .... 12,000 cu.m

6) Stability Analysis

a) Method of Stability Analysis

(1) Fill dam

Stability analysis of the dam was performed under the condition of "at the end of dam construction" by using only the same method of the dam A because this dam is small.

(2) Concrete gravity dam

Stability analysis of the concrete gravity dam was performed about the following items under the condition of full water level.

i) Overturning
ii) Sliding

b) Design Values

(1) Fill dam

Design values to be used for stability analysis of the dam were determined based on the results of soil tests. Design values are as below;

Materials	Wet Density (rt) (t/m^3)	Saturated Density (rsat) (t/m^3)	Cohesion (C) (t/m^2)	Internal Angle (ϕ) (°)
Impervious material	1.89	2.00	3.5	10°-30'
Filter	2.00	2.10	0	35°-00'

Note: * See the diversion dam C

(2) Concrete Gravity Dam

Design values are the same as the diversion dam C.

c) Results of stability analysis

i) Fill dam

The stability analysis of dam was performed by the computer under the above condition. The results are as follows;

Earthquake Condition	Water Force	Level	Safety Slope	Factor
1	k = 0.05	-	Upstream	
			Downstream	

The contour of the safety factor is shown in Figure 4.4-29 in Appendix 4.4-3.

ii) Concrete gravity dam

The combined force of total loads acting on the dambody is within Middle Third and Safety factor against sliding has more than 4.

7) Spillway

Type

The high water level is planned at EL 250 m and intake water level is at EL 248 m. the difference of the both water levels are less than that of diversion dam C. Therefore, the gated spillway is selected as studied in the previous paragraph 8.3.(2). Also, a 50 m fixed weir is to be provided in this spillway to release the normal food flow.

Site

The proposed dam type is of the same combination type as that proposed for dam site C. With this type of dam, the length of the spillway could be possible shortened, hydraulically advantageous and lessen in the filling materials for their embanking volume, if the spillway which is most concrete-consuming structure of the whole structures would be constructed at the river bed portion.

On the other hand, the fixed weir, requires a rather spacious site for construction and should be provided on the right bank terrace which is deemed most suitable site. Under the conditions, the intake facilities could be constructed on the left bank site.

Dimensions

Since the proposed site has a small storage capacity of the flood discharge, the dimensions of the spillway will be decided by the peak flood discharge of 1,809 m^3/sec.

The major dimensions are tabulated as follows:

Design flood water level	EL 250.00 m
Full water supply level	EL 249.00 m

Gate spillway

Width	12.5 m x Height 7.1 m x 4 sets
Crest elevation	EL 243.4 m
Design spillway discharge	1,704 cu.m/s
Over flow depth	6.6 m

Fixed weir

Width	50 m
Crest elevation	EL 249.00 m
Design spillway discharge	105 cu.m/s
Overflow depth	1.0 m

Hydraulic Computation

The full water level of the fixed weir, intaking the overflow depth by 1.0 m, will be the same elevation as that of the weir crest of EL 249.0 m, since the highest high water level is EL 250.0 m. When taking the overflow coefficient by 2.1 and the weir length by 50 m, the relevant discharge capacity can be expressed by;

Where,

$$Q = C \cdot L \cdot H^{3/2} = 105\ m^3/sec$$

C = overflow coefficient
L = length (m)
H = overflow depth (m)

Since, the designed flood discharge is 1,809 m^3/sec, the discharge from the gated spillway will be Q = 1,809 - 105 = 1,704 m^3/sec, and the overflow depth will become as follows for four 12.5 m wide gates and the overflow coefficient of 2.0.

$$H^{3/2} = \frac{Q}{C \cdot B \cdot N} = 17.04$$

$$\therefore H = 6.6\ m$$

Where,

Q = discharge (m^3/sec)
C = overflow coefficient
B = gate width
N = number of gates

The total gate height is 7.1 m including 50 cm freeboard. The crest of the gated spillway will elevate at EL 243.4 m which is 6.6 m lower than HWL 250.0 m.

8) Intake

Selection of intake and facilities and their location was made in accordance with those conditions of site C. Three sets of slide gates with 2.5 m x 2.5 m in dimensions are required.

The water level ranging from EL 249 m to EL 248 m is used for irrigation purposes.

9) Foundation Treatment

a) General

(Refer to Storage Dam A)

b) Planning

Terrace deposits are observed on both banks. On the right bank, the middle and higher terrace deposit distributed within five meter thickness along the dam axis, so that excavation and replacement with other suitable materials should be carried out for the right bank core zone and river deposit also. On the left bank there are middle terrace deposits having thicknesses of over ten meters. It is difficult to excavate and to replace up to the rock surface. The terrace deposits seem to have a low permeability but include sand and gravels at the bottom of the layer.

The bedrock consist of calcarious sandstone, shale and conglomerate. Usually these rocks are hard and low permeability in fresh rock, but in the weathered zone three to five meters from rock surface, the bed rocks show sometimes high permeability.

Judging from such foundation conditions, in this site shown as Drawing No.7, the grouting method is a suitable method to the effective seepage control. On the concrete structure, the consolidation grouting should be carried out.

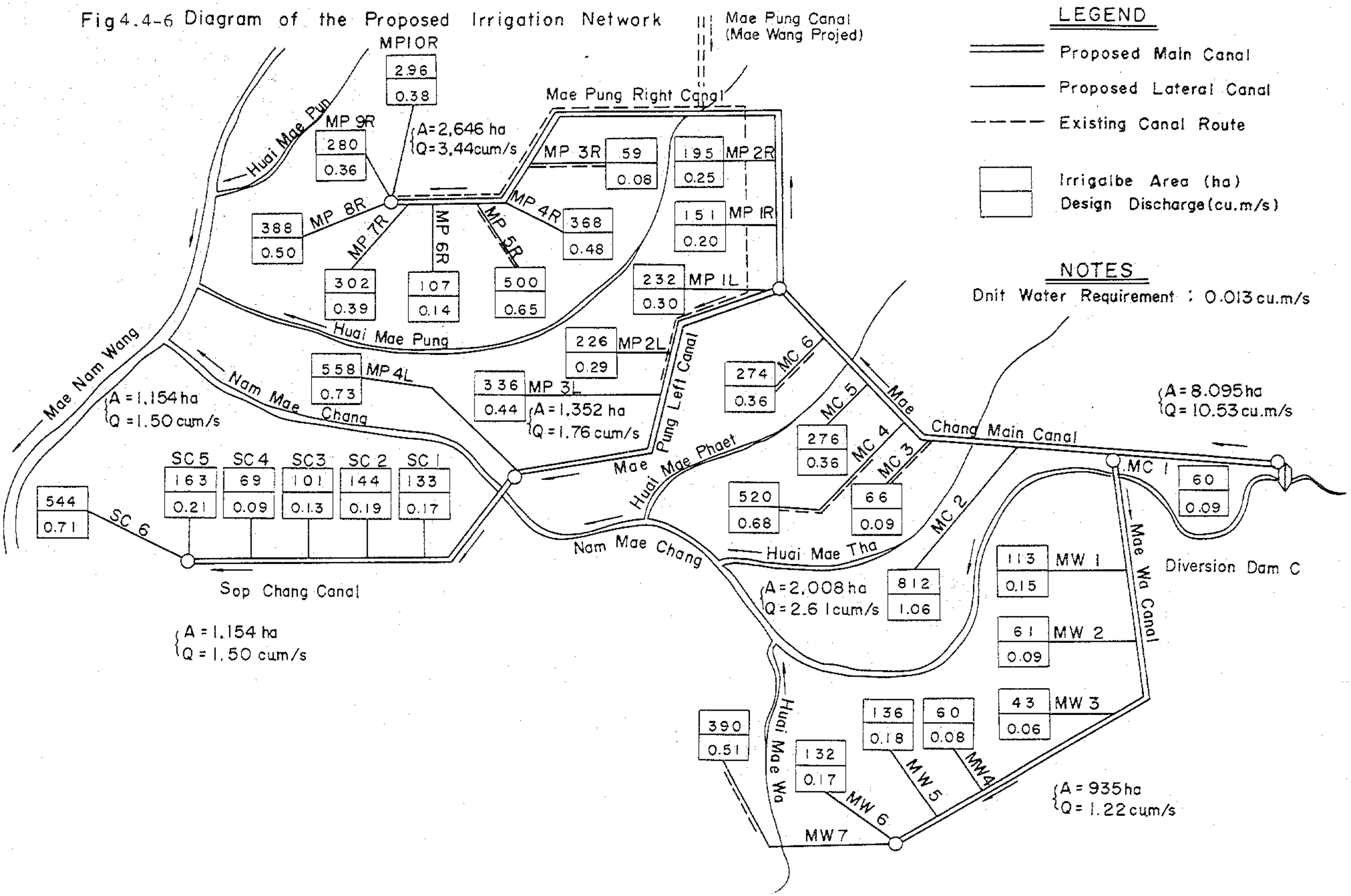
4.4.4. Irrigation Canals

1) Proposed Irrigation Network

The irrigable area can be divided into right and left bank and in detail into five zones as follows:

		Acreage (Net; rai)	
Zone	Sub-area	Case-5	Case-6
1	Mae Chang	12,550	11,600
2	Mae Pung Left	8,450	8,100
3	Mae Pung Right	16,540	14,400
	Sub-total of Right Bank	37,540	34,100
4	Mae Wa	5,840	4,800
5	Sop Chang	7,200	7,000
	Sub-total of Left Bank	13,040	11,800
	Total	50,600	45,900
		(8,095 ha)	(7,349 ha)

Fig 4.4-6 Diagram of the Proposed Irrigation Network



These two sub-areas are covered by five main canals (Fig.4.4-6). Mae Chang Main Canal (MCMC) is diverted from the right bank of the diversion dam and diverts irrigation water to the Mae Wa Area in the left bank by means of Mae Wa Canal. After supplying water in the Mae Chang Area (Mae Tha District), MCMC distributes to Mae Pung Left Canal (MPLC) and Mae Pung Right Canal (MPRC). MPLC irrigates the left bank of the Huai Mae Pung and connects to the Sop Chang Canal (SCC). SCC runs over the Mae Chang river and into the left bank of it. Downstream SCC reaches to the left bank of the Mae Wang River. MPRC takes the route of the existing canal and continues in a round about way to the elevated area of the Pun Area. The Mae Wa Canal (MWC) runs over the Mae Chang river and covers the right and left bank of Huai Mae Wa.

The water source for the Mae Pung Left and Right area will be turned from the Mae Wang - Kew Lom Project to the Project. This fact means that the latter project would be able to concentrate on irrigation to the upper part of Mae Wang Left Area. Due to the Feasibility Report by JICA, March 1980, such concentration allows the Mae Wang-Kew Lom Project to solve the drought hazard shown in Table 7 and Fig. 3. (Page 51, of the above mentioned Feasibility Report)

2) Main Irrigation Canals

Five canals are named as the main irrigation canals to irrigate the Project Area. Out of the five, two canals take the routes of existing canals partly but all facilities in the two canals will be improved as a new irrigation network. The remaining three will be newly constructed.

In Mae Tha District, now partly irrigated by NEA's pumping projects, existing facilities, such as pump stations, pipes and main canals, should be removed by NEA and new facilities of the Project will be constructed according to the new irrigation network (refer to Appendix 3.4-4, Page 5).

Main Irrigation Canals

Name of Canal	Case-5 Service Area (rai)	Case-5 Length (km)	Case-6 Service Area (rai)	Case-6 Length (km)
Mae Chang Main Canal	12,550	12.7	11,600	11.7
Mae Pung Left Canal	8,450	6.5	8,100	4.2
Mae Pung Right Canal	16,540	13.0	14,400	13.0
Mae Wa Canal	5,840	10.4	4,800	10.4
Sop Chang Canal	7,200	8.7	7,000	11.1
Total	50,600	51.3	45,900	50.4

(1) Routes of Main Canal

Selection of the routes is made from the results of the field survey and 1:10,000 scale topographical maps taking the following into consideration.

i) To maintain the highest water level to cover a larger service area and facilitate gravity irrigation, and

ii) To select a course as straight as possible to shorten the length.

(2) Canal Type and Design Discharge

The main canals are lined with concrete, taking the following into consideration; increase of the velocity of flow and easiness for maintenance of canals. The maximum design discharge for each main canal is as follows;

Maximum Design Discharge (cu.m/sec)

	Case-5	Case-6
° Mae Chang Main Canal (MCMC)	10.52	9.55
° Mae Pung Left Canal (MPLC)	3.26	3.16
° Mae Pung Right Canal (MPRC)	3.44	2.99
° Mae Wa Canal (MWC)	1.22	0.99
° Sop Chang Canal (SCC)	1.50	1.46

(3) Cross Section

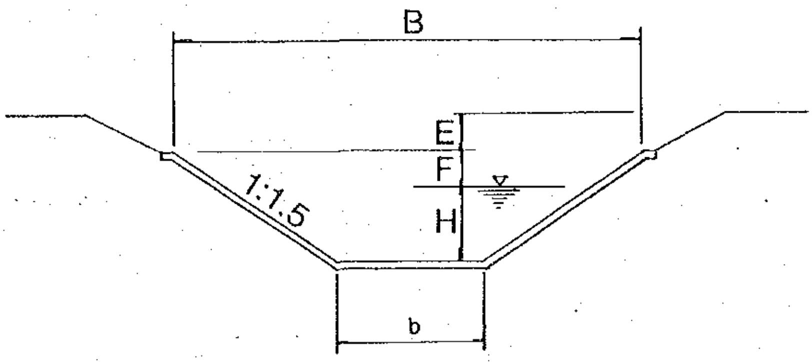
The cross sections of the main canals are designed according to the following criteria;

° Flow formula : Manning Formula
(n = 0.014)

° Canal section: Trapezoidal
(inside slope: 1:1.5)

° Other dimension:

Discharge Q (cu.m/sec)	Bed Width B (m)	Free Board F (m)	Earth Freeboard E (m)
8.5 -	2.70	0.35	0.50
5.0 - 8.5	2.70	0.30	0.45
3.5 - 5.0	2.20	0.25	0.40
1.0 - 3.5	1.80	0.20	0.40
1.0 or less	1.20	0.15	0.30



(4) Existing Canal

The existing canals of the Mae Pung Left and Right Canal are proposed to be improved and lined with concrete because their routes run through the "Mae Rim Series" of soils which is formed on old gravelly and cobbly alluvium.

3) Lateral Canals

(1) Routes of Lateral Canals

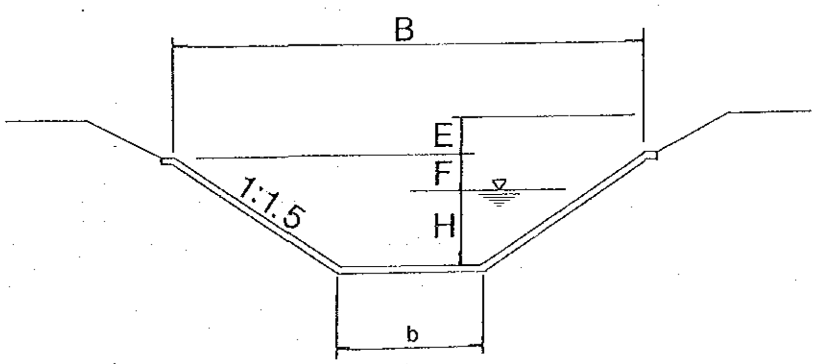
The routes are designed on 1:10,000 scale topographical maps in consideration of the following.

Table 4-4-1 Computation of the Main Canal Flow (Case 5)

Canal	Discharge (cu.m/sec)	b (m)	Slope (I)	H (m)	Velocity (m/sec)	F (m)	E (m)	B (m)	Length (km)
Mae Chang	10.52	2.70	1:7,000	1.99	0.93	0.35	0.50	9.75	1.60
Main Canal	10.45	2.70	1:7,000	1.98	0.93	0.35	0.50	9.75	2.80
	9.23	2.70	1:7,000	1.87	0.90	0.35	0.50	9.45	2.10
	8.17	2.70	1:7,000	1.76	0.87	0.30	0.45	9.00	1.00
	8.09	2.70	1:7,000	1.75	0.87	0.30	0.45	8.85	1.30
	7.41	2.70	1:7,000	1.67	0.85	0.30	0.45	8.70	0.70
	7.05	2.70	1:7,000	1.63	0.84	0.30	0.45	8.55	1.80
	6.70	2.70	1:7,000	1.59	0.83	0.30	0.45	8.40	1.40
									(12.70)
Mae Pung	3.26	1.80	1:2,000	0.93	1.11	0.20	0.40	5.25	0.80
Left Canal	2.96	1.80	1:2,000	0.88	1.07	0.20	0.40	5.10	2.50
	2.66	1.80	1:2,000	0.84	1.05	0.20	0.40	4.95	2.00
	2.23	1.80	1:2,000	0.76	1.00	0.20	0.40	4.80	1.20
									(6.50)
Mae Pung	3.44	1.80	1:6,000	1.26	0.75	0.20	0.40	6.30	0.90
Right Canal	3.24	1.80	1:6,000	1.22	0.74	0.20	0.40	6.15	1.00
	2.99	1.80	1:6,000	1.17	0.72	0.20	0.40	6.00	2.60
	2.91	1.80	1:6,000	1.16	0.72	0.20	0.40	6.00	1.80
	2.43	1.80	1:6,000	1.05	0.68	0.20	0.40	5.55	2.30
	1.78	1.80	1:5,000	0.86	0.67	0.20	0.40	5.10	1.30
	1.65	1.80	1:5,000	0.83	0.66	0.20	0.40	4.95	2.30
	1.25	1.20	1:5,000	0.83	0.62	0.20	0.40	4.35	0.80
									(13.00)
Sop Chang	1.50	1.80	1:4,000	0.74	0.69	0.20	0.40	4.65	2.30
Canal	1.33	1.80	1:4,000	0.70	0.67	0.20	0.40	4.50	1.40
	1.14	1.80	1:4,000	0.64	0.64	0.20	0.40	4.35	1.80
	1.01	1.80	1:4,000	0.60	0.62	0.20	0.40	4.20	1.80
	0.92	1.20	1:4,000	0.67	0.62	0.15	0.30	3.75	1.40
									(8.70)
Mae Wa	1.22	1.80	1:5,000	0.71	0.61	0.20	0.40	4.65	2.40
Canal	1.07	1.80	1:5,000	0.66	0.58	0.20	0.40	4.50	2.30
	0.99	1.20	1:4,000	0.70	0.64	0.15	0.30	3.75	1.30
	0.93	1.20	1:4,000	0.68	0.63	0.15	0.30	3.75	1.00
	0.86	1.20	1:4,000	0.65	0.61	0.15	0.30	3.60	0.50
	0.68	1.20	1:4,000	0.58	0.58	0.15	0.30	3.45	2.90
									(10.40)
							Total:		(51.30)

Table 4-4-2 Computation of the Main Canal Flow (Case 6)

Canal	Discharge (cu.m/sec)	b (m)	Slope (I)	H (m)	Velocity (m/sec)	F (m)	E (m)	B (m)	Length (km)
Mae Chang Main Canal	9.55	2.70	1:7,000	1.90	0.91	0.35	0.50	9.45	0.15
	8.65	2.70	1:7,000	1.80	0.89	0.35	0.50	9.15	2.95
	7.37	2.70	1:7,000	1.67	0.85	0.30	0.45	8.60	2.40
	6.92	2.70	1:7,000	1.62	0.84	0.30	0.45	8.45	1.15
	6.56	2.70	1:7,000	1.58	0.83	0.30	0.45	8.35	3.30
	6.15	2.70	1:7,000	1.53	0.81	0.30	0.45	8.20	1.75
									(11.70)
Mae Pung Left Canal	3.16	1.80	1:5,000	1.40	0.78	0.20	0.40	6.60	0.05
	2.87	1.80	1:5,000	1.30	0.76	0.20	0.40	6.30	2.75
	2.61	1.80	1:5,000	1.30	0.75	0.20	0.40	6.60	1.40
									(4.20)
Mae Pung Right Canal	2.99	1.80	1:5,000	1.12	0.77	0.20	0.40	5.75	2.10
	2.69	1.80	1:5,000	1.06	0.75	0.20	0.40	5.60	1.60
	2.61	1.80	1:5,000	1.05	0.75	0.20	0.40	5.55	2.00
	2.18	1.80	1:5,000	0.96	0.71	0.20	0.40	5.30	2.30
	1.72	1.80	1:4,000	0.80	0.72	0.20	0.40	4.80	1.20
	1.60	1.80	1:4,000	0.77	0.71	0.20	0.40	4.70	1.20
	1.22	1.80	1:4,000	0.67	0.66	0.20	0.40	4.40	1.95
	0.73	1.80	1:4,000	0.51	0.62	0.15	0.30	3.80	0.65
									(13.00)
Sop Chang Canal	1.46	1.80	1:4,000	0.73	0.69	0.20	0.40	4.60	3.70
	1.29	1.80	1:4,000	0.69	0.67	0.20	0.40	4.50	1.50
	1.10	1.80	1:4,000	0.64	0.64	0.20	0.40	4.30	1.20
	0.96	1.20	1:3,000	0.64	0.70	0.15	0.30	3.60	2.80
	0.68	1.20	1:3,000	0.54	0.64	0.15	0.30	3.30	1.90
									(11.10)
Mae Wa Canal	0.99	1.20	1:5,000	0.74	0.59	0.15	0.30	3.90	2.40
	0.87	1.20	1:5,000	0.69	0.56	0.15	0.30	3.75	2.30
	0.81	1.20	1:5,000	0.67	0.56	0.15	0.30	3.75	1.30
	0.76	1.20	1:4,000	0.61	0.59	0.15	0.30	3.60	1.00
	0.70	1.20	1:4,000	0.59	0.58	0.15	0.30	3.45	0.50
	0.55	1.20	1:4,000	0.52	0.54	0.15	0.30	3.30	2.90
									(10.40)
							Total:		(50.40)



a) The routes should cross the contourlines at approximatelly a right angle.

b) The existing irrigation canals should be utilized and improved in order to reduce land acquisition and make it easier to reorganize the existing water users group.

(2) Cross Section and Length of Canal

The typical cross section of lateral canals is shown in the attached drawing and design criteria are as follows:

Design Criteria for Lateral Canals

° Cross Section	:	Trapezoidal
° Lining	:	Earth Canal (Slope, 1:1.5)
° Hydraulic Formula	:	Manning Formula (n = 0.0225)
° Longitudinal Slope	:	1:3,000 - 1:1,000
° Free Board	:	F = 0.20 - 0.15 m
		E = 0.40 - 0.30 m
° Canal bed width	:	W = 1.2 m - 0.5 m

The soil characteristics of the regions of canals running through shows the poor or moderate permeability mostly. Some parts along the rivers, such as Mae Nam Wang, Nam Mae Chang and Huai Mae Pong belong to well drained soils but lateral canals seldom across there. Lateral canals is, therefore, designed as earth canal.

The total length of laterals is as follows:

Length of Laterals

	Number of Laterals		Total Length (km)	
Canals	Case-5	Case-6	Case-5	Case-6
MCMC	6	4	18.6	15.5
MPLC	4	3	11.6	13.5
MPRC	10	9	34.9	30.6
MWC	7	7	13.0	13.0
SCC	6	6	15.2	14.7
Total	33	29	93.3	87.3

4) Related Structures

(1) Diversions and Check-gates

The diversion works are facilitated at the distribution point from the main canals to the laterals and their number is equal to the number of laterals. Either single or double-gated types are to be used depending on the amount of discharge for the diversion.

The check-gate is designed at one point, namely at the end point of the MCMC to keep the water level in the main canal at certain specific levels for ensuring the distribution amounts to each canal.

(2) Siphon

The siphon works whose discharges are smaller than drainage discharge are designed to be constructed at points crossing major rivers and trunk roads in the region as follows:

Canal	Number of Siphons	
	Main Canal	Lateral Canal
MCMC	3	5
MPLC	2	3
MPRC	1	8
MWC	4	6
SCC	2	5
Total	12	27

The velocity in the syphon is designed to be 1.5 times that of the open canal for the purpose of preventing sedimentation in the syphon.

(3) Drainage Culvert and Inlet for Runoff

In case the drainage discharge is equal to or slightly less than canal design discharge, the crossing structure is designed as a drainage pipe culvert. On the other hand where drainage discharge is much less than that of canal discharge, inlet works are adopted to utilize the runoff water for irrigation without damage to canal facilities and under the following criteria;

Maximum Drainage Acreage for Inlet Work

	Land Utilization	
	Forest	Paddy
Main Canals	40 ha (0.97 m^3/sec)	200 ha (0.98 m^3/sec)
Laterals	20 ha (0.49 m^3/sec)	100 ha (0.49 m^3/sec)

(4) Bridge

The bridges for farm roads are proposed to be constructed of concrete at a rate of one bridge per one kilometer of irrigation canal, including the existing crosses with trunk roads.

4.4.5. Drainage Canals

The proposed drainage network is utilized the same as the existing Huai improved, such as uptrading and enlargement of trunk channels, removal of existing weirs and enlargement of crossing structures.

1) Upgrading of Trunk Channels

Upgrading works are composed of reducing the length of trunk channels to the Mae Chang River (in Huai Mae Tha) and linking the end of the irrigation canals to the trunk channels (in Huai Mae Phaet and Huai Mae Pung) as follows:

Name of Trunk Channels	Length (km)
Huai Mae Tha	0.7
Huai Mae Phaet	-
Huai Mae Pung	0.8
Total	1.5

2) Enlargement of Trunk Channels

The trunk channels which have been reduced in size at points of hairpin curves and narrow sections by sedimentary deposits and vegetational growth are proposed to be enlarged and reformed as follows:

Name of the Channels	Length (m)
Huai Mae Tha	1.3
Huai Mae Phaet	1.2
Huai Mae Pung	3.0
Name Mae Wa	-
Total	5.5

3) Removal of Existing Weirs

After completion of the proposed irrigation system, existing weirs in the trunk channels will be serious obstacles for the drainage system. These existing weirs are planned to be removed as follows:

Name of Channels	Number of Weirs (Places)
Huai Mae Tha	5
Huai Mae Phaet	2
Huai Mae Pung	5
Total	12

4) Enlargement of Existing Cross Structures

Several of the existing box culverts have sections which are smaller than required in comparison with the design discharge and these are designed to be enlarged as follows:

Name of Channels	Number of Cross Structures (Places)
Huai Mae Tha	2
Huai Mae Phaet	1
Huai Mae Pung	2
Total	5

4.4.6. Roads

Main and lateral canals are proposed in order to facilitate the operation and maintenance of roads paved with laterite and these roads should be sufficient enough to contribute to the increase in the density of the road network in the Project Area and convenience of transportation for the agricultural input and output. The standard cross section of roads is shown in the attached drawings.

4.5. Alternative Programmes of Compensation and Resettlement for the Submerged Areas

4.5.1. Affected Villages

1) The Number of the Affected Villages

The Area to be affected from the dam construction stretches out over six villages in Case 5 (storage dam A and diversion dam C or dam D). The name of the villages and their present conditions are extensively shown in Table 4.5-1.

Table 4.5-1. General Conditions of the Affected Villages

Mu Ban	No. of Farms	No. of Person	Area of (rai) Paddy	Area of (rai) Upland	Area of (rai) Sugar-cane	Area of (rai) To-bacco	No. of Animals(head) Buffa-loes	No. of Animals(head) Cows	No. of Animals(head) Pigs	Trac-tor	No. of Pumps	No. of Wells	No. of School	No. of Wat	Elec-tricity
Ban Mae Lu	22	100	200	200	10	5	50	100	-	3	1	1	1	-	x
Ban Kom	185	934	505	342	17	100	300	600	186	-	-	3	1	1	0
Ban Mai	280	1,281	491	267	-	80	500	300	200	10	-	12	1	1	0
Ban Pong Pa Pao	131	516	838	104	15	130	160	350	131	3	-	2	1	1	0
Ban Tung Ton	206	1,180	891	455	30	15	150	1,000	200	2	10	2	1	1	0
Ban Sop Po	53	207	150	100	35	10	50	150	16	-	-	2	1	-	x
Total	877	4,218	3,075	1,468	107	340	1,210	2,500	733	18	11	22	6	4	

Note: 1) The actual area cultivated by Ban Sop Po is 300 rai of paddy and 150 rai of upland fields.

2) The Number of pumps owned by two tobacco curing stations is excluded.

3) Source: Records of Mae Tha Agricultural Extension Office, supplemented by the field survey.

2) Seriousness of Each Affected Village

In Case 5, Ban Mae Lu will be completely inundated by dam A and the submerged houses, farm lands and others by diversion dam C will be as follows; in Ban Kom, five per cent of houses and farm land: in Ban Mai, 50 per cent of houses, 20 per cent of farm land, a school, a wat, a public health center, and a paved road and a electricity distribution line both one kilometer in length. In Ban Pong Pa Pao, 20 per cent of houses, 30 per cent of farm land and a wat: in Ban Tung Ton, 10 per cent of houses and 30 per cent of farm lands.

In case 6, besides Ban Mae Lu, Ban Sop Po will be slightly submerged by diversion dam D. Namely, about 10 percent of farm land located in the village area will be affected.

3) The Number of Families to be Compensated and Resettled

In both cases, all evacuees from Ban Mae Lu require resettlement somewhere. Concerning the four other partly affected villages, however, a decision is required on how to compensate or resettle according to their respective features. As for Ban Kom and Ban Sop Po both to be affected slightly, a single means of compensation may be applied. For Ban Mai, where the residential area will be mainly affected, about 40 per cent of the affected villagers shall be removed to a new settlement. For Ban Pong Pa Pao and Ban Tung Ton, where the main fertile farm land along the river will be flooded, all of the affected villagers shall be relocated. (See Table 4.5-2.)

4) Economic Benefits from the Affected Area

Annual losses of agricultural production caused by dam construction, which correspond to annual benefits coming from the affected area are counted as follows on the basis of the projected prices for 1990.

Case 5

Inundated Area	Economic Negative Benefit	= 2,504,000
Outputs		
Paddy field (841 rai)		(฿'000)
Rice (first crop)	325 kgx 4.94฿ x 841 rai =	1,350
Tobacco (second crop)	2,100 kgx 2.52฿ x 160 rai =	847
Upland field (438 rai)		
Mize (indicator)	300 kgx 3.38฿ x 438 rai =	444
Forest land		
Sugarcane	3,000 kgx 0.77 x 25 rai =	58
Animal production		
Cows	330 head x 4,000 x 1/4 per year=	330
Buffaloes	197 head x 5,000 x 1/4 per year=	246
Pigs	86 head x 2,000 x 2 times =	344
Total Output		3,619

Inputs

Animal Production (Cows=198,000 + Buffaloes=147,000 + Pigs=206,000 ฿)	=	551
Tobacco (fertiliyer, pesticide, tractor, pump, etc.)	=	288
Sugarcane (fertilizer)	=	7
Purchase of piglets	=	26
Paddy (seed, fertilizer, pumps, etc.)841raix289 ฿	=	243
Total Inputs		1,115

Case 5 Inundated Area Financial Negative Benefit ฿1,738,000

		(฿'000)
Output		
Paddy field (841 rai)		
Rice (first crop)	325 kg x 4.11 ฿ x 841 rai =	1,123
Tobacco (second crop)	2,100 kg x 1.51 ฿ x 160 rai =	508
Upland (438 rai)		
Maize (indicator)	300 kg x 2.99 ฿ x 438 rai =	393
Forest land		
Sugarcane	3,000 kg x 0.69 ฿ x 25 rai =	52
Animal production		
Cows	300 head x 3,000 ฿ x 1/4 per year =	248
Buffaloes	197 head x 4,000 ฿ x 1/4 per year =	197
Pigs	86 head x 1,500 ฿ x 2 times =	258
Total Output		2,778
Input		
Tobacco	160 rai x 2,047 ฿ =	328
Sugarcane	25 rai x 300 ฿ =	8
Small pigs	86 head x 350 ฿ =	30
Cows	300 head x 1,800 ฿ x 1/4=	149
Buffaloes	197 head x 2,400 ฿ x 1/4=	118
Pigs	86 head x 900 ฿ x 2 =	155
Paddy	841 head x 300 ฿ =	252
Total		1,040

Case 6 Inundated Area Government Negative Benefit ฿988,000

		(฿'000)
Output		
Paddy field (350 rai)		
Rice (first crop)	325 kg x 4.94 ฿ x 350 rai =	562
Tobacco (second crop)	2,100 kg x 2.52 ฿ x 20 rai =	106
Upland field (438 rai)		
Maize (indicator)	300 kg x 3.38 ฿ x 300 rai =	304
Forest land		
Sugarcane	3,000 kg x 0.77 ฿ x 14 rai =	32
Animal production		
Cows	100 head x 4,000 ฿ x 1/4 per year =	100
Buffaloes	50 head x 5,000 ฿ x 1/4 per year =	63
		1,167
Input		
Tobacco	20 rai x 1,797 ฿ =	36
Sugarcane	14 rai x 264 ฿ =	4
Others (see, etc.)		41
Animal production (Cows= 60 + Buffaloes= 38)	=	98
		179

Case 6 Inundated Area Financial Negative Benefit ฿683,000

			(฿'000)
Output			
Paddy field (350 rai)			
Rice (first crop)	325 x 4.11 ฿ x 350 rai	=	468
Tobacco (second crop)	2,100 kg x 1.51 x 20 rai	=	63
Upland (300 rai)			
Maize (indicator)	300 kg x 2.99 ฿ x 438 rai	=	393
Forest land			
Sugarcane	3,000 kg x 0.69 ฿ 14 rai	=	29
Animal production			
Cows	100 head x 3,000 ฿ x 1/4 per years	=	75
Buffaloes	50 head x 4,000 ฿ x 1/4 per years	=	50
Total Output			1,078

Input			
Tobacco	20 rai x 2,047 ฿	=	41
Sugarcane	14 rai x 300 ฿	=	4
Cows	100 head x 1,800 ฿	=	180
Buffaloes	50 head x 2,400 ฿	=	120
Others (seed, etc.)			50
Total			395

Strictly speaking, in addition to the above mentioned benefits, there needs to be taken into account some intangible benefits deriving from the living facilities.

Assuming that the annual value of the utility corresponds with five per cent of the value of houses and homelots, the value can be calculated as follows:

Table 4.5-2. Number of Families to be Compensated and Resettled

Villages	Compensated			Resettled		
	Storage Dam A	Diversion C	Diversion D	Storage Dam A	Diversion C	Diversion D
Ban Mae Lu	22	–	–	22	–	–
Ban Kom	–	10	–	–	–	–
Ban Mai	–	140	–	–	56	–
Ban Pong Pa Pao	–	26	–	–	26	–
Ban Thung Ton	–	21	–	–	21	–
Ban Sop Po	–	–	–	–	–	–
Total	22	197	0	22	103	0

		฿'000
Case 5.	219 familyx96,000฿(value of house)x1/20 =	1,051 "
	219 familyx 4,000฿(value of homelot)x1/20 =	44 "
	Total	1,095 "
Case 6.	22 familyx96,000฿(value of house)x1/20 =	106 "
	22 familyx 4,000฿(value of homelot)x1/20 =	4 "
	Total	110 "

4.5.2. Compensation

1) Compensation for Private Property

The number of families to be compensated is 219 for Case 5 and 22 for Case 6, likewise the acreage of farm land is 1,304 rai for Case 5 and 664 rai for Case 6 as shown in Table 4.5-3. Assuming that the market prices of paddy field, upland field, forest land (sugarcane) and homelot are 13,000, 5,000, 2,000 and 10,000 Baht per rai respectively, the estimated value of houses is 96,000 Baht (15 years old, 80 percent of a new building) per family, and the estimated value of trees is 800 Baht per family, total compensation money for both cases is computed as follows:

Case 5.			฿'000
	Paddy fields (841 rai)	10,933	"
	Upland fields (438 rai)	2,190	"
	Forest land (25 rai)	50	"
	Homelots (219 families)	876	"
	Trees (219 families)	176	"
	Total	14,224	"
Case 6.			฿'000
	Paddy fields (350 rai)	4,550	"
	Upland fields (300 rai).........	1,500	"
	Forest land (14 rai)	28	"
	Homelots (22 families)	88	"
	Trees (22 families)	18	"
	Total	6,184	

Table 4.5-3. Farm Land Area to be Compensated

(Unit: rai)

Villages	Storage Dam A			Diversion Dam C			Diversion Dam D		
	Paddy	Upland	Other	Paddy	Upland	Other	Paddy	Upland	Other
Ban Mae Lu	200	200	10	-	-	-	-	-	-
Ban Kom	-	-	-	25	17	1	-	-	-
Ban Mai	-	-	-	98	53	-	-	-	-
Ban Pong Pa Pao	-	-	-	251	31	5	-	-	-
Ban Thung Ton	-	-	-	267	137	9	-	-	-
Ban Sop Po	-	-	-	-	-	-	150	100	4
Total	200	200	10	641	238	15	150	100	4

2) Compensation for Public Property

Compensation for the public property to be relocated in the resettlement is appropriated in the construction costs as a replacement cost.

However, in Case 5, some public properties should be reconstructed in the vicinity of their original places. These are a school, a wat, a public health center and roads and electric lines in Ban Mai, and a wat in Ban Pong Pa Pao. They are roughly estimated below.

School	500,000	Baht
Two wats	3,000,000	"
Health center	300,000	"
Roads (1.0 km)	400,000	"
Electric lines (1.0 km)..	100,000	"
Total	4,300,000	"

4.5.3. Resettlement

1) Type of Farming and Agricultural Benefits

The type of farming is designed as follows:

Size of farm

Paddy	6 rai (irrigated by pumping)
Upland	2
Others	2 " (homelots, roads, ditches etc.)
Total	10 rai

Crop production

Rice	6 rai	600 kg per rai
Tobacco	1 "	2,500
Soybean	3 "	200
Groundnuts	2 "	250
Maize	1 "	300
Sugarcane	1 "	7,000 ton

Animal production

Cows	2 head	(yearly 1/2 head)
Pigs	1 "	(yearly 1 head)

Output	Financial (unit pc)		Economic (Unit pc)	
Rice (50% glutinous)	15,408	(4.28 ฿)	18,540	(5.15 ฿)
Tobacco	3,775	(1.51 ")	6,300	(2.52 ")
Soybeans	4,878	(8.13 ")	4,992	(8.32 ")
Groundnuts	3,055	(6.11 ")	3,190	(6.38 ")
Maize	897	(2.99 ")	1,014	(3.38 ")
Sugarcane	4,830	(0.69 ")	5,390	(0.77 ")
Cows	1,500	(3,000 ")	2,500	(5,000 ")
Pigs	1,500	(1,500 ")	2,000	(2,000 ")
Total	35,843 Baht		43,926 Baht	

Input

Water charge	900	900 (wet 300, dry 600)
Fertilizer	4,099	3,607
Chemicals	2,385	2,197
Oils	1,000	880
Tractor	4,000	3,520
Others	2,000	2,000
Total	14,384 Baht	13,104 Baht

Farm income (labor income)

Financial	21,459 Baht
Economic	30,822 Bath

Accordingly, the resettlement of 125 farms of Case 5 will give rise to a financial and an economic benefit of 2,682 and 3,853 thousand Baht respectively. Likewise, that of 22 farms will raise 472 and 678 thousand Baht, respectively.

2) Construction Costs

Resettlement areas of both cases are calculated below.

	Case 5	Case 6
Number of settlers	125	22
Distributed area	1,250 rai	220 rai
Roads & canals	125 "	22 "
Public use	25 "	12 "
Total Area	1,400 rai	254 rai

Construction costs are estimated at the following unit cost:

Land clearing ...... 300 Baht per rai
Road construction .. 400,000 Baht per km (100 m per farm and a connection road of 1 km)
Tertiary canal ..... 1,000 Bath per m (500 m for each Case)
Farm ditches .... 40 Bath per m (100 m per farm)
Pumping facilities . depending on the size of irrigable area
High voltage electricity.. 100,000 Baht per km (2 km)
Household electricity..... 9,800 Bath per farm
School .... 500,000 Baht
Wat .... 1,500,000 Baht
Wells .... 15,000 Baht per well

Therefore, the total costs are calculated as follows:

	Case 5 (฿'000)	Case 6 (฿'000)
Land clearing	37.5	6.6
Road	5,400.0	1,280.0
Canals	1,000.0	580.0
Pumps	2,500	1,000.0
Electricity	1,225.0	415.6
School	500.0	500.0
Wat	1,500.0	1,500.0
Wells	15.0	75.0
Total	12,177.5	5,357.2
(Say	12,200	5,400)

4.6. Project Cost

4.6.1. Basis of Estimation

There are two methods for the estimation of the Project cost; a force account basis and a contract basis.

The Project costs were estimated on the contract basis through international competitive bidding, based on the result of the study referred in Chapter 5.2.1.

The unit prices employed in the Project cost estimation were surveyed and studied in the Project area and RID Head Office in Bangkok in cooperation of RID office in Region II from June to July 1983.

4.6.2. Project Cost Estimate

1) Total Project Cost

The Project costs were estimated in detail for two cases, Case 5 and 6, which are expected to have high possibility for implementation studied in the alternative plan.

Case 5 : Storage Dam A + Diversion Dam C + Canal System
Case 6 : Storage Dam A + Diversion Dam D + Canal System

The total Project costs are summarized as follows;

	Total Cost	Foreign Currency	Local Currency
Case 5 :	฿ 1,017,820,000	฿ 497,670,000	฿ 520,150,000
	($ 44,253,000)	($ 21,638,000)	($ 22,615,000)
Case 6 :	฿ 927,700,000	฿ 481,170,000	฿ 446,530,000
	($ 40,335,000)	($ 20,920,000)	($ 19,415,000)

Note : 1) US one dollar ($1.00) is equivalent to twenty three Bath (฿23.0).
2) The total project cost includes the price escalation cost during the implementation period.

2) Components of the Project Cost

The Project cost consists of the following components.

(1) Civil Work Costs

Pre-Engineering : Cadastral survey, soil survey, geological investigation, quality control, etc.

Preparation	:	Access roads, transmission line, land clearing, field camps, office, technical supporting, precaution for safety, etc.
Storage Dam	:	Construction of dam body, foundation treatment, spillway and intake facilities.
Diversion Dam	:	Construction of embankment, spillway, foundation treatment, and intake facilities.
Main Canal	:	Excavation, embankment, concrete lining, canal structures, etc.
Lateral Canal	:	- ditto -
Improvement of Drainage Facilities	:	Improvement of existing drainage canals and other related facilities.

(2) Land Acquisition Costs and Compensation

Reservoir Area	:	Compensation for private properties, houses and public properties in the reservoir area.
Resettlement	:	Construction cost of resettlement.
Project Area	:	Land acquisition of project facilities and compensation for borrow site and quarry site areas.

(3)	Construction Equipment	:	Procurement cost of vehicles for construction management and peration and maintenance of the irrigation system.
(4)	Project Facilities	:	Construction of office, camps and others for implementation of the Projeot.
(5)	Project Administration	:	Administrative charge of Government staff to be engaged in the newly organized project office.
(6)	Consulting Services	:	Engineering cost for consultants of foreign and local experts for the implementation of detailed design and suppervision of the Project implementation.
(7)	Contingency	:	10 per cent of total cost for minor differences between actual and estimated quantities, unforeseeable difficulties in construction, possible change in plan and uncertainties in foundation conditions.
(8)	Price Escalation	:	Price escalation rates for both manufactured goods and civil works.

(i) Annual rates

Escalation Rate (%)

Year	F/C	L/C
1984	7.5	8.0
1985	7.0	8.0
1986 - 1987	6.0	7.0
1988 - 1990	6.0	6.0

Note: Prevailing price escalation rates being used by ADB.

(ii) Escalation rates for disbursement schedule

	1984	1985	1986	1987	1988	1989	1990
Foreign Cost(%)	3.6	10.8	17.5	24.5	32.0	39.9	48.3
Local Cost (%)	3.9	12.2	20.0	28.4	36.1	44.3	53.0

3) Unit Cost

The cost of construction works is estimated on the basis of the prevailing unit costs in Mae Kuang Dam Project and Mae Wang Project as of March 1983. The construction works are considered to be carried out by a contract basis through the international competitive bidding. The unit prices used for estimation of the Project cost and the components are as follows:

(1) Labor Unit Prices

Description	Unit	Price(฿)
Labor	day	70
Foreman (First Class)	"	180
" (Second Class)	"	130
" (Third Class)	"	115
" (Fourth Class)	"	95

Description	Unit	Price(฿)
Driver	"	100
Operator	"	120
Carpenter	"	100
Head of Carpenter	"	150
Mason	"	60
Head of Mason	"	70
Steel Worker	"	80
Head of Steel Worker	"	100
Mechanic	"	200
Head of Mechanic	"	300

(2) Unit Prices of Materials

Description	Unit	Price(฿)	Remarks
Portland Cement	bag	92.5	1 bag = 50 kg
White Cement	"	260	
Reinforcing Steel Bar	kg	10	
Wire for Binding	"	14	
Gasoline		11.48	
Diesel Oil	"	7.37	
Sand	m^3	30	Excluded Trans-portion
Aggregate for Concrete	"	160	"
Aggregate of 3/8 inch	"	135	"
Crasher Run	"	95	"
Wood (Soft)	"	6,000	
Wood (Hard)	"	9,000	
Nail	kg	14	
Dynamite	PCS	16	1 PCS = 100 kg
Detonation Cap	"	25	
Stone of ø 30 cm	m^3	120	
Brick	PCS	0.2	
Others			
Electric Charge	KWH	2.2	

(3) Proportion of Foreign and Local Currencies

Proportion of the foreign and local currencies for the Project cost was applied according to the following table, referring to actual figures used in the Projects by International Bank for Reconstruction and Development (IBRD), Asian Development Bank (ADB) and Japan International Cooperation Agency (JICA).

Description	Percentage (%) F/C	L/C	
Cement	60	40	
Steel Bar	70	30	
Lumber	20	80	
Fuel & Oil	80	20	
Labor	-	100	
Explosive	80	20	
Construction Equipment			
Depreciation Cost	100	-	
Repair Cost	80	20	Including spare parts
Administrative Cost	-	100	

(4) Component of Unit Costs

The unit costs consist of the following components.

Labor Cost : Wages of labourers, foremen, drivers and other workers.

Material Cost : Cement, steel bar and other construction materials, and fuel and oil, electric fee.

Depreciation Cost of Construction equipment: Depreciation, repair and administrative costs.

Temporary Work Cost :	Temporary work costs excluding the preparatory work costs.
Overhead Cost :	Contractors overhead and profit estimated by RID criteria. 20 per cent of the unit costs except for depreciation cost of equipment is taken up as follows:

Description	Rate (%)
(1) Profit	6.5
(2) Administrative	3.5
(3) Tax	3.4
(4) Cost reserved	4.1
(5) Insurance	1.5

$$\text{Rate of Overhead} = \frac{\{1 + (1)\}\{1 + (2)\}}{\{1 - (3)\}\{1 - (4)\}\{1 - (5)\}} \fallingdotseq 1.20$$

4.6.3. Project Cost

The investment of the Project cost is estimated based on the above-mentioned criteria and summarized in Table 4.6-1.

Detailed information on the Project cost estimate is shown in Table 4.6-1 and 4.6-2, Appendices 4.6-1 and 4.6-2.

Table 4.6-1 Investment Cost of the Project

Description	Case 5 Total ₿ '000	Case 5 Total $ '000	Case 5 Foreign C ₿ '000	Case 5 Local C ₿ '000	Case 6 Total ₿ '000	Case 6 Total $ '000	Case 6 Foreign C ₿ '000	Case 6 Local C ₿ '000
1. Civil Works (Sub-total)	523,780	22,773	280,600	243,180	496,200	21,573	269,280	226,920
1-1. Pre-engineering	10,000	435	1,000	9,000	10,000	435	1,000	9,000
1-2. Preparation	39,640	1,723	12,430	27,210	38,030	1,653	12,140	25,890
1-3. Storage Dam A	208,930	9,084	141,960	66,970	208,930	9,084	141,960	66,970
1-4. Diversion Dam C or D	76,480	3,325	45,280	31,200	67,660	2,942	39,000	28,660
1-5. Main Canal	116,920	5,083	47,190	69,730	106,300	4,621	45,790	60,510
1-6. Lateral Canal	65,520	2,849	28,970	36,550	58,990	2,564	25,620	33,370
1-7. Improvement of Drainage Facilities	6,290	274	3,770	2,520	6,290	274	3,770	2,520
2. Land Acquisition & Compensation (Sub-total)	59,760	2,598	-	59,760	21,000	913	-	21,000
2-1. Reservoir Area	39,700	1,726	-	39,700	8,400	365	-	8,400
2-2. Resettlement	12,200	530	-	12,200	5,400	235	-	5,400
2-3. Project Area	7,860	342	-	7,860	7,200	313	-	7,200
3. Construction Equipment	6,000	261	6,000	-	6,000	261	6,000	-
4. Project Facilities	10,000	435	-	10,000	10,000	435	-	10,000
5. Project Administration	16,100	700	-	16,100	16,100	700	-	16,100
6. Consulting Services	89,000	3,870	55,000	34,000	89,000	3,870	55,000	34,000
Total (1 to 6)	704,640	30,637	341,600	363,040	638,300	27,752	330,280	308,280
7. Contingency	70,460	3,063	34,160	36,300	63,830	2,775	33,030	30,800
Total (1 to 7)	775,100	33,700	375,760	399,340	702,130	30,527	363,310	338,820
8. Price Escalation	242,720	10,553	121,910	120,810	225,570	9,808	117,860	107,710
Grand Total	1,017,820	44,253	497,670	520,150	927,700	40,335	481,170	446,530

CHAPTER V : PROJECT IMPLEMENTATION AND OPERATION

CHAPTER V. PROJECT IMPLEMENTATION AND OPERATION

5.1. Project Organization

5.1.1. Executing Agency

The major works of the Project are to construct a storage dam, a diversion dam and main and lateral irrigation canals, canal structures and drainage facilities. The contract basis implementation is recommended for implementation of the Project works involving voluminous quantities of the works.

RID is responsible for the overall planning, programming and execution of irrigation and major flood control projects in the country and will be an executing agency for the implementation of the project, with the assistance and cooperation of other Government agencies concerned in their respective fields.

5.1.2. Project Office

The project office will be provided in the project site for securing smooth execution of the works under the supervision of the Project Manager. The Office will consist of administrative division, engineering division, and the two branch offices of dam and canal construction works in the respective construction sites.

The administrative diversion will consist of administrative, accounting, and land acquisition/compensation sections being responsible for budgeting, accounting personnel matters, negotiation of land acquisition and other miscellaneous matters.

The engineering diversion will consist of laboratory, survey, design, construction and mechanical sections, being responsible for various testing of soil and concrete from the viewpoints of quality control, survey, detail design, cost estimation, operation and maintenance of equipment and others.

The two branch offices will be constructed independently from the project office as mentioned above. The dam office will consist of the two section, of a storage dam and a diversion dam. The canal office will consist of the zone-based five section according to first of the five classifications. The major works of the branch office will be construction supervisions, such as controlling the construction schedules, material qualities and progress.

The proposed organization chart of project execution is shown in the Fig. 5-1.

5.2. Construction Method and Schedule

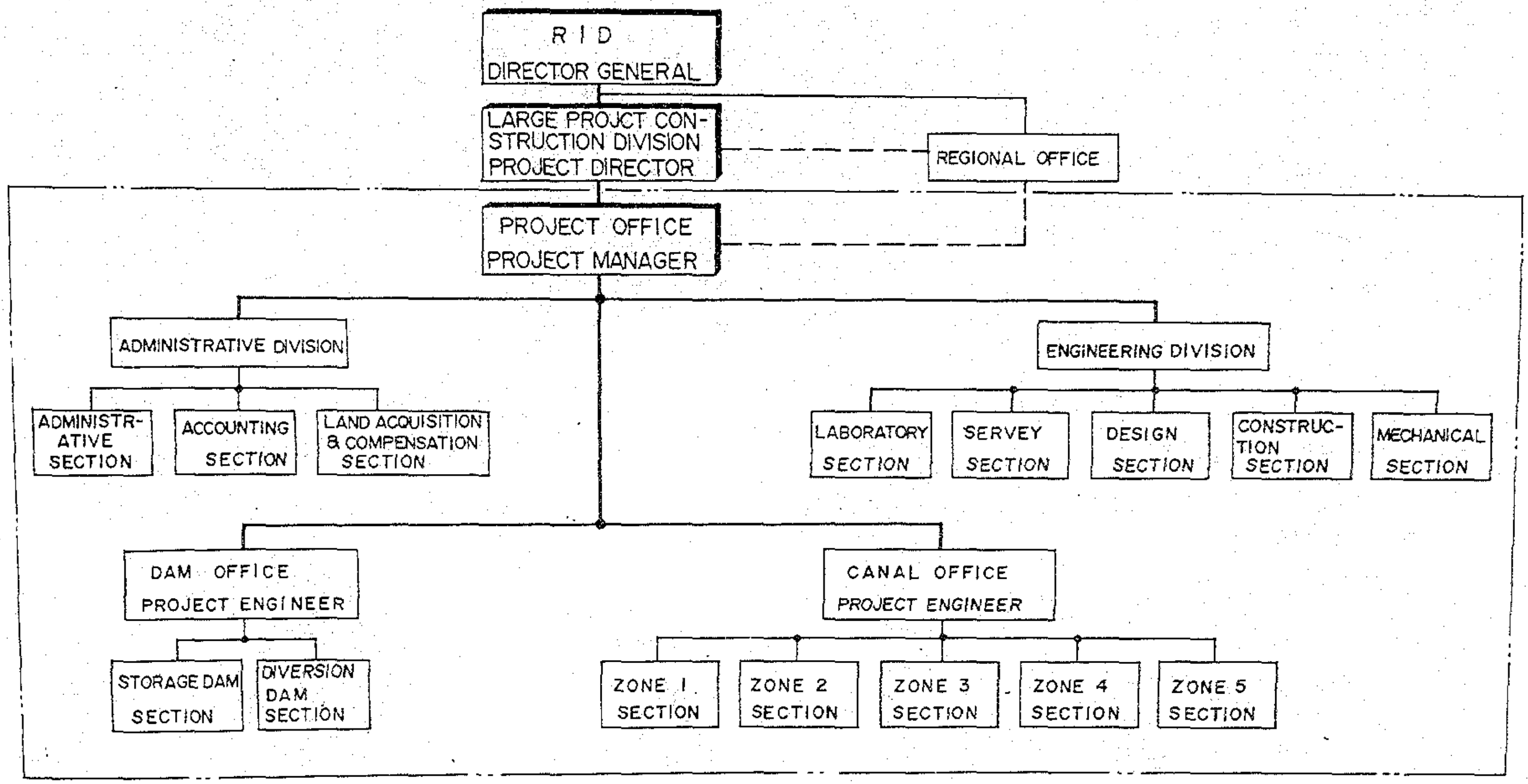
5.2.1. Mode of Construction

As for the project execution, the two modes of force account basis under RID and contract basis can be considered.

The contract basis will be adopted for the Project through the international competitive bidding in taking the following matters into consideration.

1) The project cost on the force account basis may be estimated lower than that on the contract basis due to no-overhead costs of contractors. However, if the force account basis were taken for the project construction, many engineers, labors, construction equipment and materials, in particular, experts and special equipment would be provided temporarily the RID according to characteristic features of the project. After completion of the project, it would be difficult

FIG. 5 -1 PROJECT ORGANIZATION CHART FOR IMPLEMENTION



to appropriate those equipment and personnel to any other project effectively. The RID will have to provide personal expenses and equipment maintenance fees during the period between projects (idling period). Therefore, the force account basis is not economical from the viewpoint of the total cost of the Project.

Although the force account basis in the project construction had been taken in the initial stage economic development even in advanced countries for promoting employment, the government budget has become oppressed financially due to annual increase in maintenance costs. Therefore, most projects except special cases have recently come to be constructed on a contractor basis in Thailand.

2) The capability of private firms will be very useful for the further development of Thai economy in the future. The technology level kept by Thai private firms for public works will be increased by executing the works on the contract basis.

5.2.2. Construction Method

1) Present Conditions

The present conditions of the implementing facilities provided in and around the Project Area are shown in Fig.5.1-2, Appendix.

i) Transmission line ; The three high voltage transmission lines (11,000V at present, 22,000V in future) are provided in the Project Area. One of then extends about three kilometer nearly the dam site A, crossing the Project Area in branching off at the national highway No.1. According to the information of the Lampang electric office, it would be possible to share the power more than 500 KW for the Project, and the electric power supply will be secured for the construction works.

ii) Supply of aggregate

Sand: Fine aggregates for concrete and filter materials for dam are obtained from the river deposits of the Mae Nam Wang. There exist several sand borrow pits along the Mae Nam Wang at approximately 4.5 km south-east of Lumpang city. One of them, Mong Kon Co, has a big production capacity to supply more than 300 m^3 of sand per day.

Coarse aggregate: Coarse aggregates for concrete are produced from the Mesozoic lime stone existing in the Project Area. There are two quarry sites for concrete materials. One is located in the site behind the Mae Tha railway station and another is along the national highway No.1 in Ban Wang Phrao. The former will have to produce by man-power, although located near the Project site, whereas the latter can produce more than $400m^3$ of coarse aggregates per day, even though requiring rather long transportation to the Project site.

2) Workable Days

Earth works are mostly affected by rainfall. Thereby, monthly mean workable days in the construction period are estimated by using the daily rainfall records of recent five years at the Gaging station 1615 in the vicinity of Dam site C.

a) Criteria

The workable days are estimated on the basis of the following criteria.

Daily Rainfall Intensity (mm/day)		
Normal Works	Dam Embankment of Impervious Zone	Suspension of Work (day)
0 - 10	0 - 3	Workable day
10.1 - 30	3.1 - 10	1 day
30.1 - 50	10.1 - 30	2 day
50.1 - 100	30.1 - 50	3 day
100.1 and over	50.1 or and over	4 day

The case of suspension less than 10 days per month ;

workable days per month

= 30 - (5 + suspension days/2)

The case of suspension more than 10 days per month ;

workable days per month

= 30 - (suspension days)

b) Workable Days

From the result of the above, study the workable days of each works are as follows;

Description	Workable Day per Month	
	Wet Season (May to Oct.)	Dry Season (Nov. to Apr.)
Normal Works	21	25
Dam Embankment of Impervious Zone	16	25

3) Storage Dam

a) Sequence of Construction Works

The construction of storage dam A will be divided into two sections, right and left abutments, due to the wide river bed. The sequence of construction work is planned as follows; (refer to Fig. 5.1-2, Appendix.)