8.2. Crop Water Requirement, ETcrop

The crop water requirement ETcrop is calculated from the evapotranspiration of the crop mentioned in the preceeding paragraph, by means of the following expression.

$$ETcrop = Kc \cdot ETo$$

where

Kc: Crop coefficient

The values of the crop coefficient Kc applicable to paddy in the humid areas of Asia are as follows, according to the FAO.

	Wet season		Dry season	
	Light to mod. wind	Strong wind	Light to mod. wind	Strong wind
First month	1.1	1.15	1.1	1.15
Second month	1.1	1.15	1.1	1.15
Mid-season	1.05	1.1	1.25	1.35
Last 4 weeks	0.95	1.0	1.0	1.05

Numerical data regarding the wind velocity classification adopted in the table above are as follows.

Light wind $\leq$ 175Km/day

Moderate wind = 175 through 425Km/day

The wind velocity occurred in the project area during the period of study was of the order of 4Knot (=178 Km/day) through 8Knot (355Km/day), corresponding therefore to the classification of breeze - weak wind presented in the table above.

The crop water requirement is calculated in accordance with the procedure described above. Next are calculated the area factor of each month from the cropping pattern and also the corresponding crop water requirement. The obtained results are listed in Table 8.2.1.

8.3. Field Water Requirement

The field water requirement is obtained by summing up the water requirement for land preparation and nursery, the supplying water for cultivation and the percolation.

(1) Deep Percolation

The NIA carried out measurements of deep percolation at 6 points in the project area and the location of the said points are indicated in Figure 8.3.1. The soil conditions and the deep percolation taking place at each one of the aforesaid measurement points are listed in Table 8.3.1. The measured values of deep percolation range within the limits of 0.69 through 1.10 mm/day, with an average of 0.88 mm/day.

(2) Field Water Requirement

The values of the field water requirement calculated in accordance with the method described above are listed in Table 8.2.1.

8.4. Net Farm Requirement

The net farm requirement is calculated by adding the crop water requirement with the field water requirement and subtracting the effective rainfall.

(1) Effective Rainfall

The effective rainfall is calculated by means of the "Paddy Operation Study". In this method, the rainfall taking place on the paddy area is taken as inflow, the crop water requirement is taken as run-off and the difference between them is either stored in the paddy area or covered by consuming water stored therein. In this case, water is stored in the paddy within the upper and lower limits of the supplying required for the sake of cultivation. When the upper limit of the prescribed flooding depth is exceeded, the part of water in excess is discharged to the drainage waterways, while when the water level is below the lower limit there is replenishment of irrigation water in order to ensure the lower limit value. The effective rainfall is calculated by subtracting the quantity of replenished water calculated by means of this method from the crop water requirement. This calculation is carried out in accordance with the contents of the Table 8.4.1 and the obtained results are as follows.

	1967		1968		1969		1970	
	R	Re	R	Re	R	Re	R	Re
JAN.			3	3	16	11	42	37
FEB.			2	2	20	20	T	0
MAR.			1	1	13	13	37	12
APR.			40	1	53	8	67	9
MAY	152	77	180	95	327	79		
JUNE	1,099	211	237	219	432	254		
JULY	581	210	523	220	1,045	251		
AUG.	968	182	1,013	182	826	227		
SEP.	355	92	520	96	656	89		
OCT.	376	4	128	4	222	4		
NOV.	163	86	25	25	61	25		
DEC.	2	2	2	2	1	1		

8.5. Diversion Water Requirement

The diversion water requirement is calculated from the net farm requirement by means of the following expression, which takes into account the overall efficiency.

(Diversion water requirement) = (Net farm requirement)/(Overall efficiency)

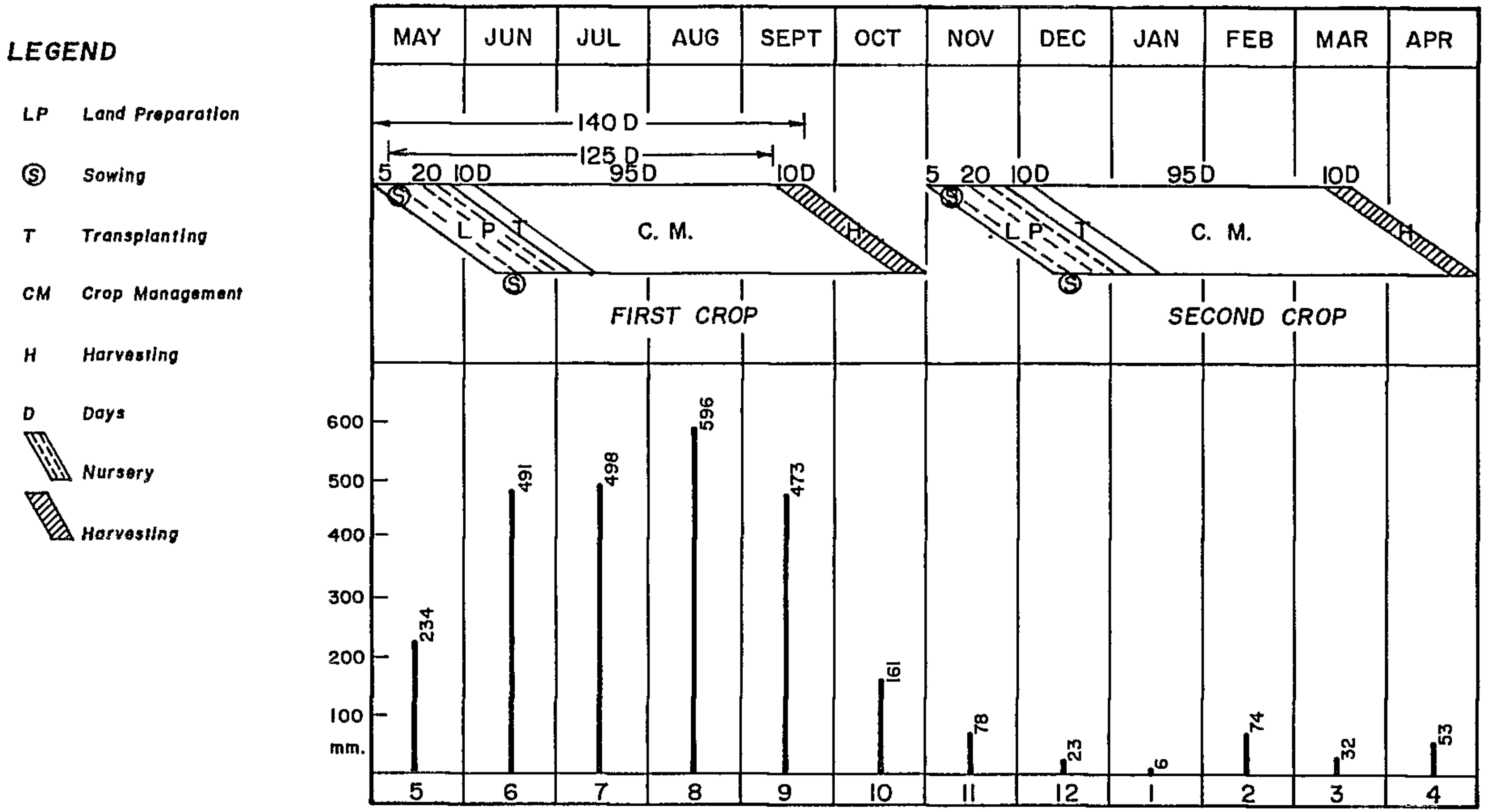
The values of the diversion water requirement calculated by means of the expression above are presented in Table 8.2.1.

Table 8-1-1 Evaporation (By Penman Method)
Dagupan City
Unit: mm

YEAR	JAN.	FEB.	MAR.	APR.	MAY	JUNE	JULY	AUG.	SEPT.	OCT.	NOV.	DEC.	TOTAL
1958	158.1	120.4	213.9	228.0	220.1	138.0	139.5	145.7	129.0	167.4	135.0	145.7	1,940.8
59	-	-	-	-	-	-	-	-	-	-	-	-	-
60	139.5	153.7	217.0	198.0	198.4	183.0	170.5	136.4	132.0	161.2	156.0	161.2	2,006.9
61	151.9	173.6	217.0	246.0	210.8	174.0	142.6	145.7	132.0	145.7	147.0	142.6	2,028.9
62	151.9	170.8	201.5	213.0	223.2	171.0	124.0	167.4	192.0	155.0	156.0	158.1	2,083.9
63	151.9	170.8	198.4	240.0	235.6	132.0	139.5	167.4	132.0	173.8	174.0	151.9	2,067.3
64	158.1	171.1	189.4	258.0	210.8	168.0	176.7	136.4	132.0	151.9	108.0	136.4	1,996.8
65	155.0	169.4	204.6	228.0	167.4	133.5	111.6	130.2	123.0	139.5	135.0	161.2	1,858.4
66	158.1	156.8	232.5	231.0	164.3	174.0	158.1	164.3	114.0	155.0	120.0	124.0	1,952.1
67	145.7	170.8	204.6	210.0	204.6	156.0	145.7	117.8	135.0	161.2	159.0	158.1	1,966.5
68	155.0	176.9	207.7	219.0	198.4	183.0	148.8	117.8	144.0	167.4	153.0	164.3	2,029.3
69	158.1	162.4	220.1	219.0	210.8	171.0	148.8	161.2	129.0	167.4	147.0	136.4	2,031.2
1970	148.8	162.4	204.6	222.0	223.2	141.0	142.6	111.6	135.0	145.7	138.0	133.3	1,908.2
MEAN	152.7	163.3	209.3	226.0	205.6	160.4	145.7	141.8	135.8	157.6	144.0	147.8	1,989.2

Fig. 8.2.1 POTENTIAL CROPPING PATTERN

Maturity - 125 day



NORMAL RAINFALL, MABINI (20 years of record), prepared by WEATHER BUREAU

Table 8.2.1 COMPUTATION OF DIVERSION REQUIREMENT (1)

		1967							1968				
		MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	JAN	FEB	MAR	APR
(1) CONSUMPTIVE USE	(MM)	204.6	156.0	145.7	117.8	135.0	161.2	159.0	158.1	155.0	176.9	207.7	219.0
(2) CROP COEFFICIENCY		1.10	1.10	1.10	1.05	0.95	0.95	1.10	1.10	1.10	1.25	1.00	1.00
(3) CROP WATER REQUIREMENT	(MM)	225.1	171.6	160.3	123.7	128.2	153.1	174.9	173.9	170.5	221.1	207.7	219.0
(4) AREA FACTOR OF C.W.R.		0.01	0.47	0.97	0.98	0.49	0.02	0.01	0.46	0.97	0.99	0.54	0.03
(5) WEIGHTED C.W.R.	(MM)	2.3	80.7	155.5	121.2	62.8	3.1	1.7	80.0	165.4	218.9	112.2	6.6
(6) AREA FACTOR OF L.P. AND N.		0.33	0.46	0.03	0.0	0.0	0.0	0.32	0.46	0.03	0.0	0.0	0.0
(7) WATER REQUIREMENT FOR L.P./N	(MM)	114.4	136.2	8.6	0.0	0.0	0.0	92.5	133.4	8.6	0.0	0.0	0.0
(8) FLOODING FOR CULTIVATION	(MM)	0.0	20.0	50.0	0.0	0.0	0.0	0.0	20.0	50.0	0.0	0.0	0.0
(9) DEEP PERCOLATION	(MM)	1.0	28.0	60.0	61.0	29.0	1.0	1.0	29.0	60.0	57.0	33.0	2.0
(10) FIELD WATER REQUIREMENT	(MM)	115.4	184.2	118.6	61.0	29.0	1.0	93.5	182.4	118.6	57.0	33.0	2.0
(11) TOTAL WATER REQUIREMENT	(MM)	117.6	264.8	274.1	182.2	91.8	4.1	95.2	262.4	284.0	275.9	145.2	8.6
(12) EFFECTIVE RAINFALL	(MM)	76.7	211.4	219.9	182.2	91.8	4.1	85.7	1.8	3.3	1.8	0.8	1.0
(13) NET FARM REQUIREMENT	(MM)	41.0	53.4	54.2	0.0	0.0	0.0	9.5	260.6	280.7	274.1	144.4	7.6
(14) OVERALL EFFICIENCY	(PERCENT)	50.0	50.0	50.0	50.0	50.0	50.0	58.0	58.0	58.0	58.0	58.0	58.0
(15) DIVERSION REQUIREMENT	(MM)	81.9	106.8	108.3	0.0	0.0	0.0	16.4	449.4	484.0	472.6	248.9	13.1

* NOTES *

(3)=(1)*(2)
(5)=(3)*(4)
(7)=(6)*((1)+(2*(NUMBER OF DAYS))+(80(WET) OR 70(DRY)))
(10)=(7)+(8)+(9)
(11)=(5)+(10)
(12)=(BY PADDY-OPERATION-STUDY)
(13)=(11)-(12)
(15)=(13)/((14)/100)

COMPUTATION OF DIVERSION REQUIREMENT (2)

		1968								1969			
		MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	JAN	FEB	MAR	APR
(1) CONSUMPTIVE USE	(MM)	198.4	183.0	148.8	117.8	144.0	167.4	153.0	164.3	158.1	162.4	220.1	219.0
(2) CROP COEFFICIENCY		1.10	1.10	1.10	1.05	0.95	0.95	1.10	1.10	1.10	1.25	1.00	1.00
(3) CROP WATER REQUIREMENT	(MM)	218.2	201.3	163.7	123.7	136.8	159.0	168.3	180.7	173.9	203.0	220.1	219.0
(4) AREA FACTOR OF C.W.R.		0.01	0.47	0.97	0.98	0.49	0.02	0.01	0.46	0.97	0.99	0.54	0.03
(5) WEIGHTED C.W.R.	(MM)	2.2	94.6	158.8	121.2	67.0	3.2	1.7	83.1	168.7	201.0	118.9	6.6
(6) AREA FACTOR OF L.P. AND N.		0.33	0.46	0.03	0.0	0.0	0.0	0.32	0.46	0.03	0.0	0.0	0.0
(7) WATER REQUIREMENT FOR L.P./N	(MM)	112.3	148.6	8.7	0.0	0.0	0.0	90.6	136.3	8.7	0.0	0.0	0.0
(8) FLOODING FOR CULTIVATION	(MM)	0.0	20.0	50.0	0.0	0.0	0.0	0.0	20.0	50.0	0.0	0.0	0.0
(9) DEEP PERCOLATION	(MM)	1.0	28.0	60.0	61.0	29.0	1.0	1.0	29.0	60.0	57.0	33.0	2.0
(10) FIELD WATER REQUIREMENT	(MM)	113.3	196.6	118.7	61.0	29.0	1.0	91.6	185.3	118.7	57.0	33.0	2.0
(11) TOTAL WATER REQUIREMENT	(MM)	115.5	291.2	277.5	182.2	96.0	4.2	93.2	268.4	287.4	258.0	151.9	8.6
(12) EFFECTIVE RAINFALL	(MM)	95.2	218.8	220.4	182.2	96.0	4.2	24.9	1.8	10.5	19.8	13.2	7.6
(13) NET FARM REQUIREMENT	(MM)	20.4	72.4	57.1	0.0	0.0	0.0	68.3	266.6	276.9	238.2	138.7	1.0
(14) OVERALL EFFICIENCY	(PERCENT)	50.0	50.0	50.0	50.0	50.0	50.0	58.0	58.0	58.0	58.0	58.0	58.0
(15) DIVERSION REQUIREMENT	(MM)	40.7	144.9	114.2	0.0	0.0	0.0	117.8	459.7	477.4	410.6	239.1	1.6

* NOTES *

(3)=(1)*(2)
(5)=(3)*(4)
(7)=(6)*((1)+(2*(NUMBER OF DAYS))+(80(WET) OR 70(DRY)))
(10)=(7)+(8)+(9)
(11)=(5)+(10)
(12)=(BY PADDY-OPERATION-STUDY)
(13)=(11)-(12)
(15)=(13)/((14)/100)

COMPUTATION OF DIVERSION REQUIREMENT (3)

		1969								1970			
		MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	JAN	FEB	MAR	APR
(1) CONSUMPTIVE USE	(MM)	210.8	171.0	148.8	161.2	129.0	167.4	147.0	136.4	148.8	162.4	204.6	222.0
(2) CROP COEFFICIENCY		1.10	1.10	1.10	1.05	0.95	0.95	1.10	1.10	1.10	1.25	1.00	1.00
(3) CROP WATER REQUIREMENT	(MM)	231.9	188.1	163.7	169.3	122.5	159.0	161.7	150.0	163.7	203.0	204.6	222.0
(4) AREA FACTOR OF C.W.R.		0.01	0.47	0.97	0.98	0.49	0.02	0.01	0.46	0.97	0.99	0.54	0.03
(5) WEIGHTED C.W.R.	(MM)	2.3	88.4	158.8	165.9	60.0	3.2	1.6	69.0	158.8	201.0	110.5	6.7
(6) AREA FACTOR OF L.P. AND N.		0.33	0.46	0.03	0.0	0.0	0.0	0.32	0.46	0.03	0.0	0.0	0.0
(7) WATER REQUIREMENT FOR L.P./N	(MM)	116.4	143.1	8.7	0.0	0.0	0.0	88.6	123.5	8.4	0.0	0.0	0.0
(8) FLOODING FOR CULTIVATION	(MM)	0.0	20.0	50.0	0.0	0.0	0.0	0.0	20.0	50.0	0.0	0.0	0.0
(9) DEEP PERCOLATION	(MM)	1.0	28.0	60.0	61.0	29.0	1.0	1.0	29.0	60.0	57.0	33.0	2.0
(10) FIELD WATER REQUIREMENT	(MM)	117.4	191.1	118.7	61.0	29.0	1.0	89.6	172.5	118.4	57.0	33.0	2.0
(11) TOTAL WATER REQUIREMENT	(MM)	119.7	279.5	277.5	226.9	89.0	4.2	91.3	241.5	277.2	258.0	143.5	8.7
(12) EFFECTIVE RAINFALL	(MM)	79.0	253.7	251.4	226.9	89.0	4.2	24.8	0.5	37.3	0.0	12.4	8.7
(13) NET FARM REQUIREMENT	(MM)	40.8	25.7	26.1	0.0	0.0	0.0	66.4	241.0	239.9	258.0	131.1	0.0
(14) OVERALL EFFICIENCY	(PERCENT)	50.0	50.0	50.0	50.0	50.0	50.0	58.0	58.0	58.0	58.0	58.0	58.0
(15) DIVERSION REQUIREMENT	(MM)	81.5	51.4	52.2	0.0	0.0	0.0	114.5	415.5	413.6	444.8	226.1	0.0

* NOTES *

(3)=(1)*(2)
(5)=(3)*(4)
(7)=(6)*((1)+(2*(NUMBER OF DAYS))+(80(WET) OR 70(DRY)))
(10)=(7)+(8)+(9)
(11)=(5)+(10)
(12)=(BY PADDY-OPERATION-STUDY)
(13)=(11)-(12)
(15)=(13)/((14)/100)

Fig.8.3.1 Percolation Test Site

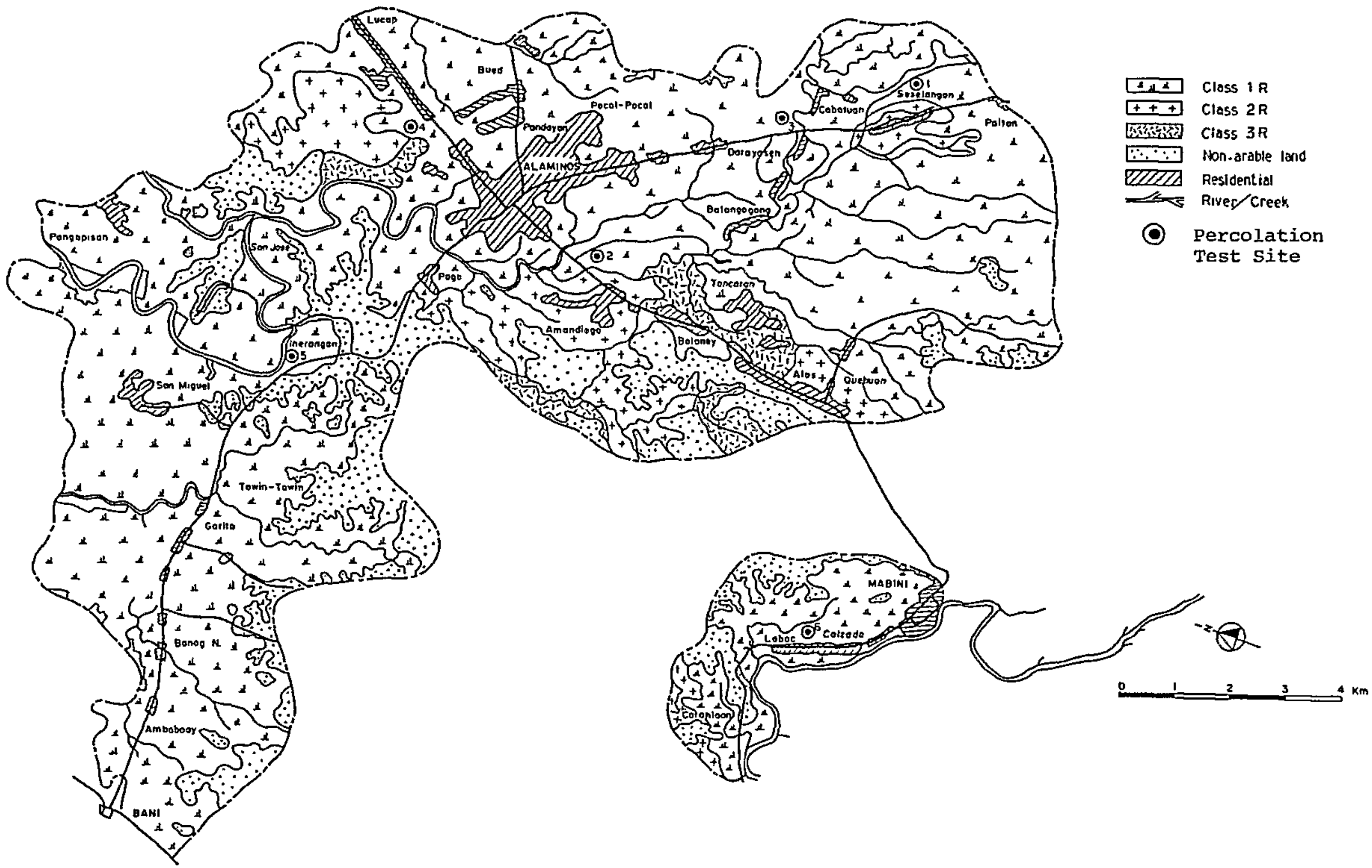


Table 8-3-1 SUMMARY OF DEEP PERCOLATION TEST[1/]
FOR RICE CLASS LANDS

TEST SITE NUMBER	LOCATION	NUMBER OF HOURS OBSERVED	PRESENT LAND USE	SOIL TEXTURE (Surface and Underlying Subsoil)	PERCOLATION RATE (m/day)
1.	Sosolangin, Sual Pangasinan	62.75	Pr	(0-20) SL/ (20-60) C	1.10
2.	Tococ, Alaminos, Pangasinan	66.58	Pr	(0-15) CL/ (15-65) CL	0.82
3.	Cabatuan, Alaminos, Pangasinan	64.16	Pr	(0-25) sic/ (25-60) C	0.69
4.	Magsaysay, Alaminos, Pangasinan	72.75	Pr	(0-10) siCL/(10-60) C	0.76
5.	Inerangan, Alaminos, Pangasinan	47.75	Pr5	(0-20) c / (20-60) C	1.05
6.	Lubao, Mabini, Pangasinan	47.25	Pr	(0-15) c / (15-60) C	0.88
T O T A L					5.30
AVERAGE					0.88

1/ Conducted from March - April, 1981

Table 8.4.1 PADDY OPERATION STUDY (1)

1967

DAY	MAY RAIN-FALL	MAY FIELD REQ.	MAY SUPPLIED	MAY SOIL MOIST. CONTENT	MAY SPILL	JUN RAIN-FALL	JUN FIELD REQ.	JUN SUPPLIED	JUN SOIL MOIST. CONTENT	JUN SPILL	JUL RAIN-FALL	JUL FIELD REQ.	JUL SUPPLIED	JUL SOIL MOIST. CONTENT	JUL SPILL
1	0.0	3.8	3.8	0.0	0.0	58.7	8.8	0.0	50.0	49.9	0.0	8.8	0.0	36.6	0.0
2	0.0	3.8	3.8	0.0	0.0	1.6	8.8	0.0	42.8	0.0	4.8	8.8	0.0	32.6	0.0
3	0.0	3.8	3.8	0.0	0.0	202.5	8.8	0.0	50.0	186.4	0.0	8.8	0.0	23.7	0.0
4	0.0	3.8	3.8	0.0	0.0	93.0	8.8	0.0	50.0	84.2	23.9	8.8	0.0	38.8	0.0
5	0.0	3.8	3.8	0.0	0.0	68.6	8.8	0.0	50.0	59.8	46.3	8.8	0.0	50.0	26.3
6	0.0	3.8	3.8	0.0	0.0	12.5	8.8	0.0	50.0	3.7	55.7	8.8	0.0	50.0	46.9
7	0.0	3.8	3.8	0.0	0.0	67.7	8.8	0.0	50.0	58.9	0.0	8.8	0.0	41.2	0.0
8	0.0	3.8	3.8	0.0	0.0	97.0	8.8	0.0	50.0	88.2	0.0	8.8	0.0	32.3	0.0
9	0.5	3.8	3.3	0.0	0.0	49.8	8.8	0.0	50.0	41.0	3.3	8.8	0.0	26.8	0.0
10	31.8	3.8	0.0	28.0	0.0	38.4	8.8	0.0	50.0	29.6	12.2	8.8	0.0	30.1	0.0
11	0.0	3.8	0.0	24.2	0.0	2.3	8.8	0.0	43.5	0.0	96.0	8.8	0.0	50.0	67.3
12	0.0	3.8	0.0	20.4	0.0	0.0	8.8	0.0	34.6	0.0	39.6	8.8	0.0	50.0	30.8
13	0.0	3.8	0.0	16.6	0.0	1.0	8.8	0.0	26.8	0.0	0.0	8.8	0.0	41.2	0.0
14	2.5	3.8	0.0	15.3	0.0	0.0	8.8	2.0	20.0	0.0	0.0	8.8	0.0	32.3	0.0
15	0.0	3.8	0.0	11.5	0.0	0.0	8.8	8.8	20.0	0.0	18.3	8.8	0.0	41.8	0.0
16	0.0	3.8	0.0	7.7	0.0	0.0	8.8	8.8	20.0	0.0	38.9	8.8	0.0	50.0	21.8
17	32.5	3.8	0.0	36.4	0.0	1.3	8.8	7.5	20.0	0.0	50.0	8.8	0.0	50.0	41.2
18	1.8	3.8	0.0	34.4	0.0	9.1	8.8	0.0	20.3	0.0	0.0	8.8	0.0	41.2	0.0
19	0.0	3.8	0.0	30.7	0.0	0.0	8.8	8.6	20.0	0.0	0.0	8.8	0.0	32.3	0.0
20	0.0	3.8	0.0	26.9	0.0	0.0	8.8	8.8	20.0	0.0	0.8	8.8	0.0	24.3	0.0
21	0.0	3.8	0.0	23.1	0.0	0.0	8.8	8.8	20.0	0.0	0.5	8.8	9.1	25.0	0.0
22	0.0	3.8	0.0	19.3	0.0	41.3	8.8	0.0	50.0	2.5	0.0	8.8	8.8	25.0	0.0
23	0.0	3.8	0.0	15.5	0.0	23.4	8.8	0.0	50.0	14.6	2.3	8.8	6.5	25.0	0.0
24	0.0	3.8	0.0	11.7	0.0	55.9	8.8	0.0	50.0	47.1	5.1	8.8	3.7	25.0	0.0
25	8.6	3.8	0.0	16.5	0.0	0.0	8.8	0.0	41.2	0.0	6.3	8.8	2.5	25.0	0.0
26	0.0	3.8	7.3	20.0	0.0	7.6	8.8	0.0	39.9	0.0	0.8	8.8	8.0	25.0	0.0
27	5.8	3.8	0.0	22.0	0.0	11.9	8.8	0.0	43.0	0.0	2.0	8.8	6.8	25.0	0.0
28	24.6	3.8	0.0	42.8	0.0	43.2	8.8	0.0	50.0	27.4	0.3	8.8	8.5	25.0	0.0
29	0.0	3.8	0.0	39.0	0.0	208.1	8.8	0.0	50.0	199.3	79.3	8.8	0.0	95.5	0.0
30	11.2	3.8	0.0	46.4	0.0	4.3	8.8	0.0	45.5	0.0	39.4	8.8	0.0	126.0	0.0
31	33.1	3.8	0.0	50.0	25.7						55.1	8.8	0.0	150.0	22.3
TOTAL	152.4	117.6	41.0		25.7	1099.2	264.8	53.4		892.3	580.9	274.1	54.2		256.4

PADDY OPERATION STUDY (2)

1967

DAY	AUG RAIN-FALL	AUG FIELD REQ.	AUG SUPPLIED	AUG SOIL MOIST. CONTENT	AUG SPILL	SEP RAIN-FALL	SEP FIELD REQ.	SEP SUPPLIED	SEP SOIL MOIST. CONTENT	SEP SPILL	OCT RAIN-FALL	OCT FIELD REQ.	OCT SUPPLIED	OCT SOIL MOIST. CONTENT	OCT SPILL
1	51.4	5.9	0.0	150.0	45.5	8.7	3.1	0.0	150.0	5.6	12.5	0.6	0.0	150.0	11.6
2	66.3	5.9	0.0	150.0	60.4	14.0	3.1	0.0	150.0	10.9	24.4	0.6	0.0	150.0	23.8
3	53.4	5.9	0.0	150.0	47.5	12.4	3.1	0.0	150.0	9.3	3.3	0.6	0.0	150.0	2.7
4	28.2	5.9	0.0	150.0	22.3	2.0	3.1	0.0	148.9	0.0	8.7	0.6	0.0	150.0	8.1
5	3.9	5.9	0.0	148.0	0.0	0.3	3.1	0.0	146.2	0.0	18.8	0.6	0.0	150.0	18.2
6	4.6	5.9	0.0	146.7	0.0	0.8	3.1	0.0	143.9	0.0	4.6	0.6	0.0	150.0	4.0
7	3.0	5.9	0.0	143.9	0.0	0.5	3.1	0.0	141.4	0.0	1.0	0.6	0.0	150.0	0.4
8	12.2	5.9	0.0	150.0	0.2	4.0	3.1	0.0	142.3	0.0	0.0	0.0	0.0	0.0	150.0
9	23.6	5.9	0.0	150.0	17.7	0.0	3.1	0.0	139.2	0.0	0.0	0.0	0.0	0.0	0.0
10	1.8	5.9	0.0	145.9	0.0	50.6	3.1	0.0	150.0	36.8	1.3	0.0	0.0	0.0	1.3
11	8.6	5.9	0.0	148.6	0.0	28.7	3.1	0.0	150.0	25.6	3.9	0.0	0.0	0.0	3.9
12	11.9	5.9	0.0	150.0	4.7	0.3	3.1	0.0	147.2	0.0	4.1	0.0	0.0	0.0	4.1
13	18.3	5.9	0.0	150.0	12.4	22.7	3.1	0.0	150.0	16.9	17.0	0.0	0.0	0.0	17.0
14	202.4	5.9	0.0	150.0	196.5	32.8	3.1	0.0	150.0	29.7	0.0	0.0	0.0	0.0	0.0
15	33.6	5.9	0.0	150.0	27.7	16.0	3.1	0.0	150.0	12.9	0.0	0.0	0.0	0.0	0.0
16	0.0	5.9	0.0	144.1	0.0	1.4	3.1	0.0	148.3	0.0	45.3	0.0	0.0	0.0	45.3
17	1.8	5.9	0.0	140.0	0.0	35.1	3.1	0.0	150.0	30.4	167.2	0.0	0.0	0.0	167.2
18	40.9	5.9	0.0	150.0	25.1	3.0	3.1	0.0	149.9	0.0	58.2	0.0	0.0	0.0	58.2
19	94.5	5.9	0.0	150.0	88.6	2.0	3.1	0.0	148.9	0.0	5.3	0.0	0.0	0.0	5.3
20	28.4	5.9	0.0	150.0	22.5	1.0	3.1	0.0	146.8	0.0	0.0	0.0	0.0	0.0	0.0
21	1.0	5.9	0.0	145.1	0.0	0.3	3.1	0.0	144.1	0.0	0.0	0.0	0.0	0.0	0.0
22	0.0	5.9	0.0	139.2	0.0	0.0	3.1	0.0	141.0	0.0	0.0	0.0	0.0	0.0	0.0
23	27.9	5.9	0.0	150.0	11.3	63.0	3.1	0.0	150.0	50.9	0.0	0.0	0.0	0.0	0.0
24	0.3	5.9	0.0	144.4	0.0	0.0	3.1	0.0	146.9	0.0	0.0	0.0	0.0	0.0	0.0
25	0.0	5.9	0.0	138.5	0.0	35.4	3.1	0.0	150.0	29.3	0.0	0.0	0.0	0.0	0.0
26	20.6	5.9	0.0	150.0	3.3	7.1	3.1	0.0	150.0	4.0	0.0	0.0	0.0	0.0	0.0
27	60.0	5.9	0.0	150.0	54.1	4.3	3.1	0.0	150.0	1.2	0.0	0.0	0.0	0.0	0.0
28	101.6	5.9	0.0	150.0	95.7	0.0	3.1	0.0	146.9	0.0	0.0	0.0	0.0	0.0	0.0
29	35.8	5.9	0.0	150.0	29.9	0.0	3.1	0.0	143.9	0.0	0.0	0.0	0.0	0.0	0.0
30	14.7	5.9	0.0	150.0	8.8	8.9	3.1	0.0	149.7	0.0	0.0	0.0	0.0	0.0	0.0
31	17.3	5.9	0.0	150.0	11.4						0.0	0.0	0.0	0.0	0.0
TOTAL	968.0	182.2	0.0		785.8	355.3	91.8	0.0		263.7	375.6	4.1	0.0		521.3

PADDY OPERATION STUDY (3)

1967

DAY	NOV RAIN-FALL	NOV FIELD REQ.	NOV SUPPLIED	NOV SOIL MOIST. CONTENT	NOV SPILL	DEC RAIN-FALL	DEC FIELD REQ.	DEC SUPPLIED	DEC SOIL MOIST. CONTENT	DEC SPILL	JAN RAIN-FALL	JAN FIELD REQ.	JAN SUPPLIED	JAN SOIL MOIST. CONTENT	JAN SPILL
1	0.0	3.2	3.2	0.0	0.0	0.0	8.5	0.0	28.8	0.0	0.0	9.2	9.2	20.0	0.0
2	0.0	3.2	3.2	0.0	0.0	0.0	8.5	0.0	20.4	0.0	0.0	9.2	9.2	20.0	0.0
3	0.0	3.2	3.2	0.0	0.0	1.5	8.5	6.6	20.0	0.0	0.0	9.2	9.2	20.0	0.0
4	125.7	3.2	0.0	122.5	0.0	0.0	8.5	8.5	20.0	0.0	0.0	9.2	9.2	20.0	0.0
5	35.1	3.2	0.0	150.0	4.5	0.3	8.5	8.2	20.0	0.0	0.0	9.2	9.2	20.0	0.0
6	0.3	3.2	0.0	147.1	0.0	0.0	8.5	8.5	20.0	0.0	0.0	9.2	9.2	20.0	0.0
7	0.0	3.2	0.0	144.0	0.0	0.0	8.5	8.5	20.0	0.0	0.0	9.2	9.2	20.0	0.0
8	0.3	3.2	0.0	141.1	0.0	0.0	8.5	8.5	20.0	0.0	1.8	9.2	7.4	20.0	0.0
9	0.0	3.2	0.0	137.9	0.0	0.0	8.5	8.5	20.0	0.0	0.0	9.2	9.2	20.0	0.0
10	0.0	3.2	0.0	134.7	0.0	0.0	8.5	8.5	20.0	0.0	0.0	9.2	9.2	20.0	0.0
11	0.0	3.2	0.0	131.6	0.0	0.0	8.5	8.5	20.0	0.0	0.0	9.2	9.2	20.0	0.0
12	0.0	3.2	0.0	128.4	0.0	0.0	8.5	8.5	20.0	0.0	0.0	9.2	9.2	20.0	0.0
13	0.0	3.2	0.0	125.2	0.0	0.0	8.5	8.5	20.0	0.0	0.0	9.2	9.2	20.0	0.0
14	0.0	3.2	0.0	122.0	0.0	0.0	8.5	8.5	20.0	0.0	0.0	9.2	9.2	20.0	0.0
15	0.0	3.2	0.0	118.9	0.0	0.0	8.5	8.5	20.0	0.0	0.0	9.2	9.2	20.0	0.0
16	0.8	3.2	0.0	116.5	0.0	0.0	8.5	8.5	20.0	0.0	0.0	9.2	9.2	20.0	0.0
17	0.3	3.2	0.0	113.6	0.0	0.0	8.5	8.5	20.0	0.0	0.0	9.2	9.2	20.0	0.0
18	0.3	3.2	0.0	110.7	0.0	0.0	8.5	8.5	20.0	0.0	0.0	9.2	9.2	20.0	0.0
19	0.0	3.2	0.0	107.6	0.0	0.0	8.5	8.5	20.0	0.0	0.0	9.2	9.2	20.0	0.0
20	0.0	3.2	0.0	104.4	0.0	0.0	8.5	8.5	20.0	0.0	0.0	9.2	9.2	20.0	0.0
21	0.0	3.2	0.0	101.2	0.0	0.0	8.5	8.5	20.0	0.0	0.0	9.2	14.2	25.0	0.0
22	0.0	3.2	0.0	98.0	0.0	0.0	8.5	8.5	20.0	0.0	0.0	9.2	9.2	25.0	0.0
23	0.0	3.2	0.0	94.9	0.0	0.0	8.5	8.5	20.0	0.0	0.0	9.2	9.2	25.0	0.0
24	0.0	3.2	0.0	91.7	0.0	0.0	8.5	8.5	20.0	0.0	0.0	9.2	9.2	25.0	0.0
25	0.0	3.2	0.0	88.5	0.0	0.0	8.5	8.5	20.0	0.0	0.0	9.2	9.2	25.0	0.0
26	0.0	3.2	0.0	50.0	35.3	0.0	8.5	8.5	20.0	0.0	0.0	9.2	9.2	25.0	0.0
27	0.0	3.2	0.0	46.8	0.0	0.0	8.5	8.5	20.0	0.0	0.0	9.2	9.2	25.0	0.0
28	0.0	3.2	0.0	43.7	0.0	0.0	8.5	8.5	20.0	0.0	0.0	9.2	9.2	25.0	0.0
29	0.0	3.2	0.0	40.5	0.0	0.0	8.5	8.5	20.0	0.0	0.0	9.2	9.2	25.0	0.0
30	0.0	3.2	0.0	37.3	0.0	0.0	8.5	8.5	20.0	0.0	1.5	9.2	7.7	25.0	0.0
31						0.0	8.5	8.5	20.0	0.0	0.0	9.2	9.2	25.0	0.0
TOTAL	162.8	95.2	9.5		39.8	1.8	262.4	243.3		0.0	3.3	284.0	285.7		0.0

PADDY OPERATION STUDY (4)

1968

DAY	FEB RAIN-FALL	FEB FIELD REQ.	FEB SUPPLIED	FEB SOIL MOIST. CONTENT	FEB SPILL	MAR RAIN-FALL	MAR FIELD REQ.	MAR SUPPLIED	MAR SOIL MOIST. CONTENT	MAR SPILL	APR RAIN-FALL	APR FIELD REQ.	APR SUPPLIED	APR SOIL MOIST. CONTENT	APR SPILL
1	0.0	9.5	9.5	25.0	0.0	0.0	4.7	4.7	25.0	0.0	0.0	1.0	1.0	25.0	0.0
2	0.0	9.5	9.5	25.0	0.0	0.0	4.7	4.7	25.0	0.0	0.0	1.0	1.0	25.0	0.0
3	0.0	9.5	9.5	25.0	0.0	0.0	4.7	4.7	25.0	0.0	0.0	1.0	1.0	25.0	0.0
4	0.0	9.5	9.5	25.0	0.0	0.8	4.7	3.9	25.0	0.0	0.0	1.0	1.0	25.0	0.0
5	0.0	9.5	9.5	25.0	0.0	0.0	4.7	4.7	25.0	0.0	0.0	1.0	1.0	25.0	0.0
6	0.0	9.5	9.5	25.0	0.0	0.0	4.7	4.7	25.0	0.0	0.0	1.0	1.0	25.0	0.0
7	0.0	9.5	9.5	25.0	0.0	0.0	4.7	4.7	25.0	0.0	0.0	1.0	1.0	25.0	0.0
8	0.0	9.5	9.5	25.0	0.0	0.0	4.7	4.7	25.0	0.0	0.0	1.0	1.0	25.0	0.0
9	0.0	9.5	9.5	25.0	0.0	0.0	4.7	4.7	25.0	0.0	10.7	1.0	0.0	34.7	0.0
10	1.8	9.5	7.7	25.0	0.0	0.0	4.7	4.7	25.0	0.0	0.5	0.0	0.0	0.0	35.2
11	0.0	9.5	9.5	25.0	0.0	0.0	4.7	4.7	25.0	0.0	0.0	0.0	0.0	0.0	0.0
12	0.0	9.5	9.5	25.0	0.0	0.0	4.7	4.7	25.0	0.0	0.0	0.0	0.0	0.0	0.0
13	0.0	9.5	9.5	25.0	0.0	0.0	4.7	4.7	25.0	0.0	2.8	0.0	0.0	0.0	2.8
14	0.0	9.5	9.5	25.0	0.0	0.0	4.7	4.7	25.0	0.0	0.0	0.0	0.0	0.0	0.0
15	0.0	9.5	9.5	25.0	0.0	0.0	4.7	4.7	25.0	0.0	0.0	0.0	0.0	0.0	0.0
16	0.0	9.5	9.5	25.0	0.0	0.0	4.7	4.7	25.0	0.0	0.0	0.0	0.0	0.0	0.0
17	0.0	9.5	9.5	25.0	0.0	0.0	4.7	4.7	25.0	0.0	0.0	0.0	0.0	0.0	0.0
18	0.0	9.5	9.5	25.0	0.0	0.0	4.7	4.7	25.0	0.0	0.0	0.0	0.0	0.0	0.0
19	0.0	9.5	9.5	25.0	0.0	0.0	4.7	4.7	25.0	0.0	0.0	0.0	0.0	0.0	0.0
20	0.0	9.5	9.5	25.0	0.0	0.0	4.7	4.7	25.0	0.0	15.3	0.0	0.0	0.0	15.3
21	0.0	9.5	9.5	25.0	0.0	0.0	4.7	4.7	25.0	0.0	0.8	0.0	0.0	0.0	0.8
22	0.0	9.5	9.5	25.0	0.0	0.0	4.7	4.7	25.0	0.0	0.0	0.0	0.0	0.0	0.0
23	0.0	9.5	9.5	25.0	0.0	0.0	4.7	4.7	25.0	0.0	1.0	0.0	0.0	0.0	1.0
24	0.0	9.5	9.5	25.0	0.0	0.0	4.7	4.7	25.0	0.0	0.0	0.0	0.0	0.0	0.0
25	0.0	9.5	9.5	25.0	0.0	0.0	4.7	4.7	25.0	0.0	0.0	0.0	0.0	0.0	0.0
26	0.0	9.5	9.5	25.0	0.0	0.0	4.7	4.7	25.0	0.0	8.1	0.0	0.0	0.0	8.1
27	0.0	9.5	9.5	25.0	0.0	0.0	4.7	4.7	25.0	0.0	0.0	0.0	0.0	0.0	0.0
28	0.0	9.5	9.5	25.0	0.0	0.0	4.7	4.7	25.0	0.0	0.0	0.0	0.0	0.0	0.0
29	0.0	9.5	9.5	25.0	0.0	0.0	4.7	4.7	25.0	0.0	0.0	0.0	0.0	0.0	0.0
30						0.0	4.7	4.7	25.0	0.0	0.5	0.0	0.0	0.0	0.5
31						0.0	4.7	4.7	25.0	0.0					
TOTAL	1.8	275.9	274.1		0.0	0.8	145.2	144.4		0.0	39.7	8.6	7.6		63.7

PADDY OPERATION STUDY (5)

1968

DAY	MAY RAIN-FALL	MAY FIELD REQ.	MAY SUPPLIED	MAY SOIL MOIST. CONTENT	MAY SPILL	JUN RAIN-FALL	JUN FIELD REQ.	JUN SUPPLIED	JUN SOIL MOIST. CONTENT	JUN SPILL	JUL RAIN-FALL	JUL FIELD REQ.	JUL SUPPLIED	JUL SOIL MOIST. CONTENT	JUL SPILL
1	0.0	3.7	3.7	0.0	0.0	0.0	9.7	0.0	39.4	0.0	3.8	9.0	5.2	20.0	0.0
2	0.0	3.7	3.7	0.0	0.0	23.9	9.7	0.0	50.0	3.6	22.9	9.0	0.0	33.9	0.0
3	0.0	3.7	3.7	0.0	0.0	0.5	9.7	0.0	40.8	0.0	3.1	9.0	0.0	28.1	0.0
4	0.0	3.7	3.7	0.0	0.0	0.5	9.7	0.0	31.6	0.0	0.0	9.0	0.9	20.0	0.0
5	0.0	3.7	3.7	0.0	0.0	44.8	9.7	0.0	50.0	16.7	2.5	9.0	6.5	20.0	0.0
6	2.0	3.7	1.7	0.0	0.0	0.0	9.7	0.0	40.3	0.0	2.5	9.0	6.5	20.0	0.0
7	15.8	3.7	0.0	12.1	0.0	0.0	9.7	0.0	30.6	0.0	0.0	9.0	9.0	20.0	0.0
8	0.0	3.7	0.0	8.3	0.0	3.2	9.7	0.0	24.1	0.0	3.8	9.0	5.2	20.0	0.0
9	6.5	3.7	0.0	11.1	0.0	0.0	9.7	5.6	20.0	0.0	5.1	9.0	3.9	20.0	0.0
10	7.4	3.7	0.0	14.8	0.0	0.0	9.7	9.7	20.0	0.0	0.0	9.0	9.0	20.0	0.0
11	2.8	3.7	0.0	13.9	0.0	0.0	9.7	9.7	20.0	0.0	0.0	9.0	9.0	20.0	0.0
12	8.1	3.7	0.0	18.2	0.0	38.6	9.7	0.0	48.9	0.0	6.6	9.0	2.4	20.0	0.0
13	0.0	3.7	0.0	14.5	0.0	16.3	9.7	0.0	50.0	5.5	26.2	9.0	0.0	37.2	0.0
14	2.0	3.7	0.0	12.8	0.0	14.0	9.7	0.0	50.0	4.3	1.0	9.0	0.0	29.3	0.0
15	0.0	3.7	0.0	9.1	0.0	14.5	9.7	0.0	50.0	4.8	22.6	9.0	0.0	42.9	0.0
16	29.5	3.7	0.0	34.8	0.0	22.4	9.7	0.0	50.0	12.7	20.6	9.0	0.0	50.0	4.6
17	43.7	3.7	0.0	74.8	0.0	0.0	9.7	0.0	40.3	0.0	8.4	9.0	0.0	49.4	0.0
18	0.5	3.7	0.0	71.6	0.0	0.0	9.7	0.0	30.6	0.0	1.5	9.0	0.0	42.0	0.0
19	4.3	3.7	0.0	72.2	0.0	19.3	9.7	0.0	40.2	0.0	31.3	9.0	0.0	50.0	14.3
20	0.0	3.7	0.0	68.4	0.0	0.0	9.7	0.0	30.5	0.0	22.4	9.0	0.0	50.0	13.4
21	2.3	3.7	0.0	67.0	0.0	0.0	9.7	0.0	20.8	0.0	67.8	9.0	0.0	108.8	0.0
22	13.7	3.7	0.0	77.0	0.0	7.6	9.7	1.3	20.0	0.0	88.7	9.0	0.0	150.0	38.6
23	0.0	3.7	0.0	73.3	0.0	0.5	9.7	9.2	20.0	0.0	27.2	9.0	0.0	150.0	18.2
24	7.9	3.7	0.0	77.4	0.0	11.4	9.7	0.0	21.7	0.0	54.8	9.0	0.0	150.0	45.8
25	0.0	3.7	0.0	73.7	0.0	1.1	9.7	6.9	20.0	0.0	42.1	9.0	0.0	150.0	33.1
26	0.0	3.7	0.0	50.0	20.0	0.0	9.7	9.7	20.0	0.0	25.4	9.0	0.0	150.0	16.4
27	1.8	3.7	0.0	48.1	0.0	10.9	9.7	0.0	21.2	0.0	15.2	9.0	0.0	150.0	6.2
28	0.0	3.7	0.0	44.3	0.0	0.0	9.7	8.5	20.0	0.0	4.1	9.0	0.0	145.1	0.0
29	2.0	3.7	0.0	42.6	0.0	7.4	9.7	2.3	20.0	0.0	9.4	9.0	0.0	145.6	0.0
30	26.9	3.7	0.0	50.0	15.8	0.3	9.7	9.4	20.0	0.0	4.0	9.0	0.0	140.6	0.0
31	2.8	3.7	0.0	49.1	0.0						0.0	9.0	0.0	131.7	0.0
TOTAL	180.0	115.5	20.4		35.8	237.2	291.2	72.4		47.5	523.0	277.5	57.1		190.9

PADDY OPERATION STUDY (6)

1968

DAY	------/ AUG/------ RAIN-FALL	FIELD REQ.	SUPPLIED	SOIL MOIST. CONTENT	SPILL	------/ SEP/------ RAIN-FALL	FIELD REQ.	SUPPLIED	SOIL MOIST. CONTENT	SPILL	------/ OCT/------ RAIN-FALL	FIELD REQ.	SUPPLIED	SOIL MOIST. CONTENT	SPILL
1	5.8	5.9	0.0	131.6	0.0	43.2	3.2	0.0	150.0	40.0	8.4	0.6	0.0	150.0	7.8
2	2.5	5.9	0.0	128.2	0.0	0.5	3.2	0.0	147.3	0.0	0.0	0.6	0.0	149.4	0.0
3	14.8	5.9	0.0	137.2	0.0	15.3	3.2	0.0	150.0	9.4	51.1	0.6	0.0	150.0	49.9
4	17.6	5.9	0.0	148.9	0.0	13.2	3.2	0.0	150.0	10.0	0.0	0.6	0.0	149.4	0.0
5	0.0	5.9	0.0	143.0	0.0	14.2	3.2	0.0	150.0	11.0	23.1	0.6	0.0	150.0	21.9
6	16.0	5.9	0.0	150.0	3.1	7.9	3.2	0.0	150.0	4.7	0.0	0.6	0.0	149.4	0.0
7	3.0	5.9	0.0	147.1	0.0	36.1	3.2	0.0	150.0	32.9	0.0	0.6	0.0	148.8	0.0
8	60.7	5.9	0.0	150.0	51.9	71.9	3.2	0.0	150.0	68.7	0.5	0.0	0.0	0.0	149.3
9	11.0	5.9	0.0	150.0	5.1	1.0	3.2	0.0	147.8	0.0	0.0	0.0	0.0	0.0	0.0
10	0.5	5.9	0.0	144.6	0.0	3.1	3.2	0.0	147.7	0.0	0.0	0.0	0.0	0.0	0.0
11	0.8	5.9	0.0	139.5	0.0	1.6	3.2	0.0	146.1	0.0	0.0	0.0	0.0	0.0	0.0
12	0.0	5.9	0.0	133.7	0.0	0.0	3.2	0.0	142.9	0.0	0.0	0.0	0.0	0.0	0.0
13	0.8	5.9	0.0	128.6	0.0	0.8	3.2	0.0	140.5	0.0	0.0	0.0	0.0	0.0	0.0
14	0.0	5.9	0.0	122.7	0.0	0.0	3.2	0.0	137.3	0.0	0.0	0.0	0.0	0.0	0.0
15	0.0	5.9	0.0	116.8	0.0	0.0	3.2	0.0	134.1	0.0	9.1	0.0	0.0	0.0	9.1
16	1.3	5.9	0.0	112.3	0.0	0.0	3.2	0.0	130.9	0.0	0.0	0.0	0.0	0.0	0.0
17	0.0	5.9	0.0	106.4	0.0	0.0	3.2	0.0	127.7	0.0	0.0	0.0	0.0	0.0	0.0
18	41.7	5.9	0.0	142.2	0.0	0.0	3.2	0.0	124.5	0.0	0.0	0.0	0.0	0.0	0.0
19	98.6	5.9	0.0	150.0	84.9	0.0	3.2	0.0	121.3	0.0	34.8	0.0	0.0	0.0	34.8
20	74.6	5.9	0.0	150.0	68.7	0.0	3.2	0.0	118.1	0.0	0.0	0.0	0.0	0.0	0.0
21	17.8	5.9	0.0	150.0	11.9	47.3	3.2	0.0	150.0	12.2	0.0	0.0	0.0	0.0	0.0
22	15.5	5.9	0.0	150.0	9.6	1.5	3.2	0.0	148.3	0.0	0.0	0.0	0.0	0.0	0.0
23	21.1	5.9	0.0	150.0	15.2	22.6	3.2	0.0	150.0	17.7	0.0	0.0	0.0	0.0	0.0
24	20.3	5.9	0.0	150.0	14.4	9.7	3.2	0.0	150.0	6.5	0.0	0.0	0.0	0.0	0.0
25	326.9	5.9	0.0	150.0	321.0	0.0	3.2	0.0	146.8	0.0	0.0	0.0	0.0	0.0	0.0
26	65.0	5.9	0.0	150.0	59.1	0.0	3.2	0.0	143.6	0.0	1.3	0.0	0.0	0.0	1.3
27	33.8	5.9	0.0	150.0	27.9	3.3	3.2	0.0	143.7	0.0	0.0	0.0	0.0	0.0	0.0
28	86.1	5.9	0.0	150.0	80.2	91.0	3.2	0.0	150.0	81.5	0.0	0.0	0.0	0.0	0.0
29	28.8	5.9	0.0	150.0	22.9	130.6	3.2	0.0	150.0	127.4	0.0	0.0	0.0	0.0	0.0
30	13.9	5.9	0.0	150.0	8.0	5.4	3.2	0.0	150.0	2.2	0.0	0.0	0.0	0.0	0.0
31	34.3	5.9	0.0	150.0	28.4						0.0	0.0	0.0	0.0	0.0
TOTAL	1013.2	182.2	0.0		812.7	520.2	96.0	0.0		424.2	128.3	4.2	0.0		274.1

PADDY OPERATION STUDY (7)

1968

DAY	NOV RAIN-FALL	NOV FIELD REQ.	NOV SUPPLIED	NOV SOIL MOIST. CONTENT	NOV SPILL	DEC RAIN-FALL	DEC FIELD REQ.	DEC SUPPLIED	DEC SOIL MOIST. CONTENT	DEC SPILL	JAN RAIN-FALL	JAN FIELD REQ.	JAN SUPPLIED	JAN SOIL MOIST. CONTENT	JAN SPILL
1	0.0	3.1	3.1	0.0	0.0	0.0	8.7	0.0	30.0	0.0	0.0	9.3	9.3	20.0	0.0
2	0.0	3.1	3.1	0.0	0.0	0.0	8.7	0.0	21.4	0.0	0.0	9.3	9.3	20.0	0.0
3	0.0	3.1	3.1	0.0	0.0	0.0	8.7	7.3	20.0	0.0	0.0	9.3	9.3	20.0	0.0
4	0.0	3.1	3.1	0.0	0.0	0.0	8.7	8.7	20.0	0.0	0.0	9.3	9.3	20.0	0.0
5	0.0	3.1	3.1	0.0	0.0	0.0	8.7	8.7	20.0	0.0	0.0	9.3	9.3	20.0	0.0
6	0.0	3.1	3.1	0.0	0.0	0.0	8.7	8.7	20.0	0.0	0.0	9.3	9.3	20.0	0.0
7	0.0	3.1	3.1	0.0	0.0	0.0	8.7	8.7	20.0	0.0	0.0	9.3	9.3	20.0	0.0
8	0.0	3.1	3.1	0.0	0.0	0.0	8.7	8.7	20.0	0.0	0.0	9.3	9.3	20.0	0.0
9	0.0	3.1	3.1	0.0	0.0	0.0	8.7	8.7	20.0	0.0	0.0	9.3	9.3	20.0	0.0
10	0.0	3.1	3.1	0.0	0.0	0.0	8.7	8.7	20.0	0.0	0.0	9.3	9.3	20.0	0.0
11	0.0	3.1	3.1	0.0	0.0	0.0	8.7	8.7	20.0	0.0	0.0	9.3	9.3	20.0	0.0
12	0.0	3.1	3.1	0.0	0.0	1.8	8.7	6.9	20.0	0.0	0.0	9.3	9.3	20.0	0.0
13	0.0	3.1	3.1	0.0	0.0	0.0	8.7	8.7	20.0	0.0	2.0	9.3	7.3	20.0	0.0
14	0.0	3.1	3.1	0.0	0.0	0.0	8.7	8.7	20.0	0.0	8.9	9.3	0.4	20.0	0.0
15	0.0	3.1	3.1	0.0	0.0	0.0	8.7	8.7	20.0	0.0	0.0	9.3	9.3	20.0	0.0
16	0.0	3.1	3.1	0.0	0.0	0.0	8.7	8.7	20.0	0.0	0.0	9.3	9.3	20.0	0.0
17	0.0	3.1	3.1	0.0	0.0	0.0	8.7	8.7	20.0	0.0	0.0	9.3	9.3	20.0	0.0
18	0.0	3.1	3.1	0.0	0.0	0.0	8.7	8.7	20.0	0.0	0.0	9.3	9.3	20.0	0.0
19	0.0	3.1	3.1	0.0	0.0	0.0	8.7	8.7	20.0	0.0	0.0	9.3	9.3	20.0	0.0
20	0.0	3.1	3.1	0.0	0.0	0.0	8.7	8.7	20.0	0.0	0.0	9.3	9.3	20.0	0.0
21	0.0	3.1	3.1	0.0	0.0	0.0	8.7	8.7	20.0	0.0	0.0	9.3	14.3	25.0	0.0
22	0.0	3.1	3.1	0.0	0.0	0.0	8.7	8.7	20.0	0.0	0.0	9.3	9.3	25.0	0.0
23	0.0	3.1	3.1	0.0	0.0	0.0	8.7	8.7	20.0	0.0	0.0	9.3	9.3	25.0	0.0
24	0.0	3.1	3.1	0.0	0.0	0.0	8.7	8.7	20.0	0.0	0.0	9.3	9.3	25.0	0.0
25	0.0	3.1	3.1	0.0	0.0	0.0	8.7	8.7	20.0	0.0	0.0	9.3	9.3	25.0	0.0
26	0.0	3.1	23.1	20.0	0.0	0.0	8.7	8.7	20.0	0.0	0.0	9.3	9.3	25.0	0.0
27	0.0	3.1	3.1	20.0	0.0	0.0	8.7	8.7	20.0	0.0	0.0	9.3	9.3	25.0	0.0
28	0.0	3.1	3.1	20.0	0.0	0.0	8.7	8.7	20.0	0.0	4.6	9.3	4.7	25.0	0.0
29	24.9	3.1	0.0	41.8	0.0	0.0	8.7	8.7	20.0	0.0	0.0	9.3	9.3	25.0	0.0
30	0.0	3.1	0.0	38.7	0.0	0.0	8.7	8.7	20.0	0.0	0.0	9.3	9.3	25.0	0.0
31						0.0	8.7	8.7	20.0	0.0	0.0	9.3	9.3	25.0	0.0
TOTAL	24.9	93.2	107.0		0.0	1.8	268.4	247.9		0.0	15.5	287.4	276.9		0.0

PADDY OPERATION STUDY (8)

1969

DAY	FEB/ RAIN-FALL	FEB/ FIELD REQ.	FEB/ SUPPLIED	FEB/ SOIL MOIST. CONTENT	FEB/ SPILL	MAR/ RAIN-FALL	MAR/ FIELD REQ.	MAR/ SUPPLIED	MAR/ SOIL MOIST. CONTENT	MAR/ SPILL	APR/ RAIN-FALL	APR/ FIELD REQ.	APR/ SUPPLIED	APR/ SOIL MOIST. CONTENT	APR/ SPILL
1	0.0	9.2	9.2	25.0	0.0	0.0	4.9	4.9	25.0	0.0	0.0	1.0	1.0	25.0	0.0
2	0.0	9.2	9.2	25.0	0.0	0.0	4.9	4.9	25.0	0.0	22.3	1.0	0.0	46.3	0.0
3	0.0	9.2	9.2	25.0	0.0	0.0	4.9	4.9	25.0	0.0	0.0	1.0	0.0	45.4	0.0
4	19.8	9.2	0.0	35.6	0.0	0.0	4.9	4.9	25.0	0.0	11.4	1.0	0.0	55.8	0.0
5	0.0	9.2	0.0	26.4	0.0	0.0	4.9	4.9	25.0	0.0	0.0	1.0	0.0	54.9	0.0
6	0.0	9.2	7.8	25.0	0.0	0.0	4.9	4.9	25.0	0.0	0.0	1.0	0.0	53.9	0.0
7	0.0	9.2	9.2	25.0	0.0	0.0	4.9	4.9	25.0	0.0	0.0	1.0	0.0	53.0	0.0
8	0.0	9.2	9.2	25.0	0.0	0.0	4.9	4.9	25.0	0.0	0.0	1.0	0.0	52.0	0.0
9	0.0	9.2	9.2	25.0	0.0	0.0	4.9	4.9	25.0	0.0	0.0	1.0	0.0	51.1	0.0
10	0.0	9.2	9.2	25.0	0.0	0.0	4.9	4.9	25.0	0.0	0.0	0.0	0.0	0.0	51.1
11	0.0	9.2	9.2	25.0	0.0	0.0	4.9	4.9	25.0	0.0	10.4	0.0	0.0	0.0	10.4
12	0.0	9.2	9.2	25.0	0.0	0.0	4.9	4.9	25.0	0.0	0.0	0.0	0.0	0.0	0.0
13	0.0	9.2	9.2	25.0	0.0	11.9	4.9	0.0	32.0	0.0	0.0	0.0	0.0	0.0	0.0
14	0.0	9.2	9.2	25.0	0.0	0.0	4.9	0.0	27.1	0.0	0.0	0.0	0.0	0.0	0.0
15	0.0	9.2	9.2	25.0	0.0	0.0	4.9	2.8	25.0	0.0	0.0	0.0	0.0	0.0	0.0
16	0.0	9.2	9.2	25.0	0.0	0.0	4.9	4.9	25.0	0.0	0.0	0.0	0.0	0.0	0.0
17	0.0	9.2	9.2	25.0	0.0	1.3	4.9	3.6	25.0	0.0	0.0	0.0	0.0	0.0	0.0
18	0.0	9.2	9.2	25.0	0.0	0.0	4.9	4.9	25.0	0.0	0.0	0.0	0.0	0.0	0.0
19	0.0	9.2	9.2	25.0	0.0	0.0	4.9	4.9	25.0	0.0	0.0	0.0	0.0	0.0	0.0
20	0.0	9.2	9.2	25.0	0.0	0.0	4.9	4.9	25.0	0.0	0.0	0.0	0.0	0.0	0.0
21	0.0	9.2	9.2	25.0	0.0	0.0	4.9	4.9	25.0	0.0	0.0	0.0	0.0	0.0	0.0
22	0.0	9.2	9.2	25.0	0.0	0.0	4.9	4.9	25.0	0.0	0.0	0.0	0.0	0.0	0.0
23	0.0	9.2	9.2	25.0	0.0	0.0	4.9	4.9	25.0	0.0	0.0	0.0	0.0	0.0	0.0
24	0.0	9.2	9.2	25.0	0.0	0.0	4.9	4.9	25.0	0.0	0.0	0.0	0.0	0.0	0.0
25	0.0	9.2	9.2	25.0	0.0	0.0	4.9	4.9	25.0	0.0	0.0	0.0	0.0	0.0	0.0
26	0.0	9.2	9.2	25.0	0.0	0.0	4.9	4.9	25.0	0.0	0.0	0.0	0.0	0.0	0.0
27	0.0	9.2	9.2	25.0	0.0	0.0	4.9	4.9	25.0	0.0	0.0	0.0	0.0	0.0	0.0
28	0.0	9.2	9.2	25.0	0.0	0.0	4.9	4.9	25.0	0.0	0.0	0.0	0.0	0.0	0.0
29						0.0	4.9	4.9	25.0	0.0	0.5	0.0	0.0	0.0	0.5
30						0.0	4.9	4.9	25.0	0.0	8.1	0.0	0.0	0.0	8.1
31						0.0	4.9	4.9	25.0	0.0					
TOTAL	19.8	258.0	238.2		0.0	13.2	151.9	138.7		0.0	52.7	8.6	1.0		70.1

PADDY OPERATION STUDY (9)

1969

DAY	MAY RAIN-FALL	MAY FIELD REQ.	MAY SUPPLIED	MAY SOIL MOIST. CONTENT	MAY SPILL	JUN RAIN-FALL	JUN FIELD REQ.	JUN SUPPLIED	JUN SOIL MOIST. CONTENT	JUN SPILL	JUL RAIN-FALL	JUL FIELD REQ.	JUL SUPPLIED	JUL SOIL MOIST. CONTENT	JUL SPILL
1	0.0	3.9	3.9	0.0	0.0	27.7	9.3	0.0	50.0	10.7	4.1	9.0	4.9	20.0	0.0
2	0.0	3.9	3.9	0.0	0.0	0.0	9.3	0.0	40.7	0.0	9.4	9.0	0.0	20.4	0.0
3	0.0	3.9	3.9	0.0	0.0	11.7	9.3	0.0	43.1	0.0	15.7	9.0	0.0	27.2	0.0
4	0.0	3.9	3.9	0.0	0.0	9.3	9.3	0.0	43.1	0.0	50.0	9.0	0.0	50.0	18.2
5	3.8	3.9	0.1	0.0	0.0	0.2	9.3	0.0	33.9	0.0	80.6	9.0	0.0	50.0	71.6
6	1.0	3.9	2.9	0.0	0.0	1.3	9.3	0.0	25.9	0.0	1.5	9.0	0.0	42.5	0.0
7	0.0	3.9	3.9	0.0	0.0	0.8	9.3	2.6	20.0	0.0	0.0	9.0	0.0	33.6	0.0
8	0.0	3.9	3.9	0.0	0.0	1.5	9.3	7.8	20.0	0.0	0.8	9.0	0.0	25.4	0.0
9	0.8	3.9	3.1	0.0	0.0	4.3	9.3	5.0	20.0	0.0	0.0	9.0	3.5	20.0	0.0
10	0.0	3.9	3.9	0.0	0.0	25.4	9.3	0.0	36.1	0.0	0.0	9.0	9.0	20.0	0.0
11	0.0	3.9	3.9	0.0	0.0	65.3	9.3	0.0	50.0	42.1	1.8	9.0	7.2	20.0	0.0
12	0.0	3.9	3.9	0.0	0.0	46.7	9.3	0.0	50.0	37.4	7.3	9.0	1.7	20.0	0.0
13	15.7	3.9	0.0	11.8	0.0	59.9	9.3	0.0	50.0	50.6	25.7	9.0	0.0	36.7	0.0
14	38.9	3.9	0.0	46.9	0.0	10.1	9.3	0.0	50.0	0.8	25.1	9.0	0.0	50.0	2.9
15	2.8	3.9	0.0	45.8	0.0	22.6	9.3	0.0	50.0	13.3	6.1	9.0	0.0	47.1	0.0
16	0.0	3.9	0.0	41.9	0.0	45.7	9.3	0.0	50.0	36.4	28.1	9.0	0.0	50.0	16.3
17	7.4	3.9	0.0	45.5	0.0	0.0	9.3	0.0	40.7	0.0	10.7	9.0	0.0	50.0	1.7
18	15.8	3.9	0.0	57.4	0.0	0.5	9.3	0.0	31.9	0.0	1.5	9.0	0.0	42.5	0.0
19	0.0	3.9	0.0	53.6	0.0	0.0	9.3	0.0	22.6	0.0	1.3	9.0	0.0	34.9	0.0
20	23.6	3.9	0.0	73.3	0.0	27.9	9.3	0.0	41.1	0.0	12.2	9.0	0.0	38.1	0.0
21	29.2	3.9	0.0	98.6	0.0	0.0	9.3	0.0	31.8	0.0	41.4	9.0	0.0	70.6	0.0
22	0.5	3.9	0.0	95.3	0.0	13.0	9.3	0.0	35.5	0.0	20.8	9.0	0.0	82.4	0.0
23	76.2	3.9	0.0	150.0	17.6	0.0	9.3	0.0	26.2	0.0	4.1	9.0	0.0	77.6	0.0
24	35.8	3.9	0.0	150.0	31.9	42.9	9.3	0.0	50.0	9.8	10.6	9.0	0.0	79.2	0.0
25	27.6	3.9	0.0	150.0	23.7	0.0	9.3	0.0	40.7	0.0	1.0	9.0	0.0	71.3	0.0
26	6.6	3.9	0.0	50.0	102.7	0.0	9.3	0.0	31.4	0.0	149.3	9.0	0.0	150.0	61.6
27	16.5	3.9	0.0	50.0	12.6	10.5	9.3	0.0	32.6	0.0	128.1	9.0	0.0	150.0	119.1
28	1.3	3.9	0.0	47.4	0.0	0.0	9.3	0.0	23.2	0.0	36.4	9.0	0.0	150.0	27.4
29	23.9	3.9	0.0	50.0	17.5	0.0	9.3	6.1	20.0	0.0	47.7	9.0	0.0	150.0	38.7
30	0.0	3.9	0.0	46.1	0.0	5.1	9.3	4.2	20.0	0.0	233.0	9.0	0.0	150.0	224.0
31	0.0	3.9	0.0	42.3	0.0						90.4	9.0	0.0	150.0	81.4
TOTAL	327.4	119.7	40.8		206.1	432.4	279.5	25.7		200.9	1044.7	277.5	26.1		663.3

PADDY OPERATION STUDY (10)

1969

DAY	AUG RAIN-FALL	AUG FIELD REQ.	AUG SUPPLIED	AUG SOIL MOIST. CONTENT	AUG SPILL	SEP RAIN-FALL	SEP FIELD REQ.	SEP SUPPLIED	SEP SOIL MOIST. CONTENT	SEP SPILL	OCT RAIN-FALL	OCT FIELD REQ.	OCT SUPPLIED	OCT SOIL MOIST. CONTENT	OCT SPILL
1	52.6	7.3	0.0	150.0	45.3	58.2	3.0	0.0	150.0	20.7	11.6	0.6	0.0	150.0	6.6
2	95.3	7.3	0.0	150.0	88.0	1.0	3.0	0.0	148.0	0.0	7.1	0.6	0.0	150.0	6.5
3	62.2	7.3	0.0	150.0	54.9	0.0	3.0	0.0	145.1	0.0	3.6	0.6	0.0	150.0	3.0
4	66.5	7.3	0.0	150.0	59.2	0.0	3.0	0.0	142.1	0.0	20.0	0.6	0.0	150.0	19.4
5	240.6	7.3	0.0	150.0	233.3	9.4	3.0	0.0	148.5	0.0	2.0	0.6	0.0	150.0	1.4
6	27.1	7.3	0.0	150.0	19.8	67.3	3.0	0.0	150.0	62.9	21.6	0.6	0.0	150.0	21.0
7	21.9	7.3	0.0	150.0	14.6	27.9	3.0	0.0	150.0	24.9	25.1	0.6	0.0	150.0	24.5
8	126.2	7.3	0.0	150.0	118.9	123.4	3.0	0.0	150.0	120.4	11.4	0.0	0.0	0.0	161.4
9	0.8	7.3	0.0	143.5	0.0	40.4	3.0	0.0	150.0	37.4	0.0	0.0	0.0	0.0	0.0
10	0.0	7.3	0.0	136.2	0.0	24.4	3.0	0.0	150.0	21.4	0.0	0.0	0.0	0.0	0.0
11	7.1	7.3	0.0	135.9	0.0	89.7	3.0	0.0	150.0	86.7	0.0	0.0	0.0	0.0	0.0
12	0.0	7.3	0.0	128.6	0.0	28.7	3.0	0.0	150.0	25.7	0.0	0.0	0.0	0.0	0.0
13	5.3	7.3	0.0	126.6	0.0	26.6	3.0	0.0	150.0	23.6	0.0	0.0	0.0	0.0	0.0
14	0.0	7.3	0.0	119.3	0.0	2.3	3.0	0.0	149.3	0.0	0.0	0.0	0.0	0.0	0.0
15	21.9	7.3	0.0	133.9	0.0	0.0	3.0	0.0	146.4	0.0	12.4	0.0	0.0	0.0	12.4
16	0.0	7.3	0.0	126.6	0.0	0.0	3.0	0.0	143.4	0.0	0.0	0.0	0.0	0.0	0.0
17	19.0	7.3	0.0	138.2	0.0	1.8	3.0	0.0	142.2	0.0	0.0	0.0	0.0	0.0	0.0
18	8.1	7.3	0.0	139.0	0.0	15.7	3.0	0.0	150.0	5.0	0.0	0.0	0.0	0.0	0.0
19	4.1	7.3	0.0	135.8	0.0	22.4	3.0	0.0	150.0	19.4	0.0	0.0	0.0	0.0	0.0
20	0.5	7.3	0.0	129.0	0.0	2.8	3.0	0.0	149.8	0.0	0.0	0.0	0.0	0.0	0.0
21	8.1	7.3	0.0	129.8	0.0	29.0	3.0	0.0	150.0	25.9	5.1	0.0	0.0	0.0	5.1
22	0.0	7.3	0.0	122.4	0.0	21.1	3.0	0.0	150.0	18.1	58.4	0.0	0.0	0.0	58.4
23	0.0	7.3	0.0	115.1	0.0	10.4	3.0	0.0	150.0	7.4	0.0	0.0	0.0	0.0	0.0
24	8.1	7.3	0.0	115.9	0.0	27.2	3.0	0.0	150.0	24.2	0.0	0.0	0.0	0.0	0.0
25	3.8	7.3	0.0	112.4	0.0	7.6	3.0	0.0	150.0	4.6	6.9	0.0	0.0	0.0	6.9
26	29.2	7.3	0.0	134.3	0.0	0.0	3.0	0.0	147.0	0.0	0.0	0.0	0.0	0.0	0.0
27	0.0	7.3	0.0	126.9	0.0	9.4	3.0	0.0	150.0	3.5	5.1	0.0	0.0	0.0	5.1
28	0.0	7.3	0.0	119.6	0.0	7.9	3.0	0.0	150.0	4.9	0.0	0.0	0.0	0.0	0.0
29	0.0	7.3	0.0	112.3	0.0	1.5	3.0	0.0	148.5	0.0	8.4	0.0	0.0	0.0	8.4
30	17.8	7.3	0.0	122.8	0.0	0.0	3.0	0.0	145.6	0.0	22.9	0.0	0.0	0.0	22.9
31	0.0	7.3	0.0	115.5	0.0						0.0	0.0	0.0	0.0	0.0
TOTAL	826.2	226.9	0.0		633.9	656.1	89.0	0.0		537.0	221.6	4.2	0.0		363.0

PADDY OPERATION STUDY (11)

1969

DAY	NOV RAIN-FALL	NOV FIELD REQ.	NOV SUPPLIED	NOV SOIL MOIST. CONTENT	NOV SPILL	DEC RAIN-FALL	DEC FIELD REQ.	DEC SUPPLIED	DEC SOIL MOIST. CONTENT	DEC SPILL	JAN RAIN-FALL	JAN FIELD REQ.	JAN SUPPLIED	JAN SOIL MOIST. CONTENT	JAN SPILL
1	0.0	3.0	3.0	0.0	0.0	0.0	7.8	0.0	28.0	0.0	0.0	8.9	8.9	20.0	0.0
2	0.0	3.0	3.0	0.0	0.0	0.5	7.8	0.0	20.7	0.0	0.0	8.9	8.9	20.0	0.0
3	0.0	3.0	3.0	0.0	0.0	0.0	7.8	7.1	20.0	0.0	9.6	8.9	0.0	20.7	0.0
4	0.0	3.0	3.0	0.0	0.0	0.0	7.8	7.8	20.0	0.0	0.0	8.9	8.3	20.0	0.0
5	0.0	3.0	3.0	0.0	0.0	0.0	7.8	7.8	20.0	0.0	0.0	8.9	8.9	20.0	0.0
6	0.0	3.0	3.0	0.0	0.0	0.0	7.8	7.8	20.0	0.0	0.0	8.9	8.9	20.0	0.0
7	0.0	3.0	3.0	0.0	0.0	0.0	7.8	7.8	20.0	0.0	0.0	8.9	8.9	20.0	0.0
8	0.0	3.0	3.0	0.0	0.0	0.0	7.8	7.8	20.0	0.0	25.6	8.9	0.0	36.7	0.0
9	0.0	3.0	3.0	0.0	0.0	0.0	7.8	7.8	20.0	0.0	1.5	8.9	0.0	29.2	0.0
10	0.0	3.0	3.0	0.0	0.0	0.0	7.8	7.8	20.0	0.0	0.0	8.9	0.0	20.3	0.0
11	0.0	3.0	3.0	0.0	0.0	0.0	7.8	7.8	20.0	0.0	0.0	8.9	8.7	20.0	0.0
12	0.0	3.0	3.0	0.0	0.0	0.0	7.8	7.8	20.0	0.0	0.0	8.9	8.9	20.0	0.0
13	0.0	3.0	3.0	0.0	0.0	0.0	7.8	7.8	20.0	0.0	4.3	8.9	4.6	20.0	0.0
14	0.0	3.0	3.0	0.0	0.0	0.0	7.8	7.8	20.0	0.0	0.8	8.9	8.1	20.0	0.0
15	0.0	3.0	3.0	0.0	0.0	0.0	7.8	7.8	20.0	0.0	0.0	8.9	8.9	20.0	0.0
16	0.0	3.0	3.0	0.0	0.0	0.0	7.8	7.8	20.0	0.0	0.0	8.9	8.9	20.0	0.0
17	0.0	3.0	3.0	0.0	0.0	0.0	7.8	7.8	20.0	0.0	0.0	8.9	8.9	20.0	0.0
18	0.0	3.0	3.0	0.0	0.0	0.0	7.8	7.8	20.0	0.0	0.0	8.9	8.9	20.0	0.0
19	0.0	3.0	3.0	0.0	0.0	0.0	7.8	7.8	20.0	0.0	0.0	8.9	8.9	20.0	0.0
20	0.0	3.0	3.0	0.0	0.0	0.0	7.8	7.8	20.0	0.0	0.5	8.9	8.4	20.0	0.0
21	0.5	3.0	2.5	0.0	0.0	0.0	7.8	7.8	20.0	0.0	0.0	8.9	13.9	25.0	0.0
22	0.0	3.0	3.0	0.0	0.0	0.0	7.8	7.8	20.0	0.0	0.0	8.9	8.9	25.0	0.0
23	20.3	3.0	0.0	17.3	0.0	0.0	7.8	7.8	20.0	0.0	0.0	8.9	8.9	25.0	0.0
24	34.5	3.0	0.0	48.7	0.0	0.0	7.8	7.8	20.0	0.0	0.0	8.9	8.9	25.0	0.0
25	5.3	3.0	0.0	51.0	0.0	0.0	7.8	7.8	20.0	0.0	0.0	8.9	8.9	25.0	0.0
26	0.0	3.0	0.0	47.9	0.0	0.0	7.8	7.8	20.0	0.0	0.0	8.9	8.9	25.0	0.0
27	0.0	3.0	0.0	44.9	0.0	0.0	7.8	7.8	20.0	0.0	0.0	8.9	8.9	25.0	0.0
28	0.0	3.0	0.0	41.8	0.0	0.0	7.8	7.8	20.0	0.0	0.0	8.9	8.9	25.0	0.0
29	0.0	3.0	0.0	38.8	0.0	0.0	7.8	7.8	20.0	0.0	0.0	8.9	8.9	25.0	0.0
30	0.0	3.0	0.0	35.8	0.0	0.0	7.8	7.8	20.0	0.0	0.0	8.9	8.9	25.0	0.0
31						0.0	7.8	7.8	20.0	0.0	0.0	8.9	8.9	25.0	0.0
TOTAL	60.6	91.3	66.4		0.0	0.5	241.5	225.2		0.0	42.3	277.2	239.9		0.0

PADDY OPERATION STUDY (12)

1970

DAY	FEB RAIN-FALL	FEB FIELD REQ.	FEB SUPPLIED	FEB SOIL MOIST. CONTENT	FEB SPILL	MAR RAIN-FALL	MAR FIELD REQ.	MAR SUPPLIED	MAR SOIL MOIST. CONTENT	MAR SPILL	APR RAIN-FALL	APR FIELD REQ.	APR SUPPLIED	APR SOIL MOIST. CONTENT	APR SPILL
1	0.0	9.2	9.2	25.0	0.0	0.0	4.6	4.6	25.0	0.0	0.0	1.0	0.0	48.6	0.0
2	0.0	9.2	9.2	25.0	0.0	0.0	4.6	4.6	25.0	0.0	0.0	1.0	0.0	47.6	0.0
3	0.0	9.2	9.2	25.0	0.0	0.0	4.6	4.6	25.0	0.0	0.0	1.0	0.0	46.7	0.0
4	0.0	9.2	9.2	25.0	0.0	0.0	4.6	4.6	25.0	0.0	0.0	1.0	0.0	45.7	0.0
5	0.0	9.2	9.2	25.0	0.0	0.0	4.6	4.6	25.0	0.0	0.0	1.0	0.0	44.7	0.0
6	0.0	9.2	9.2	25.0	0.0	0.0	4.6	4.6	25.0	0.0	2.3	1.0	0.0	46.1	0.0
7	0.0	9.2	9.2	25.0	0.0	0.0	4.6	4.6	25.0	0.0	0.0	1.0	0.0	45.1	0.0
8	0.0	9.2	9.2	25.0	0.0	0.0	4.6	4.6	25.0	0.0	0.0	1.0	0.0	44.1	0.0
9	0.0	9.2	9.2	25.0	0.0	0.0	4.6	4.6	25.0	0.0	0.0	1.0	0.0	43.2	0.0
10	0.0	9.2	9.2	25.0	0.0	0.0	4.6	4.6	25.0	0.0	0.0	0.0	0.0	0.0	43.2
11	0.0	9.2	9.2	25.0	0.0	0.0	4.6	4.6	25.0	0.0	0.0	0.0	0.0	0.0	0.0
12	0.0	9.2	9.2	25.0	0.0	0.0	4.6	4.6	25.0	0.0	0.0	0.0	0.0	0.0	0.0
13	0.0	9.2	9.2	25.0	0.0	0.0	4.6	4.6	25.0	0.0	0.0	0.0	0.0	0.0	0.0
14	0.0	9.2	9.2	25.0	0.0	0.0	4.6	4.6	25.0	0.0	18.3	0.0	0.0	0.0	18.3
15	0.0	9.2	9.2	25.0	0.0	0.0	4.6	4.6	25.0	0.0	9.9	0.0	0.0	0.0	9.9
16	0.0	9.2	9.2	25.0	0.0	0.0	4.6	4.6	25.0	0.0	0.0	0.0	0.0	0.0	0.0
17	0.0	9.2	9.2	25.0	0.0	0.0	4.6	4.6	25.0	0.0	0.0	0.0	0.0	0.0	0.0
18	0.0	9.2	9.2	25.0	0.0	0.0	4.6	4.6	25.0	0.0	0.0	0.0	0.0	0.0	0.0
19	0.0	9.2	9.2	25.0	0.0	0.0	4.6	4.6	25.0	0.0	1.0	0.0	0.0	0.0	1.0
20	0.0	9.2	9.2	25.0	0.0	0.0	4.6	4.6	25.0	0.0	0.0	0.0	0.0	0.0	0.0
21	0.0	9.2	9.2	25.0	0.0	0.0	4.6	4.6	25.0	0.0	0.0	0.0	0.0	0.0	0.0
22	0.0	9.2	9.2	25.0	0.0	0.0	4.6	4.6	25.0	0.0	31.0	0.0	0.0	0.0	31.0
23	0.0	9.2	9.2	25.0	0.0	0.0	4.6	4.6	25.0	0.0	0.0	0.0	0.0	0.0	0.0
24	0.0	9.2	9.2	25.0	0.0	0.0	4.6	4.6	25.0	0.0	0.0	0.0	0.0	0.0	0.0
25	0.0	9.2	9.2	25.0	0.0	0.0	4.6	4.6	25.0	0.0	0.0	0.0	0.0	0.0	0.0
26	0.0	9.2	9.2	25.0	0.0	3.1	4.6	1.5	25.0	0.0	3.6	0.0	0.0	0.0	3.6
27	0.0	9.2	9.2	25.0	0.0	0.0	4.6	4.6	25.0	0.0	0.0	0.0	0.0	0.0	0.0
28	0.0	9.2	9.2	25.0	0.0	0.0	4.6	4.6	25.0	0.0	0.0	0.0	0.0	0.0	0.0
29						0.0	4.6	4.6	25.0	0.0	0.0	0.0	0.0	0.0	0.0
30						33.8	4.6	0.0	54.2	0.0	0.8	0.0	0.0	0.0	0.8
31						0.0	4.6	0.0	49.5	0.0					
TOTAL	0.0	258.0	258.0		0.0	36.9	143.5	131.1		0.0	66.9	8.7	0.0		107.8

CHAPTER 9

WATER BALANCE ANALYSIS OF THE RESERVOIR

9. Water Balance Analysis of the Reservoir

9.1. General

The Mabini reservoir project is planned purely for the irrigation purposes, but it is possible to use the quantity of water discharged for irrigation purposes, the surplus water and the available head for the hydro-power generation purposes.

The possibility of hydroelectric power generation is analized in the chapter of this report regarding "Power Generation". However, strictly speaking, the agricultural development component has priority in this project. Therefore, the required storage capacity of the Mabini reservoir is determined in the first place based upon the analysis of the water balance used purely for the irrigation purposes, and the possibility of hydroelectric power generation is analyzed next, having as object the quantity of water discharge required for the irrigation purposes and the surplus water.

9.2. Required Storage Capacity of the Reservoir

The study for the operation of the Mabini reservoir is carried out in monthly terms, based upon the conditions presented below.

(1) The period of study is assumed to extend from May, 1967 through April, 1970, and the study is started by assuming, at the start of analysis (May, 1967), that the stored water level taking place in the preceeding month is the low water level.

(2) The Normal water surface (NWS) of the Mabini reservoir is set at EL63.0m, by taking into consideration the topographical, geological factors and subcharge analysis.

(3) The evaporation from the surface of the reservoir is calculated by using the ratio between the evaporation from an open-rim pan and the evaporation from the storage reservoir. This ratio presents variations according to the seasons and depth of water, but generally speaking, the ratio are shown within the limits of 0.6 through 0.7 of total annual evaporation. Therefore, for the evaporation from the Mabini reservoir, 70% of the open-rim pan evaporation which takes place in the San Manual Meteorological Observation Station, which has data recorded over a long period of time, is adopted in the report.

(4) The leakage losses from the reservoir are influenced by factors of various kinds, but it is assumed to be 0.05%/day of the quantity of stored water, i.e., 1.5%/month in the report.

(5) The river maintenance discharge at the downstream side of the dam is estimated at $2.3m^3/s$ at the damsite, by taking into consideration $1.0m^3/s/100Km^2$ and is kept constant throughout the year.

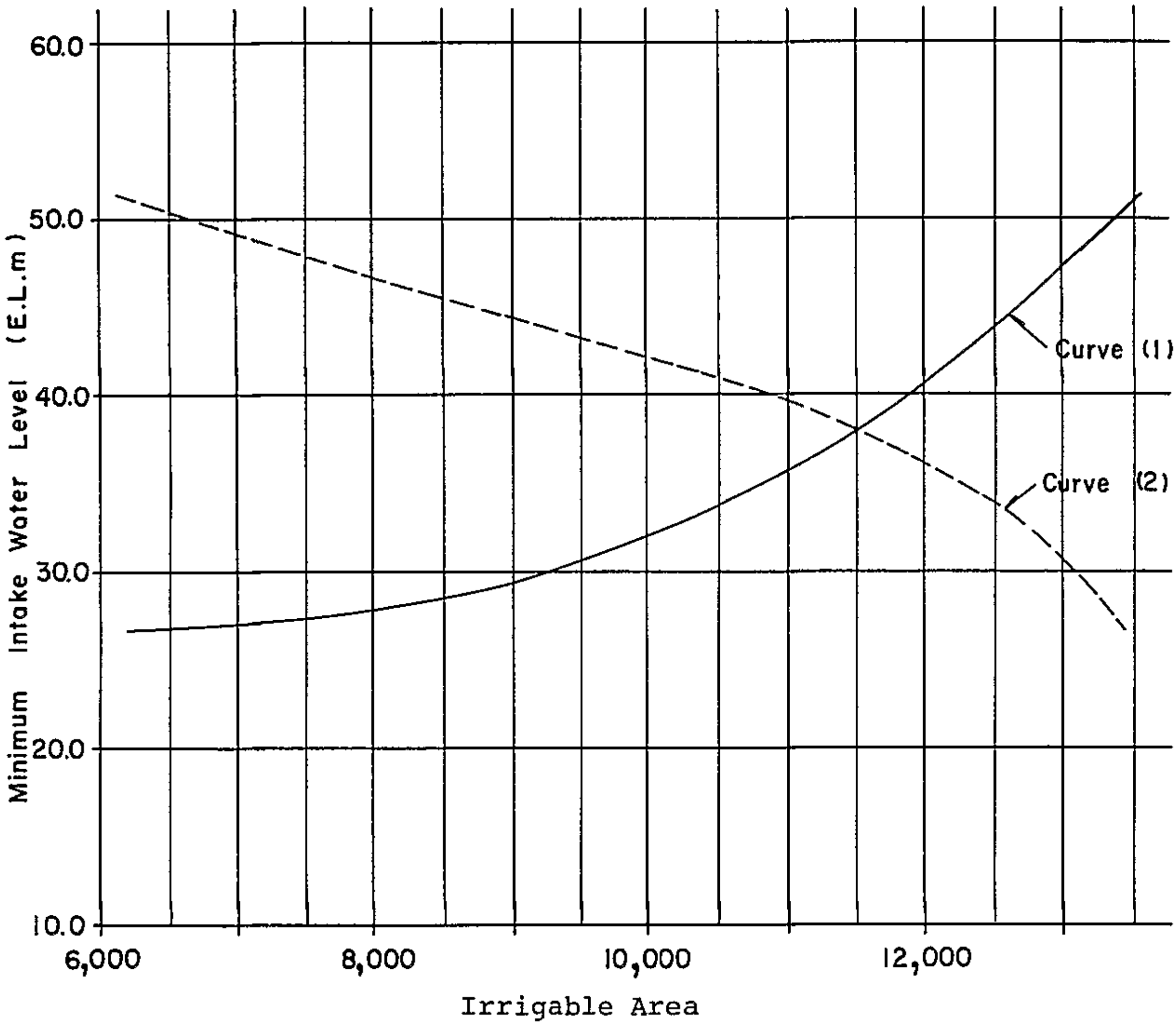
(6) The quantity of water required by unit irrigation area is adopted to be the value obtained in the Paragraph 8.2.4 of the main report.

The results of the series of water balance analysis based upon the premises described above are presented in Table 9.2.1. The relation between the irrigation area and the minimum water level obtained from the analysis is presented in Figure 9.2.1.

On the other hand, similar relation prevailing in the irrigable area estimated and planned based upon topographical factors, as described in the paragraph 10.3 of the main report, results into a curve decreasing in the leftward direction.

The planned irrigation area and the intake level are determined as 11,500ha and EL38.0m, respectively, from the two curves (with opposite tendencies) mentioned above.

Fig.9.2.1 Intake Water Level vs. Irrigation Area



NOTE: Curve (1): Irrigable area vs. intake water level curve based upon the topographical factors prevailing in the irrigable area.

Curve (2): Irrigable area vs. intake water level curve based upon the water balance analysis.

Table 9.2.1 Water Balance

Irrigable area (ha)	Calculated water level		Proposed intake water level	
	Max. water level*1	Min. water level*2	(1)*3	(2)*4
7,000	63.0	49.06	49.0	27.0
8,000	63.0	47.02	47.0	28.0
9,000	63.0	44.97	44.5	29.5
9,500	63.0	43.72	43.5	30.5
10,000	63.0	42.46	42.0	32.0
10,500	63.0	41.21	41.0	33.5
11,000	62.67	39.90	39.5	35.5
11,500	62.08	38.24	38.0	38.0
12,000	61.48	36.59	36.5	40.5
12,500	60.88	34.40	34.4	43.5
13,000	60.29	30.63	30.5	47.0

NOTE: *1 Result of the water balance analysis corresponding to the required irrigation area. (Period of May of 1968 through April of 1969)

*2 Intake level determined from the minimum water level of NOTE*1 above. (Period of May of 1968 through April of 1970)

*3 From NOTE*2 above.

*4 Intake level estimated based upon the topographical factors prevailing in the required irrigation area within the planned irrigable area.

9.3 Study of Operation of the Reservoir

A meeting for discussion of the Mabini Project based upon this F/S Draft Final Report with the concerned officers of the National Irrigation Administration (NIA) was held in 24th of February of 1982. NIA requested the study of the operation of the reservoir presented below be incorporated to the final report.

- To carry out the study of the water balance for a long period of time.
- To calculate the maximum irrigable area by the water balance study.

(1) Selection of the Period of Water Balance Study

The period of 11 years ranging from May of 1959 to April of 1970 is selected for the study, by taking into consideration the availability of meteorological and hydrographical data and the minimization of the need of complementation of the lacking data.

(2) Completion of Lacking Data

1) Daily Rainfall

Daily rainfall data of the Mabini Observation Station of the PAGASA corresponding to the period of January through May of 1965 are missing in the study period.

The missing daily rainfall data is supplemented as follows. The monthly rainfall occurring in Mabini is presumed, by the correlation analysis between the monthly rainfalls occurred in the Dagupan City Meteorological Station and in the

Mabini Meteorological Station. Next, the daily rainfall is allotted based upon data recorded in Dagupan City. The allotment of the daily rainfall data regarding to the months of January and February is impossible, because no rainfall occurred in Dagupan during the said period. Therefore, it is assumed that no rainfall occurred also in Mabini for the study.

2) Monthly Runoff

Measurement data regarding the runoff occurred at the planned dam site corresponding to the periods of May through July of 1961, April through June, August and September of 1969 are missing.

The complementation of the missing data is carried out by estimating them from the monthly rainfall observed at the Mabini Meteorological Station of the PAGASA. The monthly average runoff rate is calculated and then, the monthly runoff discharge is estimated from the monthly rainfall.

(3) Calculation Condition Regarding the Water Balance Study of the Reservoir

1) The normal water surface of the reservoir is at El.63.0m, while the low water surface is at El.38.0m.

2) The losses from the reservoir and the quantity of water required per unit irrigation area are taken same as those ones mentioned before in this report.

3) The river maintenance flow is taken to be $2.3m^3/s$.

4) The irrigation area is assumed to be 11,500ha. If the irrigation of the whole area of 11,500ha is impossible during a given period, the irrigable area is calculated by taking into consideration the conditions prevailing at that occasion. The conditions prevailing each year (rain season and dry season) are grasped by carrying out the study of the operation on a long term basis, in correspondence to various irrigation areas.

(4) <u>Results of the Study</u>

1) It is not possible to irrigate the whole area of 11,500ha during the period of May through October of 1962 (rain season). The area which can be irrigated during the aforesaid period is 270ha.

The aforesaid situation is due to the fact that the preceeding season (dry season: November of 1961 through April of 1962) corresponds to the most severe drought year, in terms of both rainfall and runoff. In addition, the values of both rainfall and runoff occurred in May and June of 1962 are relatively small compared with other years.

However, it is necessary to have in mind that the aforesaid calculations are carried out by using the proposed cropping pattern. It is considered that the influence of the drought can be restricted to a percentage of the order of 20% of the whole irrigable area even in case of very severe drought year, by changing the water control system and by postponing the commencement of the paddy cultivation by one month.

2) The water balance study of the reservoir is carried out on a long term basis in correspondence to various irrigation areas. Table 9.3.3 presents the shortage of water in each season (dry season and rain season) corresponding to each irrigation area taken into consideration in the study and the months when the water shortage take place.

The rates of successful irrigation are presented in Table 9.3.2. For example, the irrigation area corresponding to a successful irrigation rate of 80% throughout the period of approximately 11 years is 12,000ha and the irrigation area corresponding to a successful irrigation rate of 50% corresponds to 13,500ha, while the maximum irrigable area (corresponding to a successful irrigation rate of approximately 10%) is estimated to be of the order of 17,000ha.

Table 9.3.2 Irrigation Success Rate

Irrigation Area (ha)	Wet Season	Dry Season	Annual
11,500	9/10 = 0.90	11/11 = 1.0	20/21 = 0.95
12,000	7/10 = 0.70	10/11 = 0.91	17/21 = 0.81
12,500	6/10 = 0.60	10/11 = 0.91	16/21 = 0.76
13,000	6/10 = 0.60	7/11 = 0.64	13/21 = 0.62
13,500	6/10 = 0.60	6/11 = 0.55	12/21 = 0.57
14,000	4/10 = 0.40	5/11 = 0.45	9/21 = 0.43
15,000	4/10 = 0.40	3/11 = 0.27	7/21 = 0.33
16,000	4/10 = 0.40	2/11 = 0.18	6/21 = 0.29
17,000	3/10 = 0.30	1/11 = 0.09	4/21 = 0.19
18,000	2/10 = 0.20	0/11 = 0.0	2/21 = 0.09

Table 9.3.1 Runoff and Rainfall

Water year	Wet season		Dry season		Water year	
	runoff (MCM)	Rainfall (mm)	Runoff (MCM)	Rainfall (mm)	Runoff (MCM)	Rainfall (mm)
1959/60	703.2	1,782.9	192.0	225.9	895.2	2,008.8
1960/61	536.9	3,124.3	60.7	142.4	597.6	3,266.7
1961/62	-	3,229.4	29.4	53.2	-	3,282.6
1962/63	457.9	2,618.6	40.5	96.4	498.4	2,715.0
1963/64	508.9	3,609.6	57.6	278.4	566.5	3,888.0
1964/65	395.1	2,900.4	96.1	-	491.2	-
1965/66	363.8	-	41.9	238.7	405.7	-
1966/67	490.9	3,018.5	74.4	388.1	565.3	3,406.6
1967/68	422.9	3,531.4	79.1	210.2	502.0	3,741.6
1968/69	303.8	2,595.9	-	127.9	-	2,723.8
1969/70	-	3,508.4	69.5	207.2	-	3,715.6
1970/71	338.1	3,245.8	120.2	269.0	458.3	3,514.8
1971/72	395.0	2,243.0	-	169.2	-	2,412.2
1972/73	-	4,493.2	-	128.9	-	4,622.1
1973/74	-	2,298.3	70.9	144.0	-	2,442.3
1974/75	809.2	3,621.4	-	-	-	-
Mean	487.4	2,935.6	76.0	204.9	583.3	3,220.2

Table 9.3.3 Shortage Amount

	11,500 ha		12,000 ha		12,500 ha		13,000 ha		13,500 ha		14,000 ha		15,000 ha		16,000 ha		17,000 ha		18,000 ha	
	Wet season	Dry season	Wet season	Dry season	Wet season	Dry season	Wet season	Dry season	Wet season	Dry season	Wet season	Dry season	Wet season	Dry season	Wet season	Dry season	Wet season	Dry season	Wet season	Dry season
1959/60	-		-		-		-		-		-		-		-		-		-	7.14 (Mar. Apr.)
1960/61	-		-		-		-		-			1.13 (Apr.)		16.96 (Mar. Apr.)		32.93 (Mar. Apr.)	11.19 (May Jul.)	49.24 (Feb. Mar. Apr.)	19.83 (May Jul.)	65.55 (Feb. Mar. Apr.)
1961/62	-			2.67 (Apr.)		10.95 (Mar. Apr.)		19.34 (Mar. Apr.)		27.72 (Mar. Apr.)	3.30 (May)	36.80 (Mar. Apr.)	3.54 (May)	52.92 (Feb. Mar. Apr.)	3.77 (May)	69.97 (Feb. Mar. Apr.)	4.00 (May)	87.02 (Feb. Mar. Apr.)	4.23 (May)	104.08 (Feb. Mar. Apr.)
1962/63	21.30 (May Jun.)		27.64 (May Jun.)		28.59 (May Jun.)		29.54 (May Jun.)	4.56 (Apr.)	30.49 (May Apr.)	12.72 (Mar. Apr.)	31.44 (May Jun.)	20.89 (Mar. Apr.)	33.34 (May Jun.)	37.23 (Mar. Apr.)	35.24 (May Jun.)	53.64 (Feb. Mar. Apr.)	37.14 (May Jun.)	70.26 (Feb. Mar. Apr.)	39.03 (May Jun.)	86.88 (Feb. Mar. Apr.)
1963/64	-		3.69 (May)		11.94 (May)		15.73 (May)		16.20 (May)		16.67 (May)		17.62 (May)	11.24 (Mar. Apr.)	18.56 (May)	26.81 (Mar. Apr.)	19.50 (May)	42.41 (Feb. Mar. Apr.)	20.44 (May)	58.26 (Feb. Mar. Apr.)
1964/65	-		-		-		-		-		-		-		-			7.93 (Mar. Apr.)	1.81 (Jul.)	22.94 (Mar. Apr.)
1965/66	-		-		-			2.27 (Apr.)		10.23 (Mar. Apr.)		18.33 (Mar. Apr.)		34.53 (Mar. Apr.)		50.70 (Mar. Apr.)		67.02 (Feb. Mar. Apr.)		83.5 (Feb. Mar. Apr.)
1966/67	-		-		-		-		-		-			0.23 (Apr.)		14.58 (Mar. Apr.)		29.07 (Mar. Apr.)		43.55 (Mar. Apr.)
1967/68	-		-		-		-			1.42 (Apr.)	1.39 (May)	9.34 (Mar. Apr.)	15.75 (May)	25.43 (Mar. Apr.)	16.57 (May)	41.52 (Mar. Apr.)	17.39 (May)	57.67 (Feb. Mar. Apr.)	18.21 (May)	74.05 (Feb. Mar. Apr.)
1968/69	-		7.70 (Jun. Jul.)		16.73 (Jun. Jul.)		25.79 (May Jun. Jul.)	7.36 (Mar. Apr.)	33.54 (May Jun. Jul.)	15.58 (Mar. Apr.)	35.04 (May Jun. Jul.)	23.79 (Mar. Apr.)	38.04 (May Jun. Jul.)	40.20 (Mar. Apr.)	41.04 (May Jun. Jul.)	56.77 (Feb. Mar. Apr.)	44.03 (May Jun. Jul.)	73.49 (Feb. Mar. Apr.)	47.02 (May Jun. Jul.)	90.20 (Feb. Mar. Apr.)
1969/70	-		-		5.09 (May)		6.19 (May)		6.59 (May)		7.00 (May)		7.82 (May)		8.63 (May)	12.55 (Mar. Apr.)	9.45 (May)	28.13 (Mar. Apr.)	10.26 (May)	43.67 (Mar. Apr.)
	9/10	11/11	7/10	10/11	6/10	10/11	6/11	7/11	6/10	6/11	4/10	5/11	4/10	3/11	4/10	2/11	3/10	1/11	2/10	0/11

Table 9.3.4

RESERVOIR OPERATION STUDY (1) ---/ MABINI AGRICULTURAL DEVELOPMENT PROJECT /---

RESERVOIR		
NORMAL WATER LEVEL	EL. 63.00	(M)
TOTAL STORAGE CAPACITY	302.57	(MCM)
DEAD WATER LEVEL	EL. 38.00	(M)
DEAD STORAGE CAPACITY	63.24	(MCM)
IRRIGATION AREA	11500.0	(HA)
RIVER MAINTENANCE DISCHARGE	2.30	(M3/S)

YEAR	MONTH	RUNOFF	EVAPORATION LOSS		LEAKAGE LOSS	IRRIGATION DEMAND		RIVER MAINTE-NANCE	TOTAL RELEASE	STORAGE CAPA-CITY	EOM STORAGE CAPACITY	SPILL-OUT	SHORTAGE	ELEVATION OF RESERVOIR	IRRIGA-TION AREA
		/1/	/2/ (MM)	/3/	/4/	/5/ (MM)	/6/	/7/	/8/	/9/	/10/	/11/	/12/	/13/ (M)	/14/ (HA)
1959	MAY	7.84	179.2	0.99	0.95	60.0	6.90	6.16	15.00	-7.16	63.24	0.0	7.16	38.00	11500.0
	JUN	4.48	163.2	0.90	0.95	160.0	18.40	5.96	26.21	-21.73	63.24	0.0	21.73	38.00	11500.0
	JUL	14.82	117.7	0.65	0.95	208.7	24.00	6.16	31.76	-16.94	63.24	0.0	16.94	38.00	11500.0
	AUG	84.27	98.3	0.54	0.95	0.0	0.0	6.16	7.65	76.62	139.86	0.0	0.0	48.89	11500.0
	SEP	338.06	108.8	0.96	2.10	0.0	0.0	5.96	9.02	329.04	302.57	166.33	0.0	63.00	11500.0
	OCT	204.47	125.0	1.78	4.54	0.0	0.0	6.16	12.48	191.99	302.57	191.99	0.0	63.00	11500.0
	NOV	121.92	123.2	1.75	4.54	140.6	16.17	5.96	28.42	93.50	302.57	93.50	0.0	63.00	11500.0
	DEC	41.74	142.6	2.03	4.54	423.7	48.73	6.16	61.45	-19.71	282.86	0.0	0.0	61.60	11500.0
1960	JAN	3.79	148.3	2.03	4.24	452.9	52.08	6.16	64.52	-60.73	222.13	0.0	0.0	56.84	11500.0
	FEB	3.65	155.5	1.84	3.33	313.0	35.99	5.76	46.93	-43.28	178.84	0.0	0.0	52.94	11500.0
	MAR	3.88	185.2	1.91	2.68	205.5	23.63	6.16	34.39	-30.51	148.34	0.0	0.0	49.92	11500.0
	APR	3.54	164.1	1.51	2.23	3.8	0.44	5.96	10.13	-6.59	141.74	0.0	0.0	49.12	11500.0
TOTAL		832.46	1711.1	16.90	31.99	1968.2	226.34	72.73	347.96	484.50		451.81	45.82		
1960	MAY	7.79	158.3	1.41	2.13	56.8	6.53	6.16	16.23	-8.44	133.31	0.0	0.0	48.10	11500.0
	JUN	20.95	108.8	0.93	2.00	46.9	5.39	5.96	14.28	6.67	139.98	0.0	0.0	48.91	11500.0
	JUL	24.30	118.6	1.05	2.10	171.4	19.71	6.16	29.02	-4.72	135.26	0.0	0.0	48.34	11500.0
	AUG	317.19	85.5	0.74	2.03	0.0	0.0	6.16	8.92	308.27	302.57	140.95	0.0	63.00	11500.0
	SEP	58.06	90.5	1.29	4.54	0.0	0.0	5.96	11.79	46.27	302.57	46.27	0.0	63.00	11500.0
	OCT	71.04	107.6	1.53	4.54	0.0	0.0	6.16	12.23	58.81	302.57	58.81	0.0	63.00	11500.0
	NOV	28.99	135.1	1.92	4.54	152.3	17.51	5.96	29.94	-0.95	301.62	0.0	0.0	62.93	11500.0
	DEC	10.63	130.7	1.86	4.52	415.2	47.75	6.16	60.29	-49.66	251.96	0.0	0.0	59.30	11500.0
1961	JAN	6.28	169.8	2.18	3.78	483.8	55.64	6.16	67.76	-61.48	190.48	0.0	0.0	54.08	11500.0
	FEB	4.12	154.8	1.66	2.86	468.7	53.90	5.56	63.99	-59.87	130.62	0.0	0.0	47.78	11500.0
	MAR	4.35	180.0	1.51	1.96	133.4	15.34	6.16	24.97	-20.62	110.00	0.0	0.0	45.29	11500.0
	APR	2.17	198.6	1.48	1.65	16.2	1.86	5.96	10.95	-8.78	101.22	0.0	0.0	44.05	11500.0
TOTAL		555.87	1638.3	17.55	36.64	1944.7	223.64	72.53	350.36	205.51		246.03	0.0		

***** NOTE --- UNITS IN MILLION CUBIC METERS UNLESS OTHERWISE INDICATED. *****

RESERVOIR OPERATION STUDY (2)---/ MABINI AGRICULTURAL DEVELOPMENT PROJECT /---

RESERVOIR
- NORMAL WATER LEVEL EL. 63.00 (M)
- TOTAL STORAGE CAPACITY 302.57 (MCM)
- DEAD WATER LEVEL EL. 38.00 (M)
- DEAD STORAGE CAPACITY 63.24 (MCM)

IRRIGATION AREA 11500.0 (HA)
RIVER MAINTENANCE DISCHARGE 2.30 (M3/S)

YEAR	MONTH	RUNOFF /1/	EVAPORATION LOSS /2/ (MM)	/3/	LEAKAGE LOSS /4/	IRRIGATION DEMAND /5/ (MM)	/6/	RIVER MAINTE-NANCE /7/	TOTAL RELEASE /8/	STORAGE CAPA-CITY /9/	EOM STORAGE CAPACITY /10/	SPILL-OUT /11/	SHORTAGE /12/	ELEVATION OF RESERVOIR /13/ (M)	IRRIGA-TION AREA /14/ (HA)
1961	MAY	7.81	134.6	0.96	1.52	23.3	2.68	6.16	11.31	-3.50	97.71	0.0	0.0	43.53	11500.0
	JUN	71.01	100.8	0.70	1.47	122.0	14.03	5.96	22.16	48.85	146.56	0.0	0.0	49.70	11500.0
	JUL	85.07	74.7	0.68	2.20	0.0	0.0	6.16	9.04	76.03	222.59	0.0	0.0	56.88	11500.0
	AUG	100.05	84.8	1.01	3.34	0.0	0.0	6.16	10.51	89.54	302.57	9.56	0.0	63.00	11500.0
	SEP	121.30	75.9	1.08	4.54	0.0	0.0	5.96	11.58	109.72	302.57	109.72	0.0	63.00	11500.0
	OCT	31.15	74.3	1.06	4.54	0.0	0.0	6.16	11.76	19.39	302.57	19.39	0.0	63.00	11500.0
	NOV	8.66	93.4	1.33	4.54	126.1	14.50	5.96	26.33	-17.67	284.90	0.0	0.0	61.74	11500.0
	DEC	7.09	103.0	1.42	4.27	426.7	49.07	6.16	60.92	-53.83	231.07	0.0	0.0	57.58	11500.0
1962	JAN	4.31	115.6	1.40	3.47	483.8	55.64	6.16	66.67	-62.36	168.71	0.0	0.0	51.94	11500.0
	FEB	2.36	142.4	1.42	2.53	459.2	52.81	5.56	62.32	-59.96	108.75	0.0	0.0	45.14	11500.0
	MAR	2.30	157.4	1.16	1.63	229.5	26.39	6.16	35.35	-33.05	75.70	0.0	0.0	40.26	11500.0
	APR	2.70	142.4	0.88	1.14	14.5	1.67	5.96	9.65	-6.95	68.76	0.0	0.0	39.03	11500.0
TOTAL		443.61	1299.3	13.10	35.17	1865.1	216.79	72.53	337.59	106.22		138.67	0.0		
1962	MAY	3.65	141.2	0.82	1.03	87.2	10.03	6.16	18.04	-14.39	63.24	0.0	8.87	38.00	11500.0
	JUN	6.89	108.6	0.60	0.95	102.7	11.81	5.96	19.32	-12.43	63.24	0.0	12.43	38.00	11500.0
	JUL	132.88	74.0	0.41	0.95	0.0	0.0	6.16	7.52	125.36	188.60	0.0	0.0	53.90	11500.0
	AUG	95.51	85.0	0.91	2.83	0.0	0.0	6.16	9.90	85.61	274.21	0.0	0.0	60.98	11500.0
	SEP	126.07	67.2	0.91	4.11	0.0	0.0	5.96	10.98	115.09	302.57	86.73	0.0	63.00	11500.0
	OCT	60.84	94.2	1.34	4.54	0.0	0.0	6.16	12.04	48.80	302.57	48.80	0.0	63.00	11500.0
	NOV	13.85	102.6	1.46	4.54	121.6	13.98	5.96	25.95	-12.10	290.48	0.0	0.0	62.14	11500.0
	DEC	8.02	122.1	1.70	4.36	417.1	47.97	6.16	60.18	-52.16	238.31	0.0	0.0	58.18	11500.0
1963	JAN	5.77	126.1	1.56	3.57	479.3	55.12	6.16	66.42	-60.65	177.67	0.0	0.0	52.82	11500.0
	FEB	3.55	133.7	1.43	2.66	429.1	49.35	5.56	59.00	-55.45	122.21	0.0	0.0	46.77	11500.0
	MAR	3.35	125.9	1.01	1.83	241.6	27.78	6.16	36.78	-33.43	88.78	0.0	0.0	42.20	11500.0
	APR	3.15	179.5	1.20	1.33	7.1	0.82	5.96	9.31	-6.16	82.62	0.0	0.0	41.29	11500.0
TOTAL		465.53	1365.1	13.34	32.71	1885.7	216.86	72.53	335.43	128.10		135.53	21.30		

***** NOTE --- UNITS IN MILLION CUBIC METERS UNLESS OTHERWISE INDICATED. *****

RESERVOIR OPERATION STUDY(3) ---/ MABINI AGRICULTURAL DEVELOPMENT PROJECT /---

RESERVOIR
NORMAL WATER LEVEL EL. 63.00 (M)
TOTAL STORAGE CAPACITY 302.57 (MCM)
DEAD WATER LEVEL EL. 38.00 (M)
DEAD STORAGE CAPACITY 63.24 (MCM)
IRRIGATION AREA 11500.0 (HA)
RIVER MAINTENANCE DISCHARGE 2.30 (M3/S)

YEAR	MONTH	RUNOFF	EVAPORATION LOSS		LEAKAGE LOSS	IRRIGATION DEMAND		RIVER MAINTE-NANCE	TOTAL RELEASE	STORAGE CAPA-CITY	EOM STORAGE CAPACITY	SPILL-OUT	SHORTAGE	ELEVATION OF RESERVOIR	IRRIGA-TION AREA
		/1/	/2/ (MM)	/3/	/4/	/5/ (MM)	/6/	/7/	/8/	/9/	/10/	/11/	/12/	/13/ (M)	/14/ (HA)
1963	MAY	4.59	178.4	1.15	1.24	94.1	10.82	6.16	19.37	-14.78	67.84	0.0	0.0	38.86	11500.0
	JUN	104.41	80.4	0.46	1.02	0.0	0.0	5.96	7.44	96.97	164.81	0.0	0.0	51.55	11500.0
	JUL	99.93	74.5	0.73	2.47	60.5	6.96	6.16	16.32	83.61	248.42	0.0	0.0	59.01	11500.0
	AUG	89.88	86.8	1.11	3.73	26.3	3.02	6.16	14.02	75.86	302.57	21.71	0.0	63.00	11500.0
	SEP	136.68	67.9	0.97	4.54	0.0	0.0	5.96	11.47	125.21	302.57	125.21	0.0	63.00	11500.0
	OCT	37.90	98.8	1.41	4.54	0.0	0.0	6.16	12.11	25.79	302.57	25.79	0.0	63.00	11500.0
	NOV	20.93	98.3	1.40	4.54	91.4	10.51	5.96	22.41	-1.48	301.09	0.0	0.0	62.89	11500.0
	DEC	9.82	102.1	1.45	4.52	375.6	43.19	6.16	55.32	-45.50	255.59	0.0	0.0	59.60	11500.0
1964	JAN	7.35	120.4	1.56	3.83	495.5	56.98	6.16	68.54	-61.19	194.40	0.0	0.0	54.47	11500.0
	FEB	5.54	160.2	1.74	2.92	463.3	53.28	5.76	63.70	-58.16	136.24	0.0	0.0	48.46	11500.0
	MAR	5.32	177.1	1.53	2.04	191.1	21.98	6.16	31.71	-26.39	109.85	0.0	0.0	45.27	11500.0
	AFR	4.53	188.3	1.40	1.65	0.0	0.0	5.96	9.01	-4.48	105.37	0.0	0.0	44.67	11500.0
TOTAL		526.88	1433.1	14.91	37.03	1797.8	206.75	72.73	331.42	195.46		172.72	0.0		
1964	MAY	9.53	134.3	0.97	1.58	0.0	0.0	6.16	8.71	0.82	106.18	0.0	0.0	44.79	11500.0
	JUN	33.03	85.2	0.62	1.59	60.3	6.93	5.96	15.11	17.92	124.10	0.0	0.0	46.99	11500.0
	JUL	22.80	99.7	0.81	1.86	183.7	21.13	6.16	29.95	-7.15	116.95	0.0	0.0	46.13	11500.0
	AUG	151.66	78.3	0.61	1.75	0.0	0.0	6.16	8.52	143.14	260.09	0.0	0.0	59.97	11500.0
	SEP	68.30	79.0	1.04	3.90	0.0	0.0	5.96	10.90	57.40	302.57	14.92	0.0	63.00	11500.0
	OCT	82.22	84.8	1.21	4.54	0.0	0.0	6.16	11.91	70.31	302.57	70.31	0.0	63.00	11500.0
	NOV	27.19	58.9	0.84	4.54	24.3	2.79	5.96	14.13	13.06	302.57	13.06	0.0	63.00	11500.0
	DEC	39.26	84.5	1.20	4.54	403.2	46.37	6.16	58.27	-19.01	283.56	0.0	0.0	61.65	11500.0
1965	JAN	11.28	101.1	1.39	4.25	489.6	56.30	6.16	68.11	-56.83	226.73	0.0	0.0	57.22	11500.0
	FEB	4.13	107.7	1.29	3.40	459.7	52.87	5.56	63.12	-58.99	167.74	0.0	0.0	51.84	11500.0
	MAR	5.67	142.8	1.42	2.52	199.5	22.94	6.16	33.03	-27.36	140.38	0.0	0.0	48.96	11500.0
	AFR	5.12	137.9	1.22	2.11	0.0	0.0	5.96	9.29	-4.17	136.21	0.0	0.0	48.45	11500.0
TOTAL		460.19	1194.2	12.61	36.58	1820.3	209.33	72.53	331.06	129.13		98.29	0.0		

***** NOTE --- UNITS IN MILLION CUBIC METERS UNLESS OTHERWISE INDICATED. *****

RESERVOIR OPERATION STUDY (4)---/ MABINI AGRICULTURAL DEVELOPMENT PROJECT /---

RESERVOIR	
NORMAL WATER LEVEL	EL. 63.00 (M)
TOTAL STORAGE CAPACITY	302.57 (MCM)
DEAD WATER LEVEL	EL. 38.00 (M)
DEAD STORAGE CAPACITY	63.24 (MCM)
IRRIGATION AREA	11500.0 (HA)
RIVER MAINTENANCE DISCHARGE	2.30 (M3/S)

YEAR	MONTH	RUNOFF	EVAPORATION LOSS		LEAKAGE LOSS	IRRIGATION DEMAND		RIVER MAINTE-NANCE	TOTAL RELEASE	STORAGE CAPA-CITY	EOM STORAGE CAPACITY	SPILL-OUT	SHORTAGE	ELEVATION OF RESERVOIR	IRRIGA-TION AREA
		/1/	/2/ (MM)	/3/	/4/	/5/ (MM)	/6/	/7/	/8/	/9/	/10/	/11/	/12/	/13/ (M)	/14/ (HA)
1965	MAY	22.51	138.0	1.19	2.04	10.1	1.16	6.16	10.56	11.95	148.16	0.0	0.0	49.90	11500.0
	JUN	84.23	101.8	0.94	2.22	93.3	10.73	5.96	19.85	64.38	212.54	0.0	0.0	56.05	11500.0
	JUL	81.16	106.0	1.22	3.19	38.2	4.39	6.16	14.96	66.20	278.74	0.0	0.0	61.30	11500.0
	AUG	78.84	104.4	1.42	4.18	0.0	0.0	6.16	11.76	67.08	302.57	43.25	0.0	63.00	11500.0
	SEP	49.24	90.2	1.28	4.54	0.0	0.0	5.96	11.78	37.46	302.57	37.46	0.0	63.00	11500.0
	OCT	22.30	115.2	1.64	4.54	0.0	0.0	6.16	12.34	9.96	302.57	9.96	0.0	63.00	11500.0
	NOV	17.25	95.3	1.36	4.54	97.0	11.15	5.96	23.01	-5.76	296.81	0.0	0.0	62.59	11500.0
	DEC	9.03	119.8	1.69	4.45	414.0	47.61	6.16	59.91	-50.88	245.93	0.0	0.0	58.81	11500.0
1966	JAN	5.45	113.4	1.43	3.69	451.0	51.86	6.16	63.15	-57.70	188.23	0.0	0.0	53.86	11500.0
	FEB	2.63	136.8	1.46	2.82	432.8	49.77	5.56	59.62	-56.99	131.24	0.0	0.0	47.86	11500.0
	MAR	2.34	197.5	1.66	1.97	273.4	31.44	6.16	41.23	-38.89	92.35	0.0	0.0	42.74	11500.0
	APR	2.33	168.9	1.28	1.39	12.0	1.38	5.96	10.01	-7.68	84.67	0.0	0.0	41.59	11500.0
TOTAL		377.31	1507.4	16.58	39.57	1821.8	209.51	72.53	338.19	39.12		90.66	0.0		
1966	MAY	39.46	150.7	0.98	1.27	13.4	1.54	6.16	9.95	29.51	114.18	0.0	0.0	45.80	11500.0
	JUN	53.25	89.9	0.69	1.71	191.9	22.07	5.96	30.43	22.82	137.00	0.0	0.0	48.55	11500.0
	JUL	77.23	102.3	0.89	2.05	114.6	13.18	6.16	22.28	54.95	191.95	0.0	0.0	54.23	11500.0
	AUG	86.59	116.5	1.26	2.88	0.0	0.0	6.16	10.30	76.29	268.24	0.0	0.0	60.56	11500.0
	SEP	164.04	80.2	1.07	4.02	0.0	0.0	5.96	11.06	152.98	302.57	118.65	0.0	63.00	11500.0
	OCT	36.02	119.1	1.70	4.54	0.0	0.0	6.16	12.39	23.63	302.57	23.63	0.0	63.00	11500.0
	NOV	32.89	79.5	1.13	4.54	13.4	1.54	5.96	13.17	19.72	302.57	19.72	0.0	63.00	11500.0
	DEC	14.18	96.2	1.37	4.54	305.7	35.16	6.16	47.22	-33.04	269.53	0.0	0.0	60.65	11500.0
1967	JAN	7.58	121.9	1.63	4.04	470.7	54.13	6.16	65.96	-58.38	211.14	0.0	0.0	55.93	11500.0
	FEB	5.51	157.8	1.81	3.17	462.7	53.21	5.56	63.75	-58.24	152.90	0.0	0.0	50.38	11500.0
	MAR	4.46	200.6	1.88	2.29	247.4	28.45	6.16	38.79	-34.33	118.57	0.0	0.0	46.33	11500.0
	APR	4.43	188.6	1.48	1.78	11.1	1.28	5.96	10.49	-6.06	112.51	0.0	0.0	45.59	11500.0
TOTAL		525.64	1503.4	15.88	36.84	1830.9	210.55	72.53	335.81	189.83		161.99	0.0		

***** NOTE --- UNITS IN MILLION CUBIC METERS UNLESS OTHERWISE INDICATED. *****

RESERVOIR OPERATION STUDY (5)---/ MABINI AGRICULTURAL DEVELOPMENT PROJECT /---

RESERVOIR

NORMAL WATER LEVEL	EL. 63.00 (M)
TOTAL STORAGE CAPACITY	302.57 (MCM)
DEAD WATER LEVEL	EL. 38.00 (M)
DEAD STORAGE CAPACITY	63.24 (MCM)
IRRIGATION AREA	11500.0 (HA)
RIVER MAINTENANCE DISCHARGE	2.30 (M3/S)

YEAR	MONTH	RUNOFF	EVAPORATION LOSS		LEAKAGE LOSS	IRRIGATION DEMAND		RIVER MAINTE-NANCE	TOTAL RELEASE	STORAGE CAPA-CITY	EOM STORAGE CAPACITY	SPILL-OUT	SHORTAGE	ELEVATION OF RESERVOIR	IRRIGA-TION AREA
		/1/	/2/ (MM)	/3/	/4/	/5/ (MM)	/6/	/7/	/8/	/9/	/10/	/11/	/12/	/13/ (M)	/14/ (HA)
1967	MAY	4.76	203.1	1.53	1.69	81.9	9.42	6.16	18.80	-14.04	98.47	0.0	0.0	43.64	11500.0
	JUN	70.65	94.7	0.66	1.48	106.8	12.28	5.96	20.38	50.27	148.74	0.0	0.0	49.97	11500.0
	JUL	74.21	117.0	1.08	2.23	108.3	12.45	6.16	21.92	52.29	201.02	0.0	0.0	55.10	11500.0
	AUG	101.42	82.8	0.92	3.02	0.0	0.0	6.16	10.10	91.32	292.34	0.0	0.0	62.27	11500.0
	SEP	75.37	99.8	1.39	4.39	0.0	0.0	5.96	11.74	63.63	302.57	53.40	0.0	63.00	11500.0
	OCT	67.04	117.3	1.67	4.54	0.0	0.0	6.16	12.37	54.67	302.57	54.67	0.0	63.00	11500.0
	NOV	37.43	103.6	1.48	4.54	16.4	1.89	5.96	13.86	23.57	302.57	23.57	0.0	63.00	11500.0
	DEC	15.66	139.3	1.98	4.54	449.4	51.68	6.16	64.36	-48.70	253.87	0.0	0.0	59.46	11500.0
1968	JAN	7.43	148.0	1.91	3.81	484.0	55.66	6.16	67.54	-60.11	193.76	0.0	0.0	54.40	11500.0
	FEB	5.04	152.7	1.66	2.91	472.6	54.35	5.76	64.68	-59.64	134.12	0.0	0.0	48.20	11500.0
	MAR	4.33	203.3	1.74	2.01	248.9	28.62	6.16	38.53	-34.20	99.92	0.0	0.0	43.86	11500.0
	APR	3.65	195.5	1.38	1.50	13.1	1.51	5.96	10.35	-6.70	93.22	0.0	0.0	42.86	11500.0
TOTAL		466.99	1657.2	17.41	36.64	1981.4	227.86	72.73	354.64	112.35		131.64	0.0		
1968	MAY	4.49	175.8	1.20	1.40	40.7	4.68	6.16	13.44	-8.95	84.27	0.0	0.0	41.54	11500.0
	JUN	5.46	109.2	0.71	1.26	144.9	16.66	5.96	24.60	-19.14	65.13	0.0	0.0	38.35	11500.0
	JUL	20.19	92.5	0.52	0.98	114.2	13.13	6.16	20.79	-0.60	64.53	0.0	0.0	38.24	11500.0
	AUG	92.22	80.8	0.45	0.97	0.0	0.0	6.16	7.58	84.64	149.17	0.0	0.0	50.02	11500.0
	SEP	93.43	86.0	0.79	2.24	0.0	0.0	5.96	8.99	89.44	238.61	0.0	0.0	58.20	11500.0
	OCT	61.77	85.8	1.06	3.58	0.0	0.0	6.16	10.80	50.97	289.58	0.0	0.0	62.08	11500.0
	NOV	22.16	95.2	1.32	4.34	117.8	13.55	5.96	25.18	-3.02	286.56	0.0	0.0	61.86	11500.0
	DEC	8.84	89.7	1.24	4.30	459.7	52.87	6.16	64.56	-55.72	230.84	0.0	0.0	57.56	11500.0
1969	JAN	7.37	101.8	1.24	3.46	477.4	54.90	6.16	65.76	-58.39	172.45	0.0	0.0	52.31	11500.0
	FEB	4.47	126.0	1.27	2.59	410.6	47.22	5.56	56.64	-52.17	120.28	0.0	0.0	46.53	11500.0
	MAR	3.19	165.7	1.31	1.80	239.1	27.50	6.16	36.77	-33.58	86.69	0.0	0.0	41.90	11500.0
	APR	2.96	171.9	1.13	1.30	1.6	0.18	5.96	8.58	-5.62	81.08	0.0	0.0	41.06	11500.0
TOTAL		331.55	1380.5	12.25	28.22	2006.0	230.69	72.53	343.69	-12.14		0.0	0.0		

***** NOTE --- UNITS IN MILLION CUBIC METERS UNLESS OTHERWISE INDICATED. *****

RESERVOIR OPERATION STUDY (6)---/ MABINI AGRICULTURAL DEVELOPMENT PROJECT /---

RESERVOIR
NORMAL WATER LEVEL EL. 63.00 (M)
TOTAL STORAGE CAPACITY 302.57 (MCM)
DEAD WATER LEVEL EL. 38.00 (M)
DEAD STORAGE CAPACITY 63.24 (MCM)
IRRIGATION AREA 11500.0 (HA)
RIVER MAINTENANCE DISCHARGE 2.30 (M3/S)

YEAR	MONTH	RUNOFF	EVAPORATION LOSS		LEAKAGE LOSS	IRRIGATION DEMAND		RIVER MAINTE-NANCE	TOTAL RELEASE	STORAGE CAPA-CITY	EOM STORAGE CAPACITY	SPILL-OUT	SHORTAGE	ELEVATION OF RESERVOIR	IRRIGA-TION AREA
		/1/	/2/ (MM)	/3/	/4/	/5/ (MM)	/6/	/7/	/8/	/9/	/10/	/11/	/12/	/13/ (M)	/14/ (HA)
1969	MAY	12.52	182.1	1.16	1.22	81.5	9.37	6.16	17.91	-5.39	75.68	0.0	0.0	40.26	11500.0
	JUN	31.13	101.9	0.63	1.14	51.4	5.91	5.96	13.64	17.49	93.17	0.0	0.0	42.86	11500.0
	JUL	102.85	101.4	0.69	1.40	52.2	6.00	6.16	14.25	88.60	181.77	0.0	0.0	53.22	11500.0
	AUG	158.01	119.3	1.24	2.73	0.0	0.0	6.16	10.13	147.88	302.57	27.08	0.0	63.00	11500.0
	SEP	137.29	74.8	1.06	4.54	0.0	0.0	5.96	11.56	125.73	302.57	125.73	0.0	63.00	11500.0
	OCT	60.81	81.6	1.16	4.54	0.0	0.0	6.16	11.86	48.95	302.57	48.95	0.0	63.00	11500.0
	NOV	18.63	84.5	1.20	4.54	114.5	13.17	5.96	24.87	-6.24	296.33	0.0	0.0	62.56	11500.0
	DEC	15.44	95.8	1.35	4.44	415.5	47.78	6.16	59.74	-44.30	252.04	0.0	0.0	59.31	11500.0
1970	JAN	11.19	107.7	1.39	3.78	413.6	47.56	6.16	58.89	-47.70	204.34	0.0	0.0	55.37	11500.0
	FEB	8.59	111.2	1.25	3.07	444.8	51.15	5.56	61.03	-52.44	151.89	0.0	0.0	50.28	11500.0
	MAR	7.93	138.5	1.29	2.28	226.1	26.00	6.16	35.73	-27.80	124.09	0.0	0.0	46.99	11500.0
	APR	4.45	163.0	1.32	1.86	0.0	0.0	5.96	9.14	-4.69	119.40	0.0	0.0	46.43	11500.0
TOTAL		568.84	1361.9	13.75	35.52	1799.6	206.95	72.53	328.76	240.08		201.75	0.0		

CHAPTER 10

OUTLINE OF THE SERVICE AREA

10. Outline of the Service Area

10.1. Conditions Prevailing in the Service Area

The relation between the arable land and the altitudes within the project area, determined based upon the 1:50,000 scale topographical map and using complementarily the 1:4,000 scale map provided by the NIA, is presented in Figure 10.1.1.

10.2. Intake Water Level and Service Area

Losses from the intake tunnel, slope of the main irrigation canal, altitude at the extremity of the irrigation canal, etc., are assumed to have the values listed in the table below , for the purpose of determining the relation between the intake water level of the dam and the service area.

Elevation		*2 Min. intake water level	Area		Remarks
End point	*1 Intake point		Total	Arable	
10.0m	24.8m	27.0m	9,930ha	6,950ha	
20.2	34.8	37.0	16,140	11,300	
30.3	45.1	47.0	18,500	12,950	

NOTE:

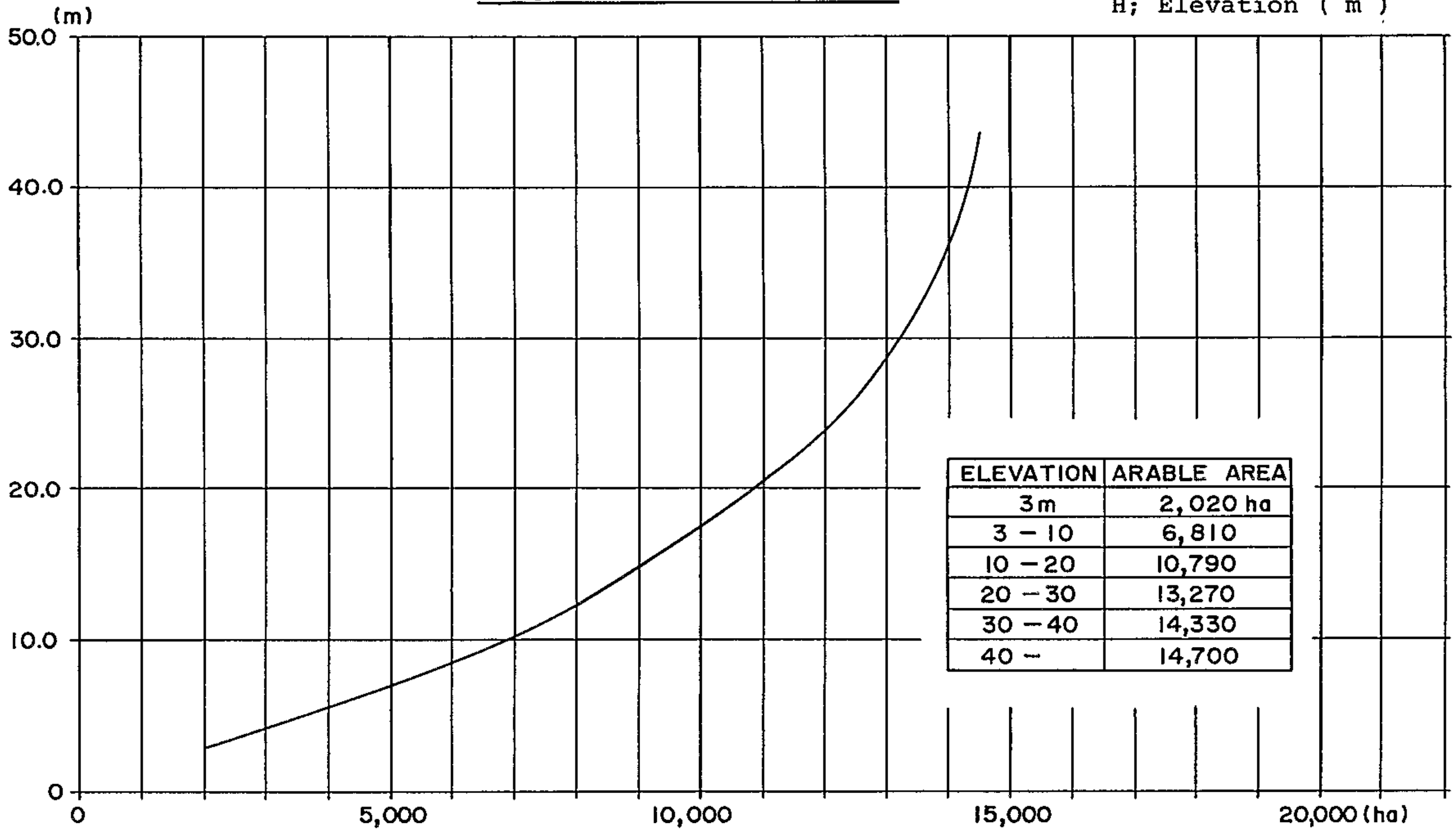
*1 The slope of the irrigation canal is assumed to be 1:2,500. The extensions of the waterways are assumed to be 7Km for the driving channel and 26Km for the main irrigation canal (west main canal).

*2 The inflow and run-off losses of the intake tunnel and the loss at the check gate of the irrigation canal are assumed to be of the order of 2.2m which is added to the altitude of the intake water level.

The relation between the intake water level and the arable area is presented in Figure 10.2.1.

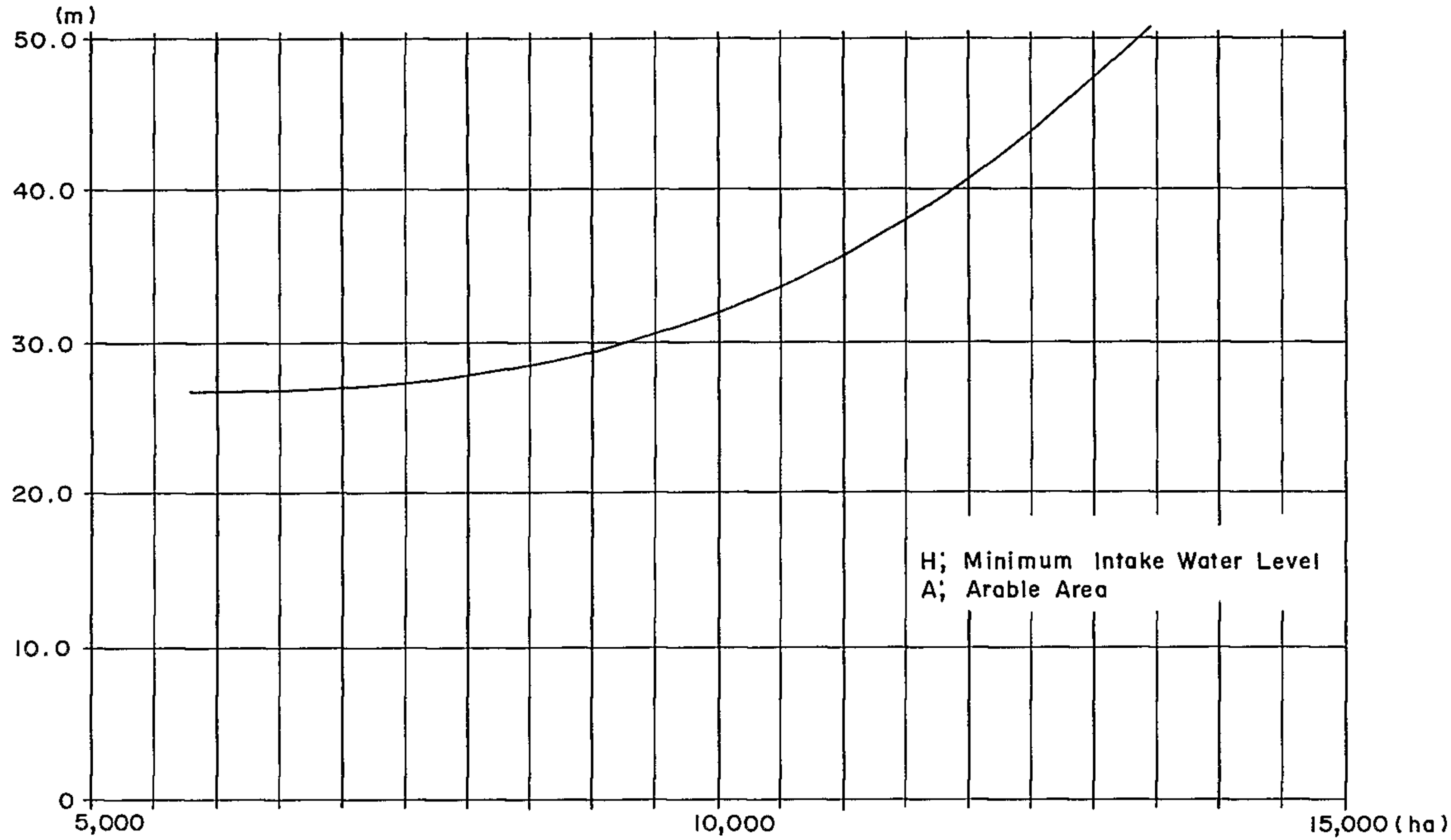
Fig. 10.1.1 H - A Curve

A; Arable Area (ha)
H; Elevation (m)



ELEVATION	ARABLE AREA
3m	2,020 ha
3 - 10	6,810
10 - 20	10,790
20 - 30	13,270
30 - 40	14,330
40 -	14,700

Fig.10.2.1 H - A Curve



CHAPTER 11

STUDY OF DAM TYPE

11. STUDY OF DAM TYPE

11.1 Determination of Dam Height

11.1.1 Height of Wave caused by Wind (hw) and Height of Wave caused by Earthquake (he)

1) Height of wave caused by wind

The height of wave caused by wind (hw) is calculated from the creeping diagram (Figure 11.1.1), by means of the SMB Method (Sherdrup-Munk-Breschneider) and Saville's Method.

Wind velocity (average of 10 minutes)	30 m/sec
Upstream slope (Riprap)	1:3.0
Fetch	4,800 m

Under the conditions presented above, the height of wave caused by wind is hw = 1.10 m.

2) Height of wave caused by earthquake

The height of wave caused by earthquake is calculated by the formula of Seiichi Sato

$$he = 1/2 \cdot \frac{k.\tau}{\pi} \cdot \sqrt{g.H_o}$$

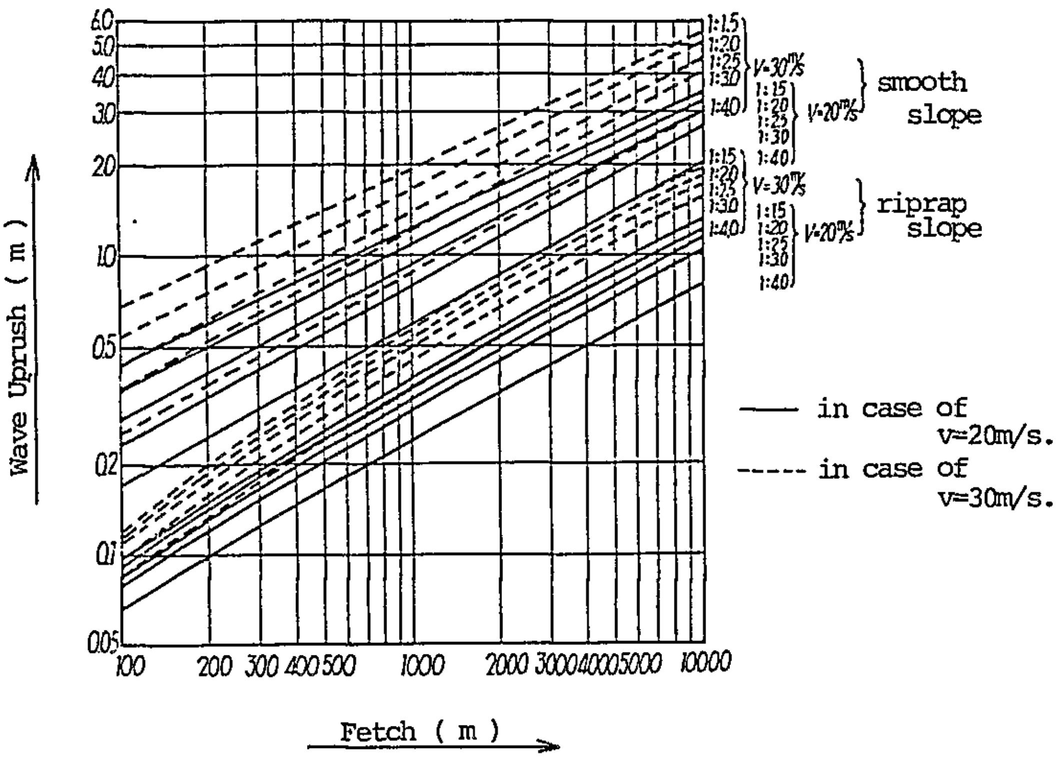
where, he: Height of wave caused by earthquake

k :	Seismic coefficient	0.2
τ :	Earthquake cycle	1.0 sec
g :	Acceleration of gravity	9.8 m /sec
Ho:	Depth of the storage reservoir at the occasion of NWS	63 - 12 = 51 m

$\therefore$ he = 0.71 ≒ 0.75 m

Accordingly, the height of wave caused by earthquake will be he = 0.75 m.

Fig. 11-1-1 Height of Wave caused by Wind



11.2 Countermeasures to Protect Seepage through Foundation of the Temporary Cofferdam

The blanket method and the slurry trench method can be taken into consideration as methods to restrict the seepage water coming out from the foundation of the temporary cofferdam.

When seepage coefficient of the riverbed materials is $k = 1 \times 10^{-1}$cm/s, and no countermeasure is taken for seepage, the total leakage amount through foundation of cofferdam will be estimated at more than 300 cubic meters per minute. In this case, quite big pumping facilities which require considerable amount of electric power are necessary to drain the leakage water coming out to the cut off trench from riverbed materials. Therefore, to protect the leakage water to the trench, blanket and/or slurry trench method will be required.

1) In case of no countermeasure for seepage

Seepage water amount per meter of width will be calculated by the following formula.

$$q_f = \frac{k \cdot d \cdot h}{x_d}$$

where, k : Seepage coefficient 1×10^{-1}cm/s

d : Thickness of pervious zone, means riverbed materials 30 m

x_d: Bottom width of impervious zone 16 m

The results of calculation is shown below.

$q_f = 0.043$ m^3/s

Q = 5,391 m^3/s = 323.4 m^3/min

2) In case of blanket method

Seepage water amount per meter of width will be calculated by the following formula.

$$q_x = \frac{k \cdot d \cdot h}{x_r + x_d}$$

$$x_r = \frac{e^{zax} - 1}{a(e^{zax} + 1)}$$

$$a = \sqrt{\frac{k_1}{t \cdot k \cdot d}}$$

where, k : Seepage coefficient $K = 1 \times 10^{-1}$ cm/s

d : Thickness of pervious zone (riverbed materials) 30 m

k : Seepage coefficient of the blanket $K = 1 \times 10^{-5}$ cm/sec

t : Thickness of the blanket 3 m

x : Length of the blanket 140 m

The result of the computation is shown below.

$a = 1.054 \times 10^{-3}$

$x_r = 138.9$ m

$q_f = 3.065 \times 10^{-3}$ m^3/sec

$Q = 0.388$ m^3/sec $= 22.99$ m^3/min

3) In case of slurry trench

The thickness of slurry trench is 0.6 m

Seepage water amount is estimated by the following formula.

$$qf = \frac{K_1 \cdot K_2 \cdot (h_1 - h_2)}{K_2 \cdot L_1 + K_1 \cdot L_2} \cdot d$$

q_f : Seepage water amount (m^3/s)

K_1 : Seepage coefficient of previous zone (means river bed materials)

$K = 1 \times 10^{-1}$ cm/s

K_2 : Seepage coefficient of slurry trench

$K = 1 \times 10^{-5}$ cm/s

L_1 : Horizontal length of previous zone

113.2 m

L_2 : Thickness of slurry trench 0.6 m

h_1 : Head of upstream 53 m

h_2 : Head of downstream 30 m

d : Thickness of pervious zone 30 m

Results of the computation is shown below.

$K_1K_2(h_1 - h_2) = 2.3 \times 10^{-9}$

$K_2L_1 + K_1L_2 = 6.11 \times 10^{-4}$

$qf = 1.129 \times 10^{-4}$

$Q = 0.014\ m^3/s = 0.847\ m^3/min.$

Fig.11.2.1

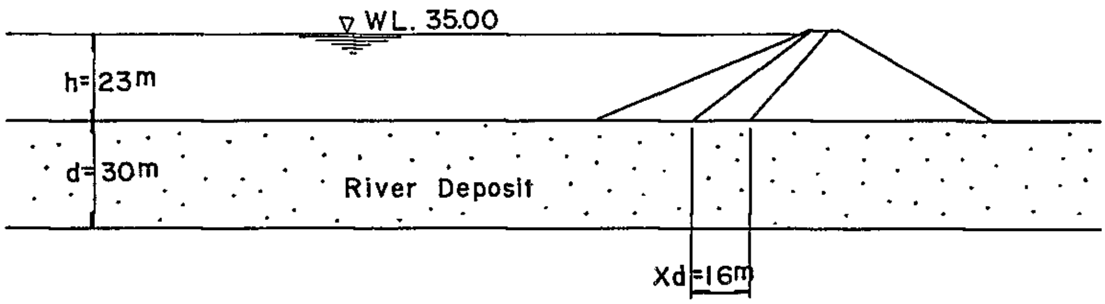


Fig.11.2.2

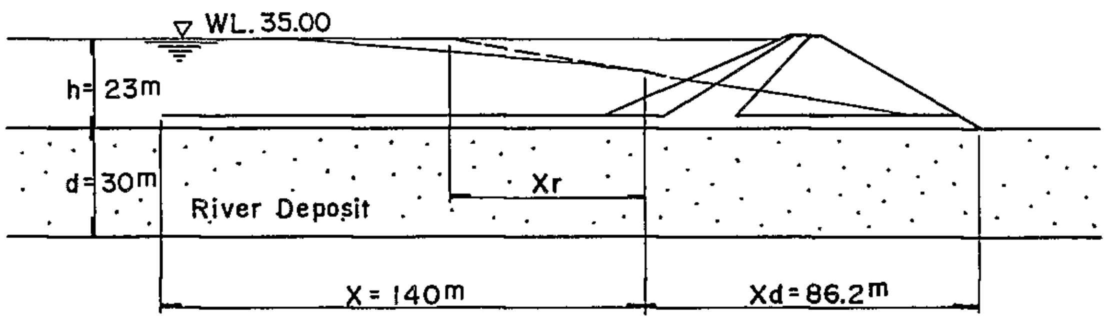
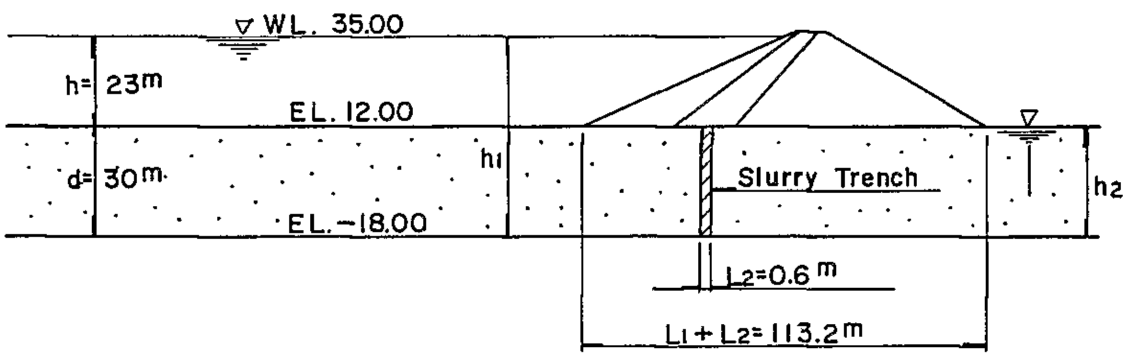


Fig.11.2.3



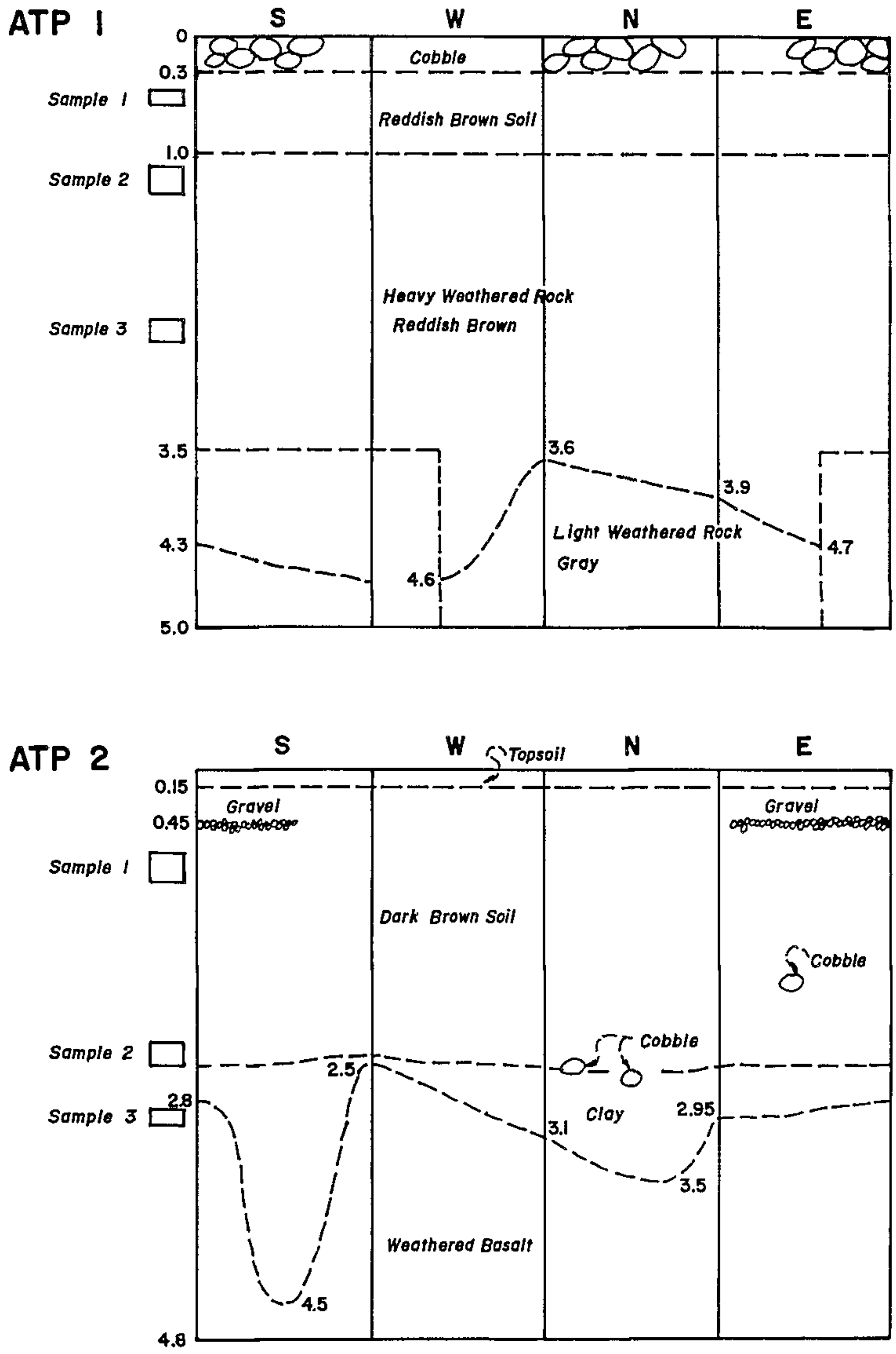


Fig. II.2.2 a LOG CHARTS OF TEST PITS

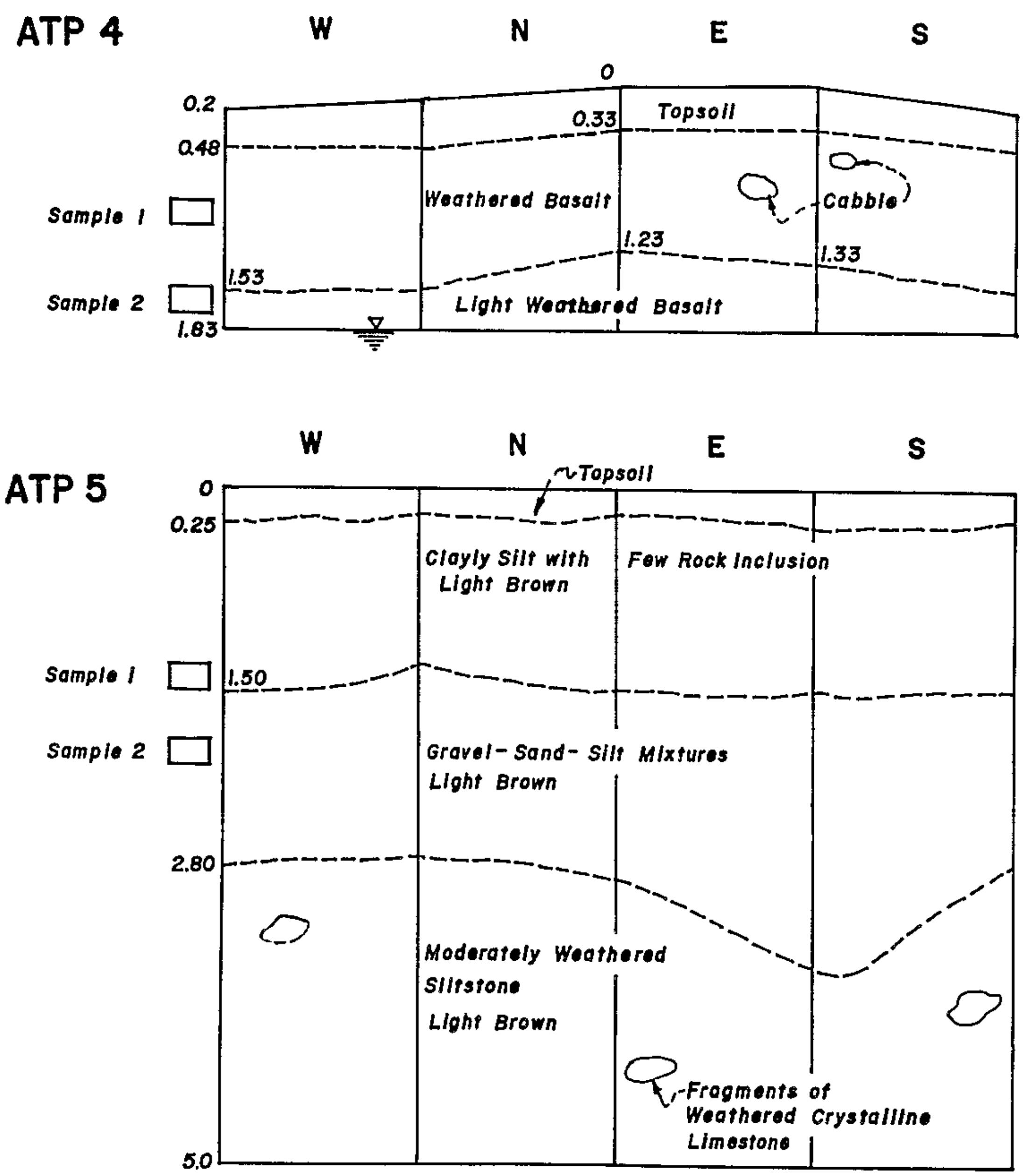


Fig. II.2.2b **LOG CHART OF TEST PITS**

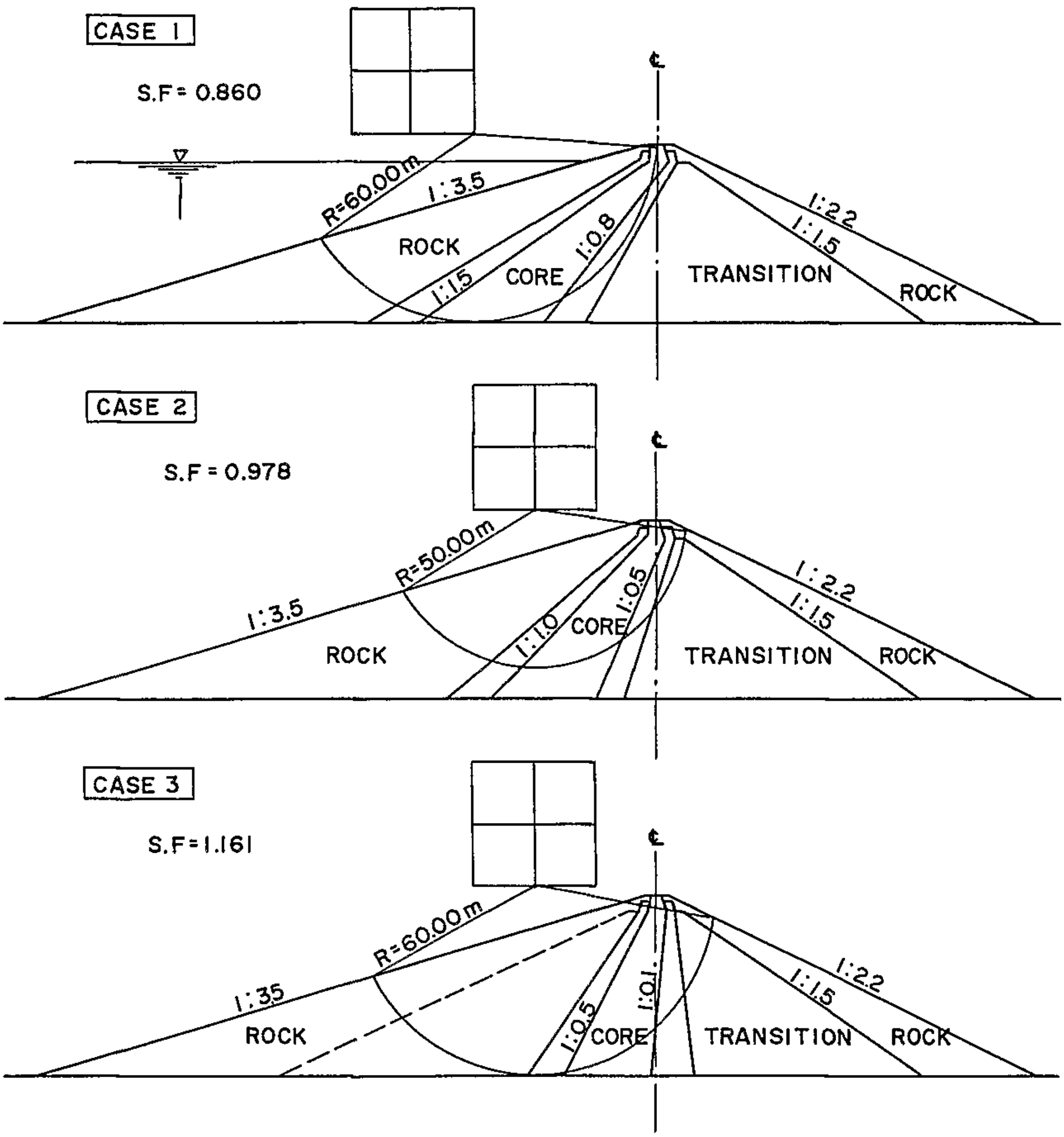
11.3 Dam Type

11.3.1 Stability Analysis of the Inclined Core Type Rockfill Dam

The inclined core type rockfill dam is advantageous from the point of view of ease of construction during the rainy season, because the transition and rock zone embankments of the downstream side are not influenced by the core zone. On the other hand, it has its disadvantages, i.e., the core material is required to have a large shearing resistance and, in addition, the slope of the upstream side becomes more gentle compared with the central core type dam, resulting a larger volume of the dam embankment. Furthermore, there is a risk of the occurrence of cracks in the core zone when the subsidence of ground of the river deposit is developed. Results of the stability analysis, assuming a cohesion of $C = 5t/m^2$ and internal friction angle of 10° as parameters of the core material for the design, are presented in the table below. The slope of the upstream rock is 1:3.5 and the safety factor of the upstream side in case of the normal water surface and a horizontal seismical coefficient of $k = 0.2$.

	Slope of Core (Upstream)	Minimum Safety Factor
Case 1.	1:1.5	0.860
Case 2.	1:1.0	0.978
Case 3.	1:0.5	1.161

Fig.11.3.1 Stability Analysis Inclined Core Type



As can be seen from the considerations above, the slope of core upstream is required to be steeper than 1:0.5 in order to ensure stability. The said condition corresponds to central core type dam, instead of inclined core type dam. Therefore, the inclined core type dam is discarded in the Mabini Dam. The stability analysis diagram is presented in Figure 11.3.1.

11.3.2 Comparison of Dam Types

1) Study of Type-A

The standard cross section of the Type-A obtained as a result of the stability analysis is presented in Figure 11.2.3. The upstream slope of the dam body is 1:3.0, while, at the downstream side, it is 1:2.2. A random zone is provided as counterweight at the temporary cofferdam upstream side. The random zone is aimed at coping with the safety factor of slip circle through foundation. The temporary cofferdam has its center located at 130 m upstreams of the dam axis, in order to make possible the excavation of the cut-off trench.

In view of the considerations presented above, volumes of both excavation and banking are large. The excavation volume is approximately 620,000 m^3 larger and the banking volume is approximately 1.01 million m^3 larger compared with the Type-C and Type-D. The construction costs, excluding the preparation works and temporary works are as follows.

Excavation (1,092,200 m^3)	14,971,800 Pesos
Foundation treatment	11,140,200 Pesos
Embankment (4,118,400 m^3)	73,921,700 Pesos
Total	100,033,700 Pesos
	(12,504,200 US$)

2) Study of Type-B

In the Type-B, the river deposit is left untouched and the impermeabilization is carried out with an underground continuous wall. The standard cross section of the Type-B obtained as a result of the stability analysis is shown in Figure 11.3.4. The use of asphalt lining is planned, aiming at preventing the leakage of water from the junction of the continuous wall and the core. The center of the temporary cofferdam is located at 110 m upstreams of the dam center.

The approximate construction cost for construction of the dam body and foundation treatment, excluding the preparation works and the temporary works is as follows.

Excavation (472,100 m^3)	6,824,200 Pesos
Foundation treatment (continuous wall, asphalt lining and grouting)	52,783,900 Pesos
Embankment (3,107,800 m^3)	55,989,900 Pesos
Total	115,598,000 Pesos
	(14,449,800 US$)

Compared with Type-A, additional drilling of the river deposit for grouting at the section of 350 m ranging from No. 4 through No. 11, is required in this alternative, for the purpose of carrying out the grouting of the bedrock. The total length of this drilling holes is estimated at 28,825 m.

The continuous cut off wall is to be made of concrete, and the cost is estimated by a thickness of 0.80 m. The construction of this continuous cut off wall extends over a section of 250 m ranging from No. 5 to No. 10, and the construction area is 5,800 m^2.

Asphalt lining will be constructed on the core zone bottom, in the section of 250 m ranging from No. 5 through No. 10. This asphalt lining has a thickness of 0.50 m. The construction area is 10,000 m^2.

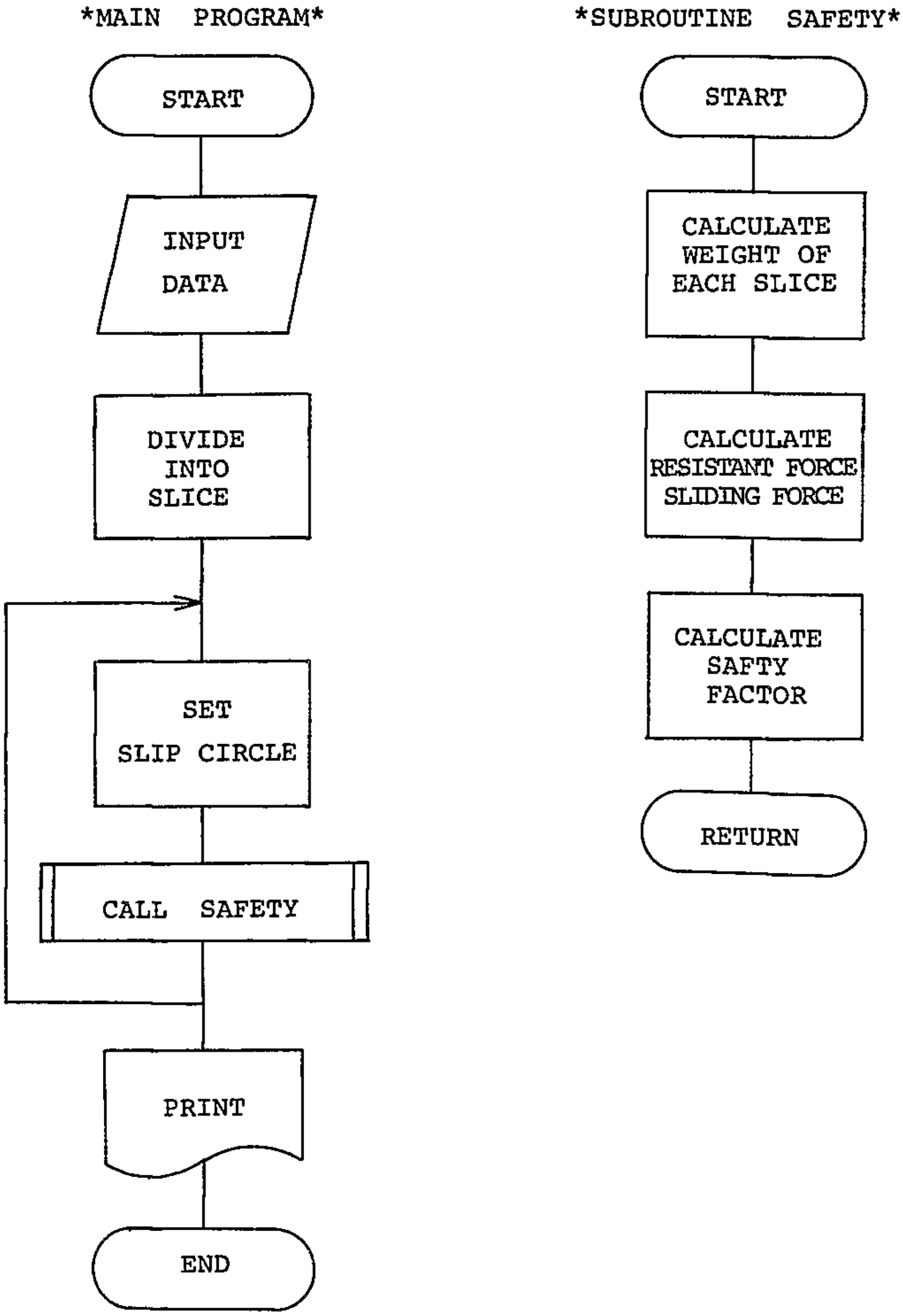
3) Study of Type-C

The dam body cross section of Type-C is similar to that one of dam Type-B. However, with regard to the method of treatment of the river deposit, the cut-off wall is made by Soletanche Grouting instead of a continuous concrete wall. The standard cross section of this case is presented in Figure 11.3.5. Soletanche Grouting is a method developed in Franch which uses a double pipe named Manchette tube to carry out the grouting work. This method is adopted in case of the construction of a cut-off wall at strata like sand and gravel layer, etc., where the use of conventional grouting methods is difficult. So far, this method was adopted in the Aswan Dam, Serre Poncon Dam, etc. The standard value of the spacing in the Soletanche Grouting is 1.5 m and it will be carried out on a section of 250 m, ranging from No. 5 through No. 10. The estimation of the cost is carried out by assuming unit costs 2 times as large as those ones of ordinary grouting methods, because the increases of costs are expected to take place with regard to drilling of sand and gravel layer, purchase of Manchete tube, etc.

The approximate cost for the construction of the dam body and the foundation treatment, exclusing the preparation works and temporary works is as follows.

Cost of excavation work (472,100 m^3)	6,824,200 Pesos
Foundation treatment cost (ordinary grouting, Soletanche grouting)	51,695,300 Pesos
Embankment work cost	55,989,900 Pesos
Total	114,509,400 Pesos
	(14,313,700 US$)

Fig. 11. 3. 2 General Flow Chart



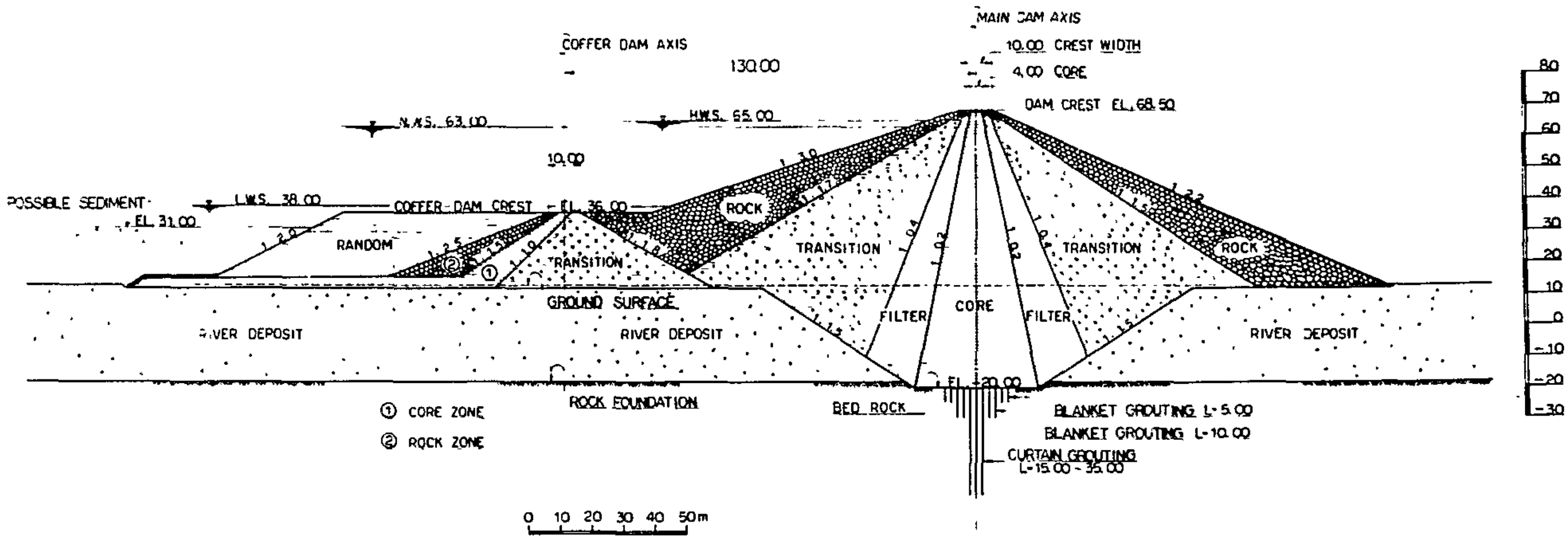


FIG. II.3.3. TYPICAL CROSS SECTION OF DAM (TYPE-A)

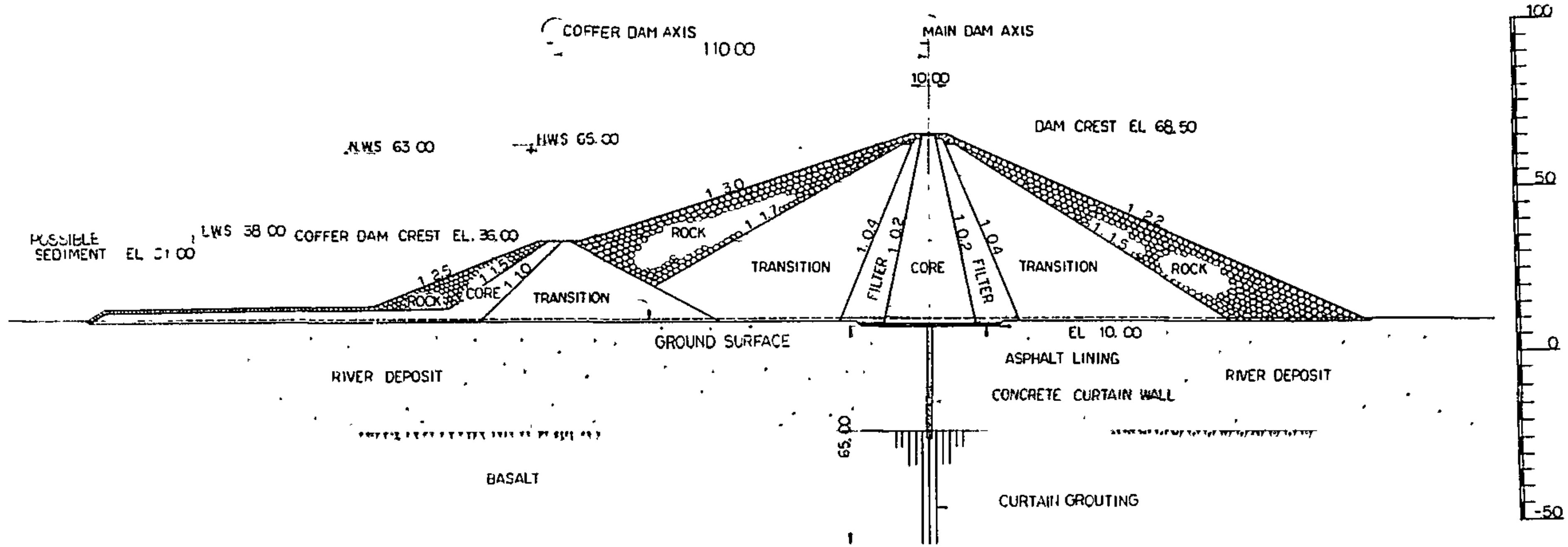


FIG. II.3.4. TYPICAL CROSS SECTION OF DAM (TYPE-B CONCRETE CURTAIN WALL)

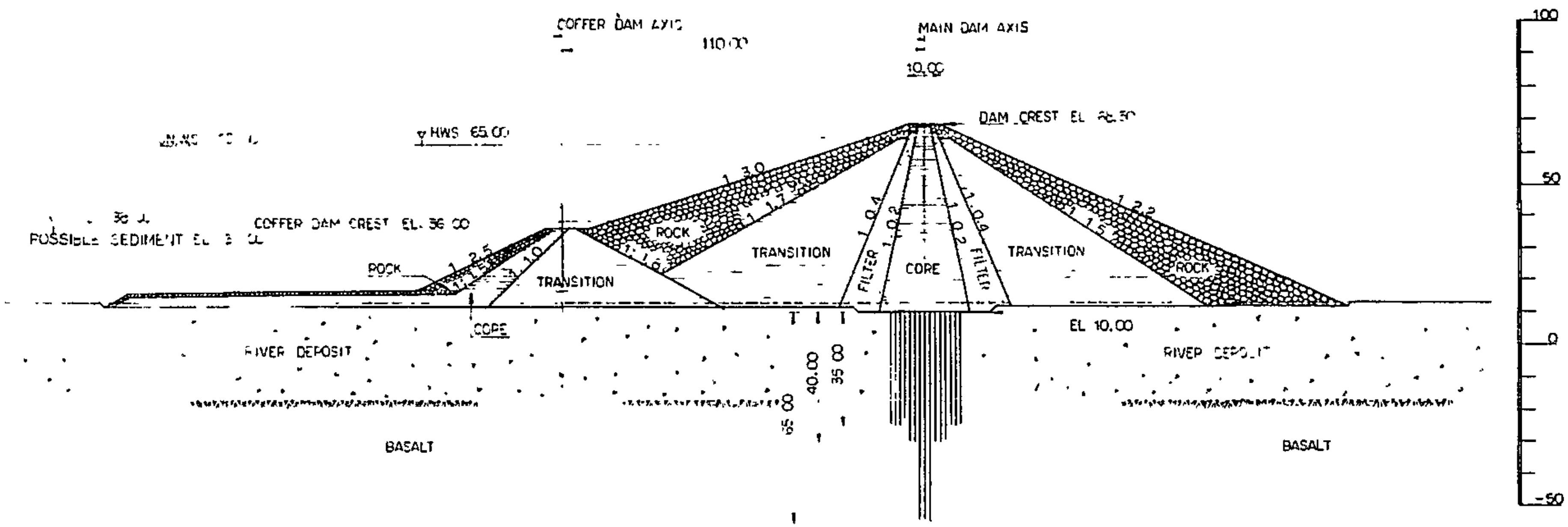


FIG II.3.5. TYPICAL CROSS SECTION OF DAM (TYPE-C SOLETANCHE CURTAIN GROUTING)

11.4 Design of the Dam Body

11.4.1 Analysis of Slipping of Surface Layer of Rock

1) Analysis of the Upstream Side Slope

The safety factor of the surface layer slipping of the upstream side slope in case of normal water level is given by the following formula.

$$F.S = \frac{\left(1 - m \cdot k \cdot \frac{\gamma sat}{\gamma'}\right) \cdot \tan\phi'}{m + \frac{\gamma sat}{\gamma'} \cdot k}$$

where, F.S : Safety factor

F.S	:	Safety factor	
ϕ	:	Angle of internal friction	42°
m	:	Slope 0.333 (1:3.0)	
k	:	Seismic coefficient	0.2
γsat	:	Saturated density	2.15
γ'	:	Submerged density	1.15

$\therefore$ F.S = 1.12

2) Study of the downstream side slope

The safety factor of the downstream slope is calculated with the following formula, because no water is stored therein.

$$F.S = \frac{1 - m \cdot k}{m + k} \tan\phi' = 1.25 > 1.2$$

Note: Downstream slope (1:2.2) $\rightarrow$ m = 0.455

11.4.2 Analysis of the Percolation Current

The flow line net of the percolation current in case of the normal water surface of the Mabini Dam is shown in the main report (11.6). The downstream water level is to be WL = 12,00.

Water leakage from the dam body is calculated with the following formula.

$$Q = \frac{N_f}{N_p} \cdot k \cdot h \cdot L$$

where, Q : Water leakage (m^3/sec)

N_f: Interval of the flow line in parts (8 parts)

N_p: Interval of the equipotential line 5 parts

h : Total head 63- 12 = 51 m

k : Coefficient of permeability $K = 1.10^{-7}$ m/sec

L : Length of the dam body L = 290 m

$\therefore Q = 2.3664 \times 10^{-4} m^3/sec$

CHAPTER 12

SPILLWAY STRUCTURE

12. SPILLWAY STRUCTURE

12.1 Flood Tracing

The flood tracing calculations are carried out for the gated-spillway and for the side-channel-spillway.

Results of the flood tracing calculations are presented in Figure 12.1.1 and Figure 12.1.2.

12.2 Selection of the Type of Spillway

There are two types of spillway which can be taken into consideration, namely, ungated type spillway and gated type spillway which are explained in 12.3 and 12.4, respectively. Comparisons of the quantitative data and the construction costs of each case are presented in Table 12.2.1 and Table 12.2.2.

12.3 Gated-Spillway

12.3.1 Spillway Structure

The gated-spillway is composed of the parts described below. (Refer to Figure 12.3.1)

1) Approach channel

The approach channel is the waterway between the reservoir and gate structure of the spillway. The water flow speed in this channel should not exceed 4 m/s, which does not cause disturbances in the flow.

2) Overflow weir

The overflow weir in this project should be a gated type one, and is designed with perfect overflow in order to improve the inflow capacity.

3) Chute portion

In view of the topographical conditions prevailing in the project area, the chute is designed with a gentle slope (1:20) at the upstream part and a steep slope (1:2.5) at the downstream part. The center line of the chute portion should be straight in plain and it should be designed in such a way to minimize the occurrence of disturbances like cavitation, etc., in the flow of water.

4) Energy disipator

A stilling basin as the energy dissipator is provided at the downstream end of the chute, aiming at dissipating the energy of the high speed flow.

5) Tailrace

The tailrace is provided to connect smoothly between the spillway and the downstream natural river.

12.3.2 Number of Gates

The economic comparison is carried out by assuming designed discharge Q = 3,100 m^3/s, high water surface 65.00 m, and cases with 4 gates, 3 gates and 2 gates. The configuration of the gate corresponding to each case (number of gates) is presented in Figure 12.3.2, while the comparisons of the construction cost and other data are presented in Table 12.3.1 and Table 12.3.2.

12.3.3 Design of the Overflow Section

1) Study of the overflow discharge
(Refer to Figure 12.3.3)

The overflow discharge is given by the following formula.

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

where, Q : Discharge

C : Coefficient of discharge 2.1

L : Effective width of the overflow crest

He: Total water head of the overflow crest

Effective length of the weir (L)

$$L = L' - 2N \cdot Kp \cdot He$$

where, L : Effective weir length

L': Actual length of the weir

N : Number of Pia

kp: Coefficient of contraction of Pia (0.01)

He: Total water head of the weir crest

The free overflow discharge corresponding to an arbitrary water head is presented in Table 12.3.3.

Approach flow speed

$3100/(47 \times 17) = 3.880$ m/s < 4.0 m/s ..... OK

Difference of elevation between overflow weir crest and approach channel bottom

$P/H = 5/12 = 0.416 > 1/5$ ..... OK

2) Type of the overflow weir

The Randolph's standard overflow crest type is adopted here (Refer to Figure 12.3.6) and the various parameters are determined in case of the conditions of $Q = 3{,}100\ m^3/sec$ and $Hd = 12\ m$.

$$X_1 = 0.282 \times Hd \fallingdotseq 3.40$$

$$X_2 = 0.175 \times Hd = 2.10$$

$$r_1 = 0.5 \times Hd = 6.00$$

$$r_2 = 0.2 \times Hd = 2.40$$

$$Y/Hd = 0.50\ (X/Hd)^{1.85}$$

$$Y = 0.5 \times 12\ (X/12)^{1.85} = 0.06049\ X^{1.85}$$

$$Y' = 0.1120 \times X^{0.85}$$

The junction with the downstream slope (1:2.0) is given as follows.

$$0.1120\ X^{0.85} = 1/2 \qquad X = 5.813 \qquad Y = 1.570$$

12.3.4 <u>Design of the Chute Section</u>

1) Hydraulic study at the chute

The calculations are carried out for

$Q_1 = 3100\ m^3/s$

The Manning's Formula is used for the sake of calculation. (Table 12.3.5)

Concrete roughness coefficient	n = 0.015
Energy coefficient	= 1.10

2) Wall height of the chute

The wall height of the chute is determined by means of the following formula, by taking into consideration the air mixed in the water.

Proportion between the quantity of air and the quantity of water

$$m = \frac{-1 + \sqrt{1 + \frac{F_2}{50}}}{2} \qquad F^2 = \frac{V^2}{g \cdot h}$$

Compensated water depth

$d = (1 + m) \times h$

Freebaord $\quad Fb = 0.60 + 0.037 \times V \cdot d^{1/3}$

From the result of the calculation by standard step method for $Q = 3{,}100\ m^3/s$ we have

Point No. 3

$h = 4.125$ m $\quad V = 15.991$ m/s

$F^2 = 6.326$

$m = 0.031$

$d = 4.253$ m

$Fb = 1.559$ m

Point No. 8

$h = 3.450$ m $\quad V = 19.120$ m/s

$F^2 = 10.813$

$m = 0.051$

$d = 3.626$

$Fb = 1.687$ m

12.3.5 Design of the Stilling Basin

1) Selection of type

A stilling basin is provided at the downstream end of the chute, aiming at killing the high energy of the high-speed rapid flow in order to convert it into an ordinary flow.

There are various types of stilling basins, but among them, the sub-dam type, which kills energy by utilizing the jumping action, is safe from the hydraulic point of view, and appropriate to the topographical and geological conditions prevailing in the project area, and is adopted here.

2) Design discharge

The 100 year probability discharge Q = 2,115 m^3/s is adopted as design discharge of the stilling basin. Check of the discharge is carried out at Qmax = 3,100 m^3/s.

3) Inflow specifications of the stilling basin

From the chute Q = 2,115 m^3/s point No. 18 we have

Channel width	B = 47.00 m
Bed height	Z = 7.425 m
Depth of water	$d_1 = h$ = 1.588 m
Velocity	V = 28.338 m/s
Fluid Number	$F = \frac{V}{\sqrt{g \cdot h}} = 7.183$

4) Hydraulic specifications after hydraulic jump

Hydraulic jump depth $d_2 = \frac{d1}{2}(\sqrt{1+8F^2}-1)$=15,357 m

Velocity $V_2 = \frac{Q}{b \cdot d_2}$ = 2.930 m/sec

Velocity head $hv_2 = \frac{V_2}{2g}$ = 0.438 m

Specific energy $E_2 = d_2 + hv_2$ = 15,795 m

Elevation of hydraulic jump WL_2 = 22,782 m

Energy height EH_2 = 23,220 m

The hydraulic specifications of the stilling basin corresponding to a discharge of Q1 = 3,100 m^3/sec, calculated in the same manner as those ones corresponding to Q2 = 2,115 m^3/sec, are presented in Table 12.3.6.

5) Downstream river water level

The river water level is calculated for the discharge of Q1 = 3,100 m^3/s and Q2 = 2,115 m^3/s. Refer to Figure 12.3.8 for details regarding the standard cross section of the river.

Coefficient of roughness n = 0.040

Slope of the river I = 1/1,500

Assuming H = 5,910 m^2	Assuming H = 7,240 m^2
A = 117.315 m^2	A = 1522.688 m^2
P = 249.956 m	P = 272.871 m
R = 4.694 m	R = 5,580 m
V = 1,810 m/s	V = 2,031 m/s
Q ≒ 2,120 m^3/s	Q ≒ 3,100 m^3/s

6) Dimensions of each part

Length of the stilling basin (L)

$L = 6 \times d_2 = 92.50$ m

Height of stilling basin wall (H)

Freeboard $Fb = 0.1 \times (V_1 + d_2) = 4.370$ m

$H = 15.357 + 4.370 = 19.80$ m

Freeboard is given below in case of discharging $Q = 3{,}100\ m^3$.

$Fb = H - d_2 = 0.923$ m

Height of the sub dam (W)

The height of the sub dam is calculated by using the Iwasaki's formula.

$$\frac{W}{d_1} = \frac{(1+2F_1^2)\sqrt{1+8F_1^2} - 1 - 5F_1^2}{1+4F_1^2 - \sqrt{1+8F_1^2}} - \frac{3}{2}F_1^{2/3}$$

where, W : Height of the secondary dam

F_1: Fluid number before hydraulic jump ($V_1/\sqrt{gd_1}$)

d_1: Water depth before hydraulic jump

V_1: Velocity before hydraulic jump

$\therefore W = 6.720$ m

Configuration of the sub dam crest

The Randolph's standard overflow crest configuration is adopted (Refer to Figure 12.3.7) and the various parameters are determined by assuming the conditions of $Q = 2{,}115\ m^3/s$ and $Hd = 8.6$ m.

$X_1 = 0.282 \times Hd = 2.40$

$X_2 = 0.175 \times Hd = 1.50$

$r_1 = 0.5 \times Hd = 4.30$

$r_2 = 0.2 \times Hd = 1.70$

$Y/Hd = 0.50 \ (X/Hd)^{1.85}$

$Y = 0.08029 \ X^{1.85}$

$Y' = 0.1485 \ X^{0.85}$

The junction with the downstream slope (1:2.0) is given as follows.

$0.1485 \ X^{0.85} = 1/2$

$X = 4.171 \qquad Y = 1.127$

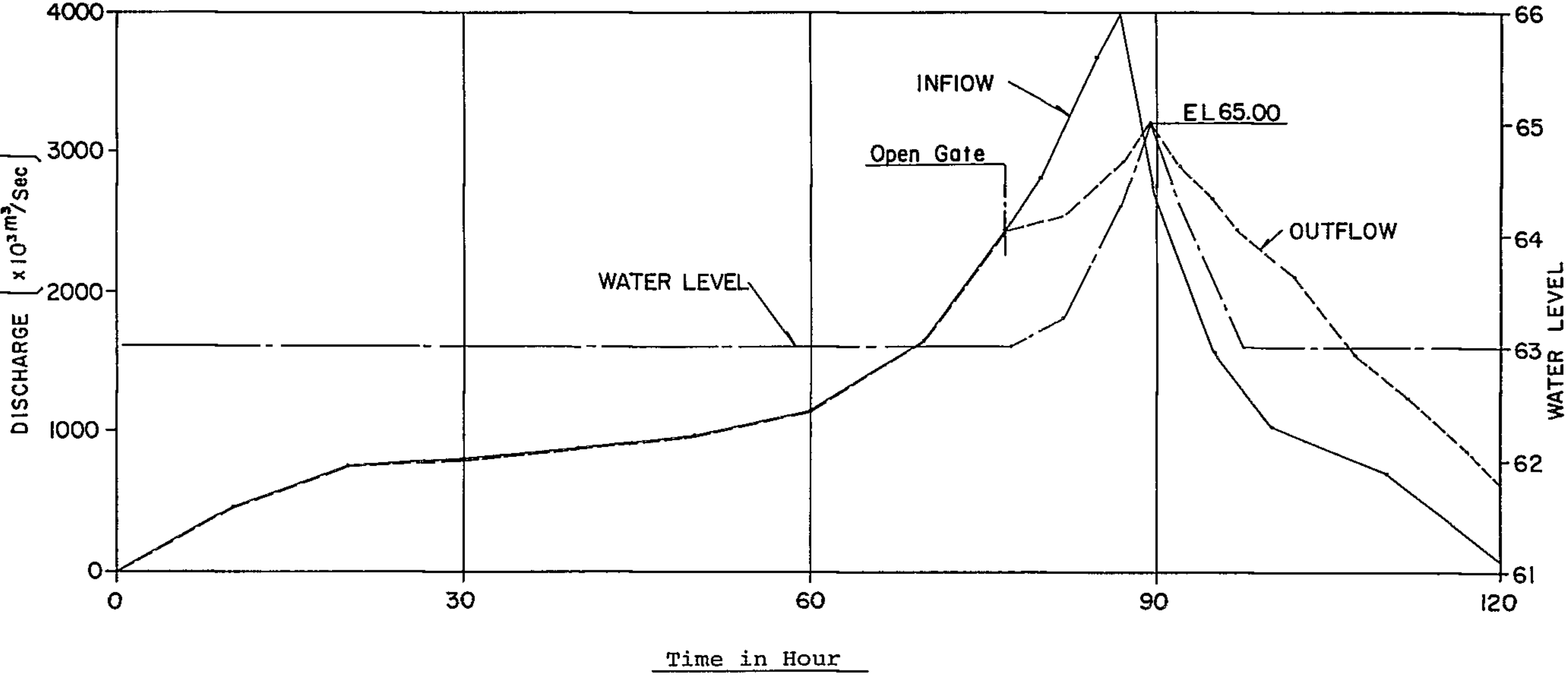


Fig. 12.1.1 Flood Routing for Gated Spillway

Fig.12.1.2 Flood Routing for Side-channel Spillway

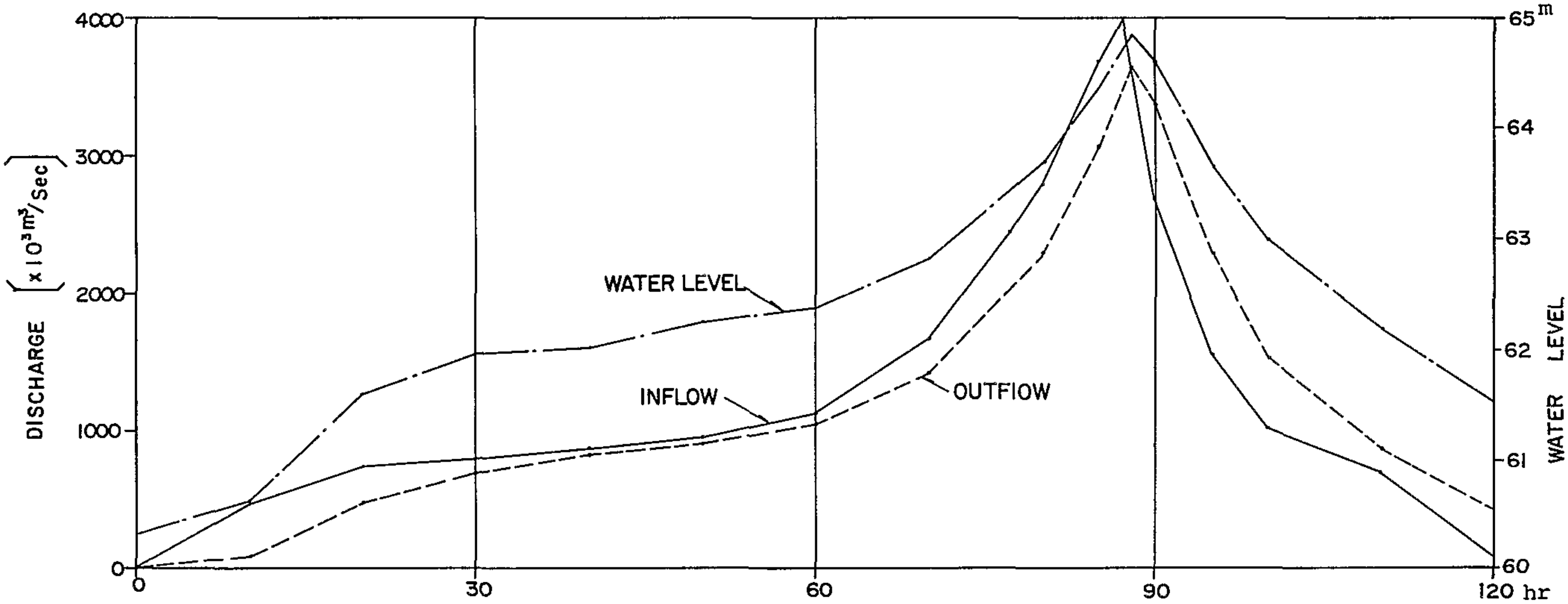


Table 12-2-1 Comparison of Construction Cost

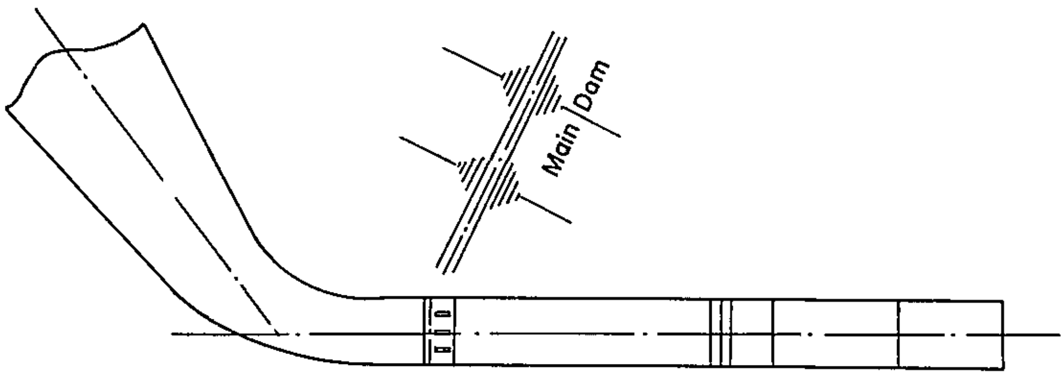
Item	Unit	Unit Cost	Comparison of Spillway type			
			Gated-Spillway (4-Gate)		Side Channel-Spillway	
			Quantity	Amount	Quantity	Amount
Excavation				$x10^3$ ₱		$x10^3$ ₱
Sand and Gravel	m^3	12.6	910,000	11,466	860,000	10,836
Rock	m^3	18.9	2,120,000	40,068	2,010,000	37,989
Reinforced Concrete	m^3	615	76,680	47,158	179,500	110,392
Reinforcement Bars	t	6,500	1,530	9,945	3,590	23,335
Gate	t	63,000	350	22,050	-	-
Total				130,687		182,552

Table 12-2-2 Comparison of Quantity

Item	4 - Gated - Spillway		Side Channel - Spillway	
Reinforced Concrete	Approach Channel	13,990 m^3	Side Channel	96,960 m^3
	Control Structure	20,300	Transition	32,730
	Chute	16,860	Chute	31,390
	Stilling Basin	15,070	Stilling Basin	24,100
	Apron	10,460	Apron	5,920
	Total	76,680	Total	191,100
Gate	Steel Roller Gate B-9.5 x H-10.5	4 LS		
Excavation	Sand and Gravel	910,000 m^3	Sand and Gravel	860,000 m^3
	Rock	2,120,000	Rock	2,010,000

Fig.12.3.1 Gated Spillway Illustration

Plan



Profile

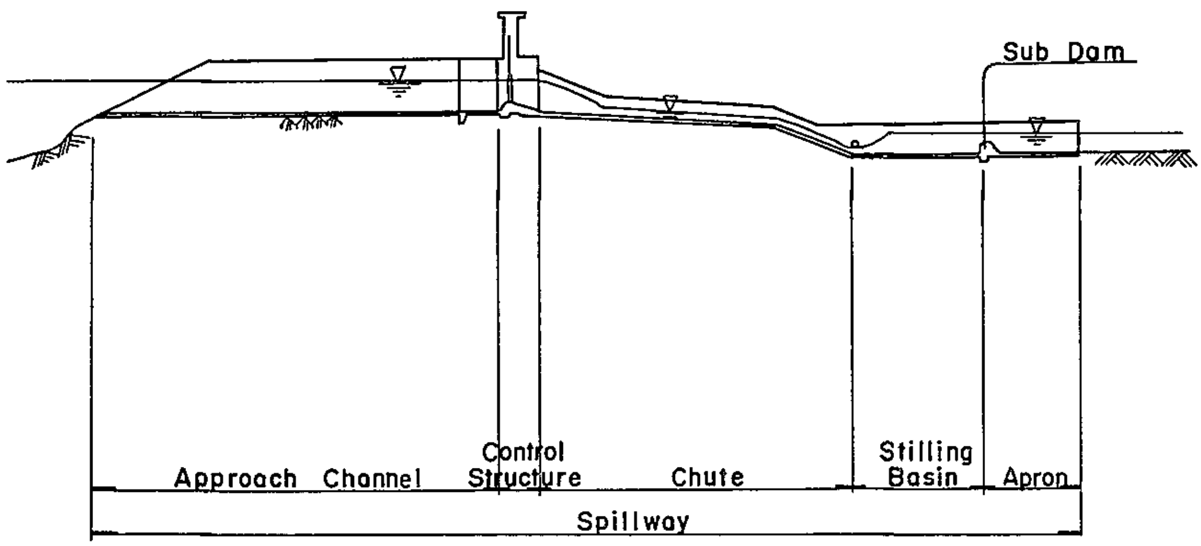
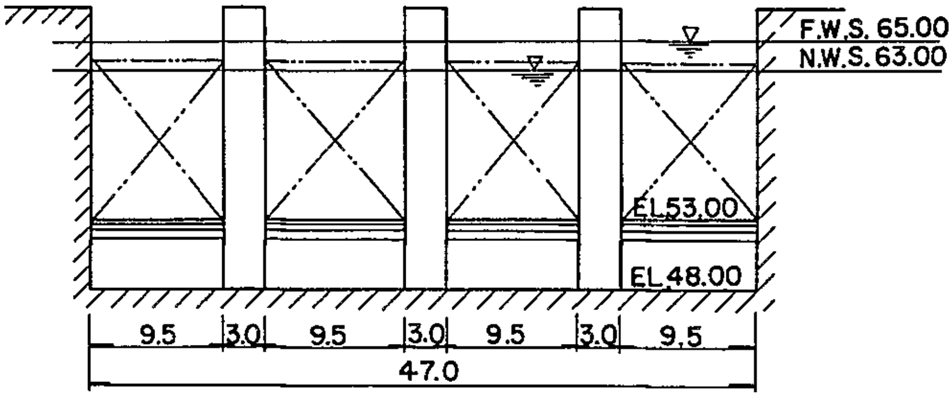
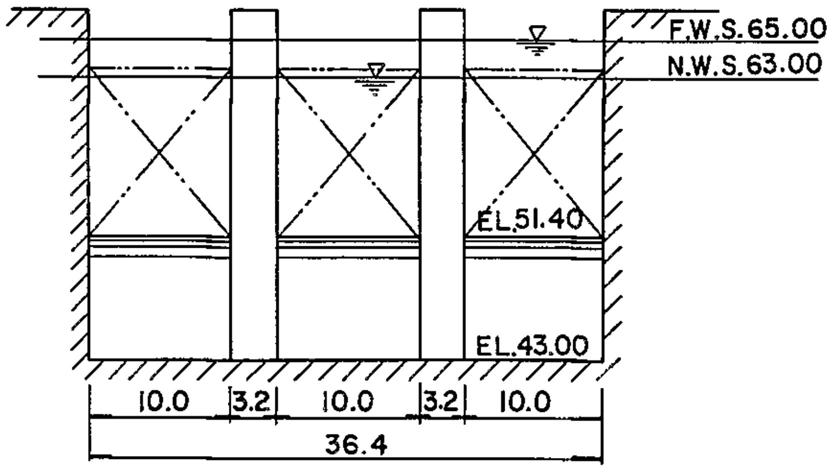


Fig. 12.3.2 Gate Section

4 - Gate



3 - Gate



2 - Gate

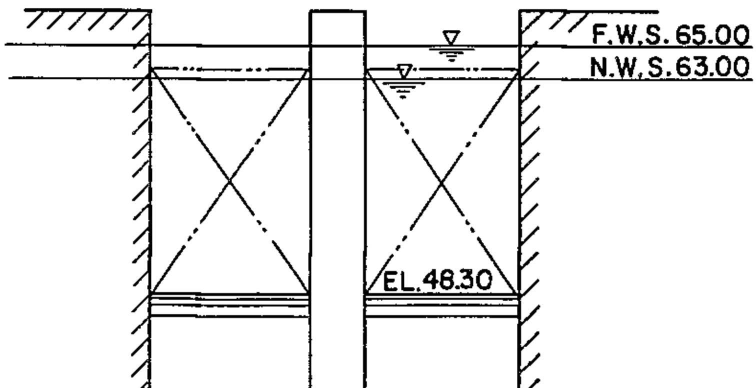
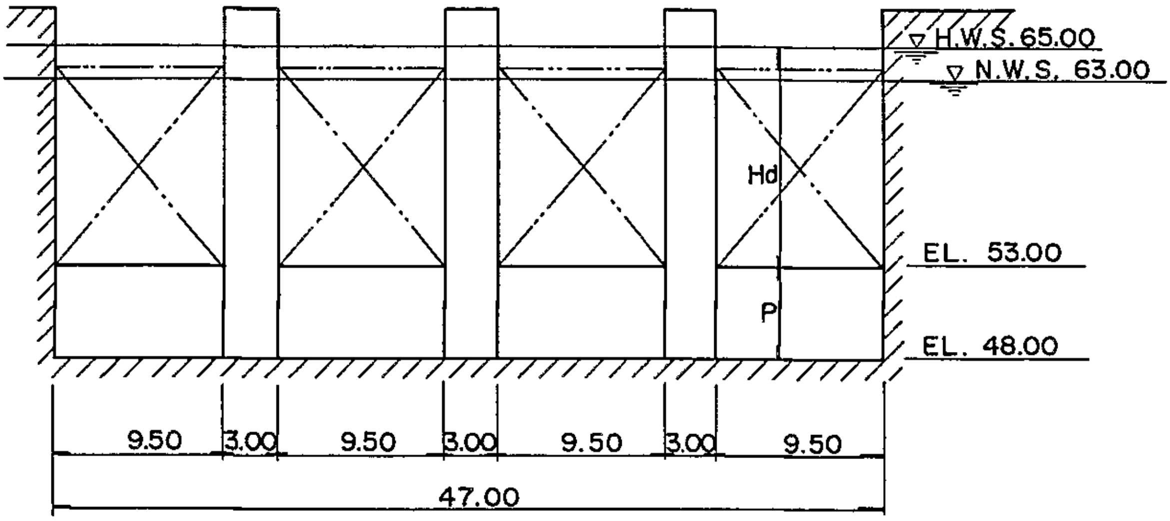


Fig. 12.3.3 Gate Standard Section



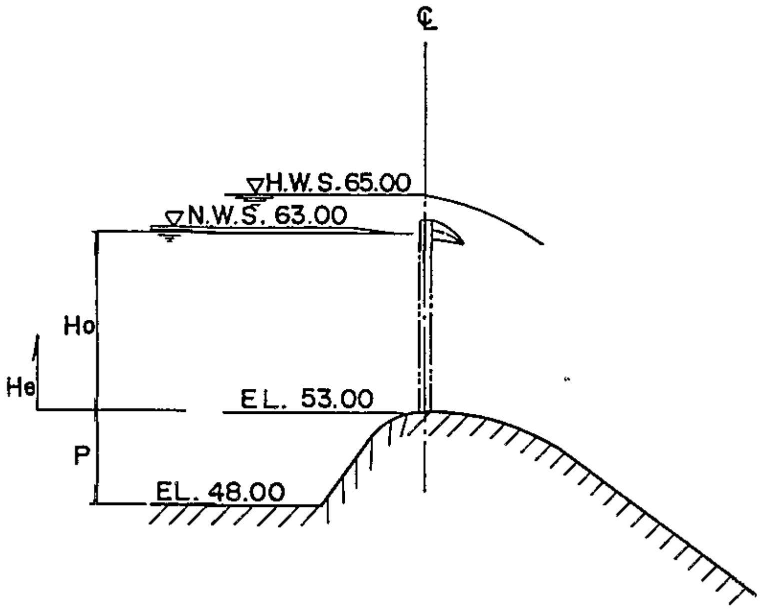


Fig. 12.3.4 Coefficient of Discharge for Other than the Design Head

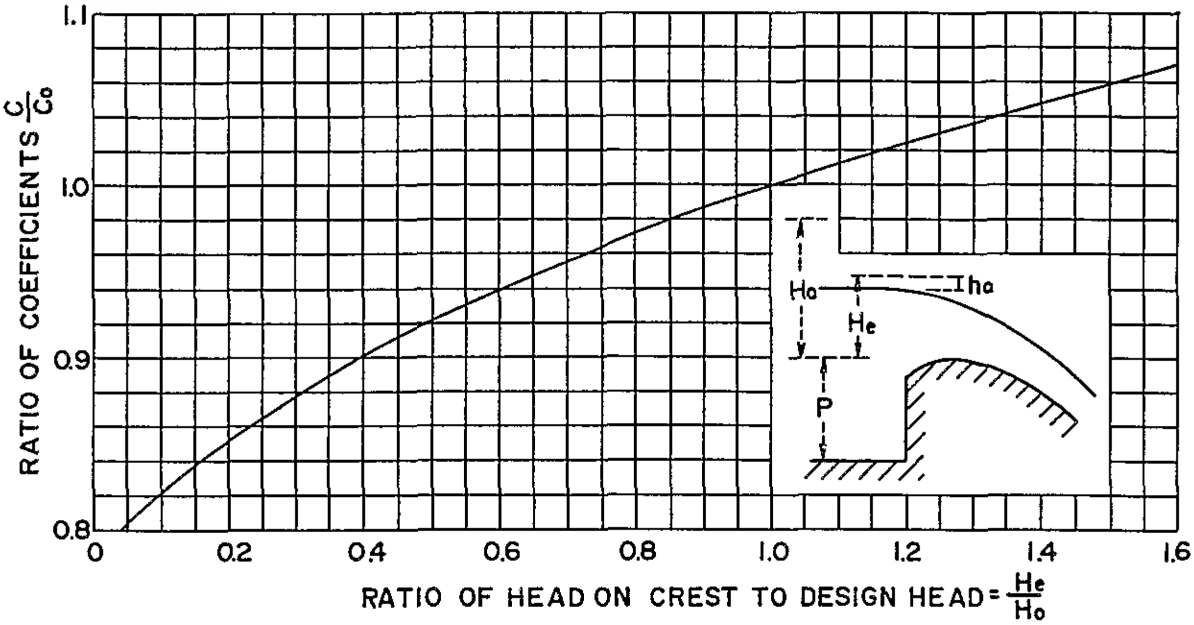


Fig. 12.3.5 Spillway Design Flood Hydrograph

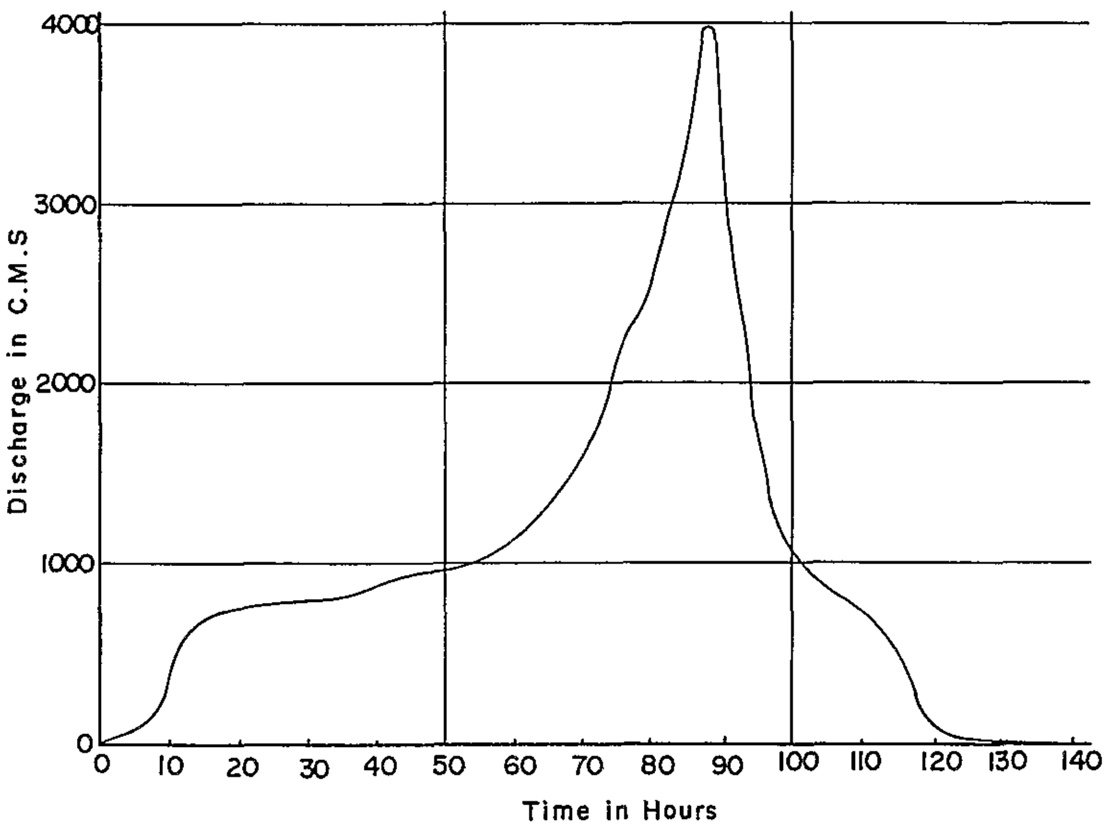


Fig. 12.3.6 Shape for Ogee Crest (Main Dam)

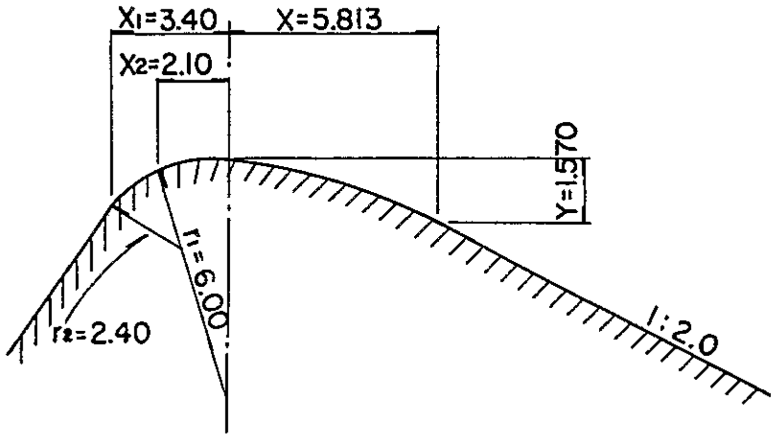


Fig.12.3.7 Shape for Ogee Crest (Sub Dam)

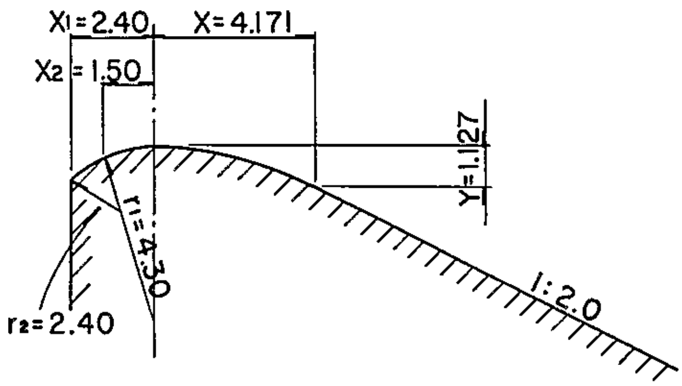


Fig.12.3.8 Typical Cross Section of River

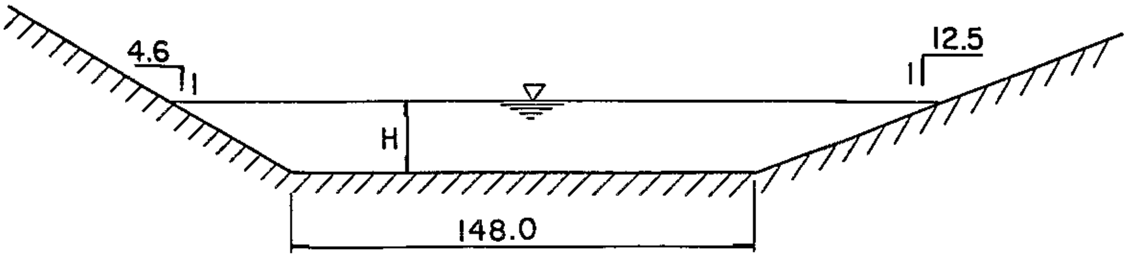


Fig.12.3.9 Standard Section of Stilling Basin

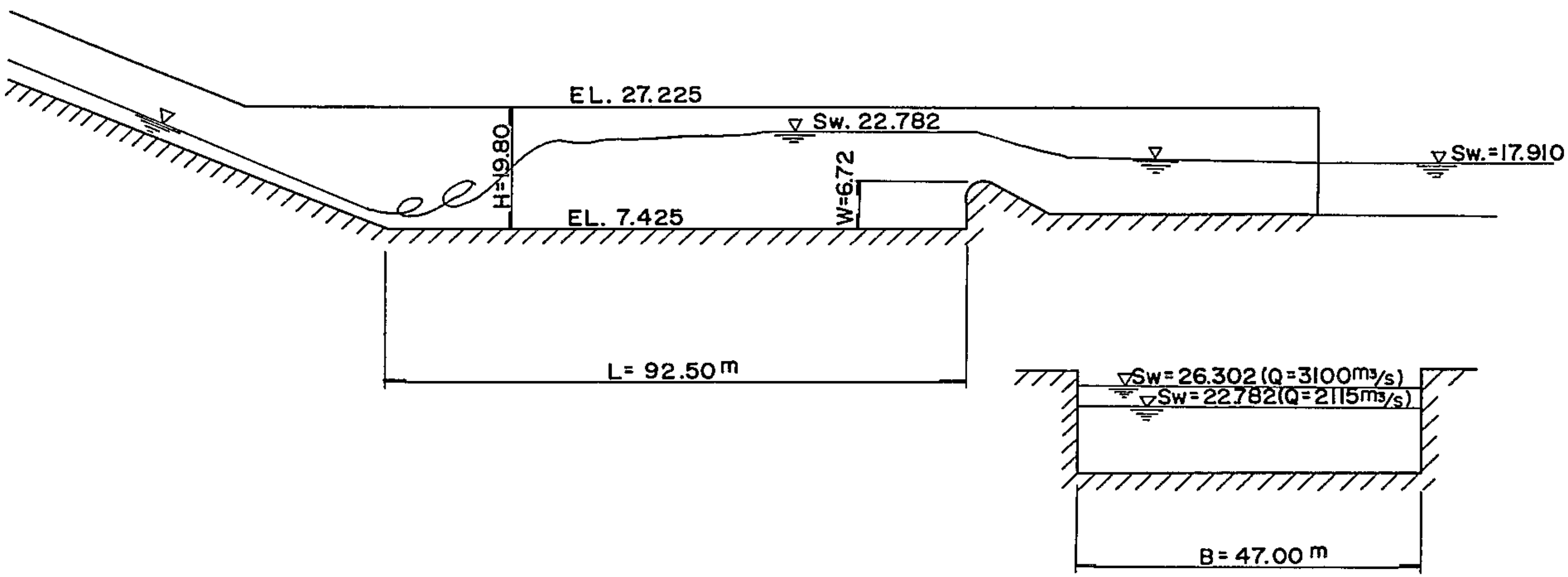


Table 12-3-1 Comparison of Construction Cost

Item	Unit	Unit Cost	Comparison of Gate Number					
			2 - Gate		3 - Gate		4 - Gate	
			Quantity	Amount	Quantity	Amount	Quantity	Amount
Excavation		₱		$x10^3$ ₱		$x10^3$ ₱		$x10^3$ ₱
Sand	m^3	12.6	960,000	12,096	920,000	11,591	910,000	11,466
Rock	m^3	18.9	2,230,000	42,147	2,130,000	40,257	2,120,000	40,068
Reinforced Concrete	m^3	615	120,870	74,335	90,250	55,503	76,680	47,158
Reinforcement Bars	t	6,500	2,413	15,710	1,850	11,732	1,530	9,945
Gate	t	63,000	270	17,010	330	20,790	350	22,050
Total				161,298		139,873		130,687

Table 12-3-2 Comparison of Quantity

Item		2-Gate	3-Gate	4-Gate
Reinforced Concrete	Aproach Channel	20,300 m^3	16,420 m^3	13,990 m^3
	Control Structure	36,200	23,680	20,300
	Chute	24,230	18,610	16,860
	Stilling Basin	24,540	18,010	15,070
	Apron	15,600	13,530	10,460
	Total	120,870	90,250	76,680
Gate	Steel Roller Gate	B-11.0 x H-15.2 2 LS	B-10.0 x H-12.1 3 LS	B-9.5 x H-10.5 4 LS
Excavation	Sand	960,000 m^3	920,000 m^3	910,000 m^3
	Rock	2,230,000	2,130,000	2,120,000

Table 12-3-3 Discharge Over an Uncontrolled Overflow Ogee Crest

Overflow Depth (He)	Dam Water Level (EL)	He/Ho	C/Co	Coefficient of Discharge (C)	Effective Weir Length (L')	Q $=C \cdot L' \cdot He^{3/2}$	V	Hv $0.35 V^2/2G$	Stored Water Level
8.5 m	61.5 m	0.726	0.960	2.016	37.490 m	1872.989 m^3/s	3.008 m/s	0.162 m	61.7 m
9.5	62.5	0.812	0.975	2.048	37.430	2244.575	3.249	0.188	62.7
10.0	63.0	0.854	0.980	2.058	37.400	2433.974	3.407	0.207	63.2
10.5	63.5	0.897	0.988	2.075	37.370	2638.307	3.564	0.227	63.7
11.5	64.5	0.983	0.997	2.094	37.310	3046.830	3.859	0.266	64.8
11.7	64.7	1.000	1.000	2.100	37.298	3134.604	3.923	0.275	65.0
12.5	65.5	1.068	1.008	2.116	37.250	3483.431	4.164	0.310	65.8
13.5	66.5	1.154	1.020	2.142	37.190	3951.360	4.458	0.350	66.9

Ho: Design head over ogee crest

Table 12-3-4 FLOOD ROUTING

TIME (HR)	INFLOW (M**3/S)	OUTFLOW (M**3/S)	WATER LEVEL (M)	STORAGE CAPACITY (M**3)
0.00	2400.000	2433.974	63.00	302999810.
1.00	2500.000	2434.996	63.00	303055620.
2.00	2600.000	2444.593	63.03	303452160.
3.00	2800.000	2465.757	63.08	304333310.
4.00	2920.000	2498.526	63.16	305693440.
5.00	3080.000	2540.069	63.26	307423740.
6.00	3240.000	2591.427	63.39	309562880.
7.00	3420.000	2652.632	63.54	312111360.
8.00	3660.000	2726.356	63.72	315173120.
9.00	3990.000	2817.410	63.94	318964220.
10.00	3990.000	2942.991	64.25	322959360.
11.00	3720.000	3048.470	64.50 DAM Ws	326052610.
12.00	3240.000	3101.776	64.63 65.00	327510020.
13.00	2670.000	3084.298	64.58	327012860.
14.00	2480.000	3022.949	64.44	325289730.
15.00	2240.000	2946.482	64.25	323040510.
16.00	2000.000	2850.878	64.02	320237060.
17.00	1720.000	2766.425	63.81	316821760.
18.00	1560.000	2673.046	63.58	312934660.
19.00	1320.000	2570.799	63.33	308679680.
20.00	1220.000	2462.908	63.07	304190980.
21.00	1140.000	2340.823	62.75	299792130.
22.00	1100.000	2219.604	62.43	295615230.
23.00	1040.000	2107.073	62.13	291678980.
24.00	1000.000	2011.171	61.87	287938050.
25.00	940.000	1927.573	61.65	284340220.
26.00	920.000	1847.502	61.43	280892930.
27.00	880.000	1771.451	61.23	277618690.
28.00	860.000	1699.097	61.03	274503680.
29.00	830.000	1622.802	60.83	271566080.
30.00	800.000	1549.126	60.63	268790530.
31.00	760.000	1478.978	60.44	266147940.
32.00	740.000	1412.486	60.26	263643300.
33.00	720.000	1350.417	60.09	261298060.
34.00	680.000	1289.272	59.93	259066610.
35.00	630.000	1227.203	59.76	256894940.
36.00	560.000	1165.333	59.59	254730370.
37.00	520.000	1104.126	59.43	252589330.
38.00	440.000	1043.049	59.27	250452400.
39.00	340.000	979.149	59.09	248216430.
40.00	240.000	919.109	58.93	245843550.
41.00	160.000	864.633	58.79	243352820.
42.00	120.000	809.770	58.64	240842880.
43.00	90.000	756.404	58.49	238401760.
44.00	70.000	705.182	58.36	236058900.
45.00	50.000	656.318	58.22	233824190.
46.00	45.000	610.210	58.10	231715440.
47.00	40.000	567.209	57.99	229749070.

Table 12-3-5 CHUTE HYDRAULIC COMPUTATION

B =	47.000	1:T, T= 0	
Q =	3100.000	TL=	247.500
Z1=	53.000	Z2 =	7.425
N =	0.015	ALFA=	1.100

Uniform Flow				Critical		
Q(M*3/S)	H(M)	A(M*2)	V(M/S)	HC(M)	AC(M*2)	VC(M/S)
3100.000	1.249	58.697	52.814	7.875	370.109	8.376

	Distance		Water Elevation	Depth	Area	Velocity	Velocity Head	Water Elevation	Froude Number	
NO	L(M)	TL(M)	Z(M)	H(M)	A(M*2)	V(M/S)	HV(M)	H+Z(M)	F	B
0	0.000	0.000	53.000	7.875	370.109	8.376	3.937	60.875	0.953	47.000
1	4.500	4.500	50.750	5.276	247.967	12.502	8.772	56.026	1.739	47.000
2	4.500	9.000	48.500	4.568	214.703	14.439	11.700	53.068	2.158	47.000
3	4.500	13.500	46.250	4.125	193.853	15.991	14.352	50.375	2.515	47.000
4	31.300	44.800	44.685	3.939	185.116	16.746	15.739	48.624	2.695	47.000
5	31.300	76.100	43.120	3.785	177.905	17.425	17.041	46.905	2.861	47.000
6	31.300	107.400	41.555	3.656	171.828	18.041	18.267	45.211	3.014	47.000
7	31.300	138.700	39.990	3.545	166.632	18.604	19.424	43.535	3.156	47.000
8	31.300	170.000	38.425	3.450	162.137	19.120	20.516	41.875	3.288	47.000
9	7.750	177.750	35.325	3.210	150.885	20.545	23.690	38.535	3.663	47.000
10	7.750	185.500	32.225	3.020	141.922	21.843	26.777	35.245	4.015	47.000
11	7.750	193.250	29.125	2.863	134.555	23.039	29.789	31.988	4.350	47.000
12	7.750	201.000	26.025	2.731	128.360	24.151	32.734	28.756	4.668	47.000
13	7.750	208.750	22.925	2.618	123.057	25.192	35.616	25.543	4.973	47.000
14	7.750	216.500	19.825	2.520	118.454	26.170	38.438	22.345	5.266	47.000
15	7.750	224.250	16.725	2.434	114.412	27.095	41.202	19.159	5.547	47.000
16	7.750	232.000	13.625	2.358	110.830	27.971	43.908	15.983	5.818	47.000
17	7.750	239.750	10.525	2.290	107.630	28.802	46.558	12.815	6.080	47.000
18	7.750	247.500	7.425	2.229	104.751	29.594	49.152	9.654	6.332	47.000

Table 12-3-6 Hydraulic Specification of the Stilling Basin

Item \ Discharge		$3,100 m^3/s$	$2,115 m^3/s$
Channel Width	b m	47.00	47.00
Elevation of Stilling Basin	z m	7.425	7.425
Inflow Depth	d_1 m	2.229	1.588
Inflow Velocity	v_1 m/s	29.594	28.338
Fluid Number	F	6.332	7.183
Hydraulic Jump Depth	d_2 m	18.877	15.357
Velocity	v_2 m/s	3.494	2.930
Velocity Head	hv_2 m	0.623	0.438
Specific Energy	E_2 m	19.500	15.795
Hydraulic Jump	WL_2 m	26.302	22.782
Energy	Eh_2 m	26.925	23.220

12.4 Side Channel Spillway

12.4.1 Construction of the Spillway

The side channel spillway is composed of the parts presented in Figure 12.4.1.

12.4.2 Design of the Overflow Section

1) Study of the overflow discharge (Refer to Figure 12.4.2 and Figure 12.4.3)

The overflow discharge is given by the following expression.

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

where, Q : Discharge

C : Coefficient of discharge

L : Effective width of the overflow crest

He: Total water head of the overflow crest

The values of the free overflow discharges corresponding to an arbitrary water head are given in Table 12.4.1.

Approaching velocity

$3,630/(180 \times 7) = 2.857$ m/s < 4.0 m/s ..... OK

Difference of height between overflow crest and approaching bed height.

$P/H = 2.3/4.7 = 0.489 > 1/5$ ..... OK

12.4.3 Design of the Side Channel

1) Profile

The side channel is designed with trapezoidal cross section and slopes of 1:0.7 at the reservoir side and upright at the mountain side.

2) Width of the bottom at the extremity of the side channel (b_2)

In this area the base of the side channel has the same width b_2 = 35.00 m as the gentle slope channel, in view of the approach to the downstream side.

3) Bottom width at the upstream extremity (b_1)

The bottom width at the upstream side is 1/2 of the bottom width at the extremity, i.e., b_1 = 17.50 m.

4) Bottom slope (I)

The bottom slope is made I = 1/20, because it is required to be more gentle than 1/13.

5) Calculation by the standard step method

The calculation is carried out with discharges of Q = 3,630 m^3/s and 2,115 m^3/s, based upon the conditions mentioned above.

6) Hydraulic analysis at the extremity

The analysis is carried out aiming at confirming that the fluid coefficient does not exceed F = 0.50 when Q = 3,630 m^3/s.

The hydraulic specifications are as follows. (Refer to Table 12.4.3)

Water depth	$d = 17.700$ m
Area of cross section	$A = 729.151$ m^2
Velocity	$v = 4.978$ m/s
Width of water surface	$T = 47.390$ m
Hydraulic water depth $h = A/T$	$= 16.805$ m

$$\text{Fluid coefficient } F = \frac{V}{\sqrt{g \cdot h}} = 0.388 < 0.50 \text{ ..... OK}$$

7) Wall height

At the upstream side of the dam axis, the dam body non-overflow elevation EL65.00m is made identical as the wall crest.

8) Gentle slope channel

The bottom slope is determined in such a way to obtain a fluid coefficient not exceeding approximately $F = 0.45$ when the discharge is $Q = 3{,}630$ m^3/s, in order to comprise the dam axis crossing and the plan curve and to obtain a stable sub-critical flow.

$Q = 3{,}630$ m^3/s $\quad$ $b = 35.0$ m $\quad$ $n = 0.015$

The uniform flow specifications corresponding to a slope $I = 1/2{,}400$ are as follows.

$d = 17.700$ m

$A = 619.5$ m^2

$R = 8.800$ m

$V = 5.800$ m/s

$hv = 1.716$ m

Fluid number $F = \frac{V}{\sqrt{gh}} = 0.440 < 0.45$ ..... OK

Accordingly, the slope is made I = 1/2,400, as assumed initially.

9) Wall height of the gentle slope channel

Open channel

In this case the wall height is the margin regarding the uniform flow of Q = 3,630 m^3/s, b = 35.00 m and I = 1/2,400

d = 17.70 m v = 5.80 m/s hv = 1.716 m

$Fb = 0.07d + hv + (0.05 \sim 0.15) = 3.005$ m

The wall height should exceed the following value

$H = d + Fb = 20.705$ m.

12.4.4 Design of the chute

1) Calculation by standard step method

The calculation is carried out for two cases, namely, Q_1 = 3,630 m^3/s and Q_2 = 2,115 m^3/s.

The calculation is carried out with the Manning's formula.

Concrete roughness coefficient	n = 0.015
Energy coefficient	α = 1.10

2) Wall height of the chute

The wall height of the chute is determined by means of the following formula, taking into consideration the air mixed in the water.

Proportion between the quantity of air and the quantity of water

$$m = \frac{-1 + \sqrt{1+\frac{F^2}{50}}}{2} \qquad F^2 = \frac{V^2}{g \cdot h}$$

Compensated water depth

$d = (1 + m) \times h$

Freeboard $\qquad Fb = 0.60 + 0.037 \times V \cdot d^{1/3}$

From the result of the calculation by standard step method for $Q = 3,630\ m^3/s$ we have

Point No. 3

$h = 7.154$ m $\qquad V = 14.497$ m/s

$F^2 = 2.998$

$m = 0.015$

$d = 7.261$

$Fb = 1.639$ m

Point No. 8

$h = 5.491$ m $\qquad V = 18.889$ m/s

$F^2 = 6.630$

$m = 0.032$

$d = 5.667$

$Fb = 1.846$ m

12.4.5 Design of the Energy Dissipator

1) Determination of the type of energy dissipator

An energy dissipator is provided at the downstream extremity of the chute, aiming at killing the high energy of the high-speed flow and converring it into a sub-critical flow.

There are various types of energy dissipator, namely, ski-jump type, sub dam type, etc. The sub dam type stilling basin, which kills energy by utilizing the hydraulic jump effect is adopted in this case, because it is safe from the hydraulical point of view and in addition it is appropriate for the topographical conditions prevailing in the damsite.

2) Design discharge

The 100 year probability discharge Q = 2,115 m^3/s is adopted as design discharge of the energy dissipator. In addition, it is checked with the discharge of Qmax = 3,630 m^3/s.

3) Specifications of the beginning point of stilling basin

From the calculation by standard step method of chute, (Q = 2,115 m^3/s point No. 20) we have the following specifications.

Channel width	B = 35.00 m
Bed height	Z = 6.550 m
Depth of water	$d1 = h = 2.273$ m
Velocity	V = 26.588 m/s
Fluid number	$F = \frac{V}{\sqrt{gh}} = 5.633$

4) Hydraulic specifications after hydraulic jump

Hydraulic jump depth	$d_2 = \frac{d1}{2}(\sqrt{1+8F^2}-1) = 17.006$ m
Velocity	$v_2 = \frac{Q}{b \cdot d_2} = 3.553$ m/s
Velocity head	$hv_2 = \frac{V_2^2}{2g} = 0.644$ m

Specific energy	$E2 = d_2 + hv_2$	= 17.650 m
Elevation of hydraulic jump	$wL2 = Z_2 + d_2$	= 23.956 m
Energy height	$EH_2 = Z_2 + E_2$	= 24.600 m

The results of calculation for Q = 3,100 m^3/s, hydraulic specifications of the stilling basin for Q = 2,100 m^3/s, are calculated in the same way. (Refer to Table 12.4.6)

5) Dimensions of each part

Length of the stilling basin (L)

$L = b \times d_2 = 102.0$ m

Wall height (H) of the stilling basin

Freeboard $Fb = 0.1 \times (V_1 + d_2) = 4.359$

H = 17.006 + 4.359 = 21.50 m

However, the wall height is made 23.00 m, because the hydraulic jumping depth is 22.612 m in case of Q = 3,630 m^3/s.

Height of the sub-dam

The height of the sub-dam is calculated by using the Iwasaki's Formula.

$$W/d_1 = \frac{W}{d_1} = \frac{(1+2F_1^2)\sqrt{1+8F_1^2} - 1 - 5F_1^2}{1+4F_1^2 - \sqrt{1+8F_1^2}} - \frac{3}{2}F_1^{2/3}$$

where, W : Height of the sub-dam

F : Fluid number before hydraulic jump ($V_1/\sqrt{gd_1}$)

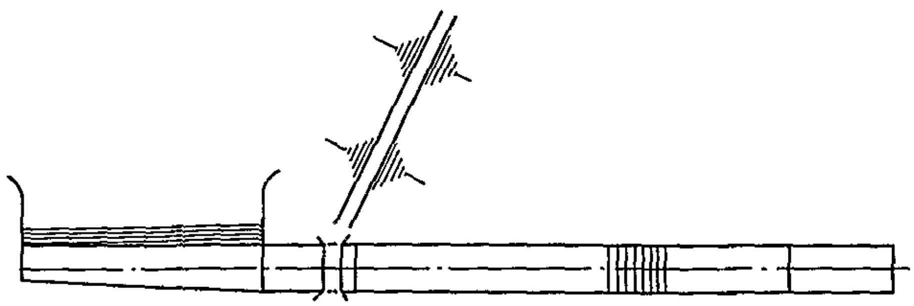
d : Water depth before hydraulic jump

V : Velocity before hydraulic jump

Therefore, W = 6.859 m.

Fig.12.4.1 Side Channel Spillway Illustration

Plan



Profile

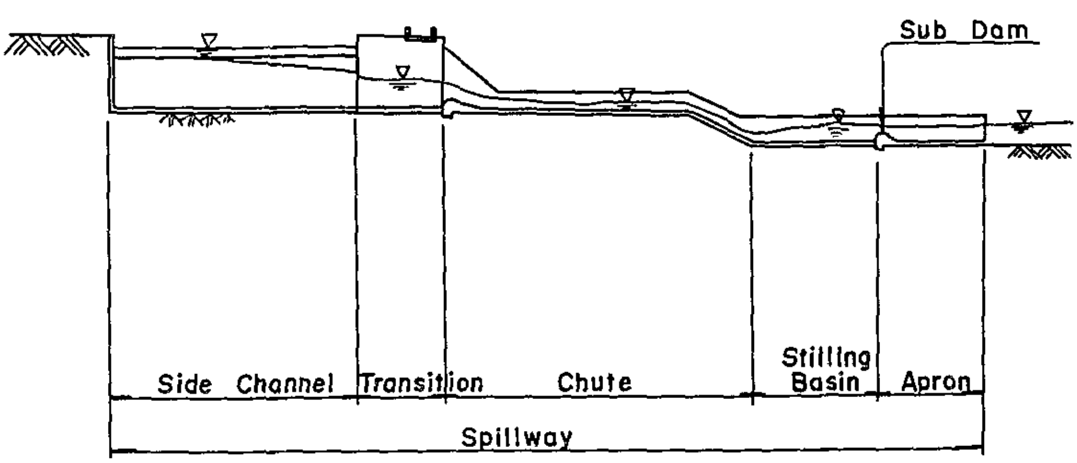


Fig.12.4.2 Side Channel Profile

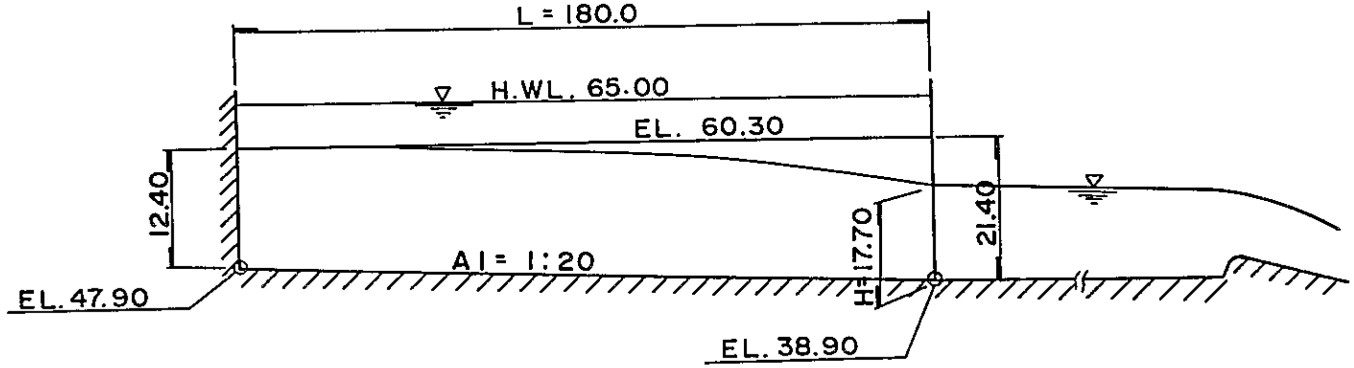


Fig.12.4.3 Side Channel Cross Section

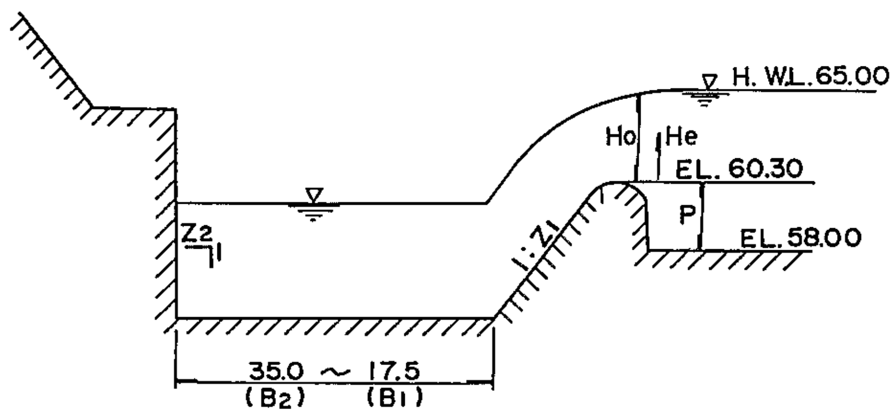


Fig.12.4.3 Standard Section of Stilling Basin

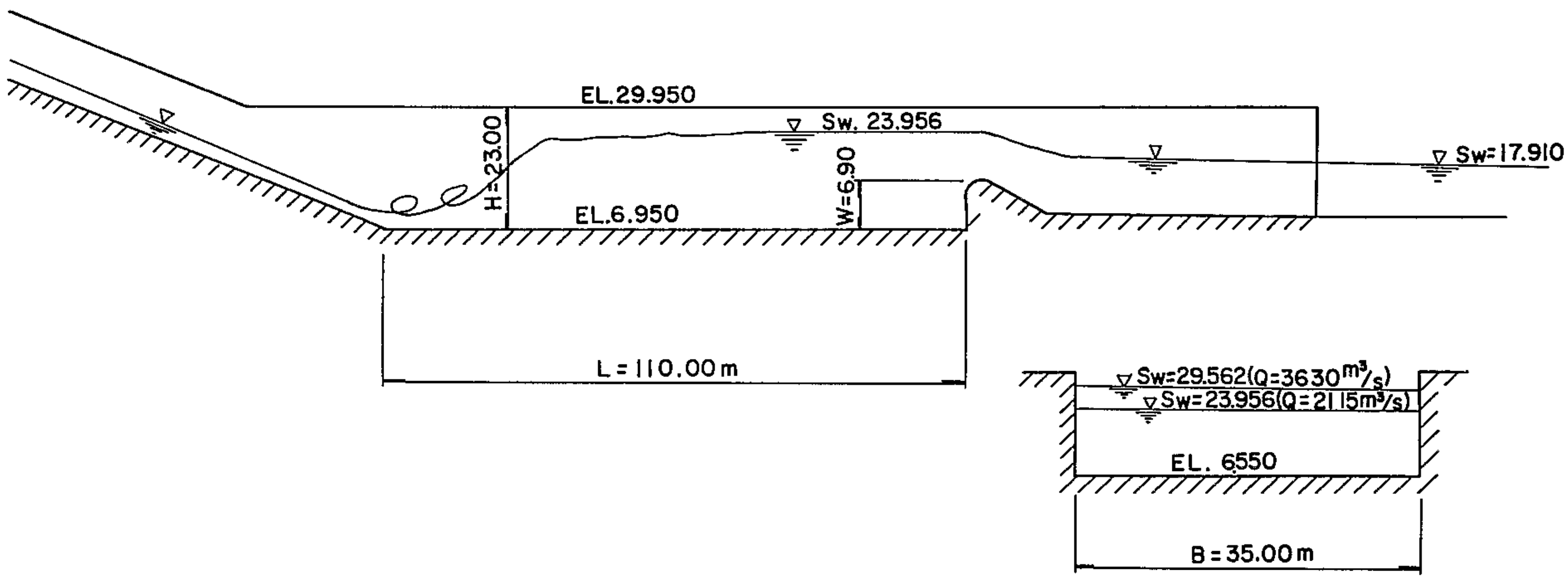


Fig.12.4.4 Profile of Gated Spillway

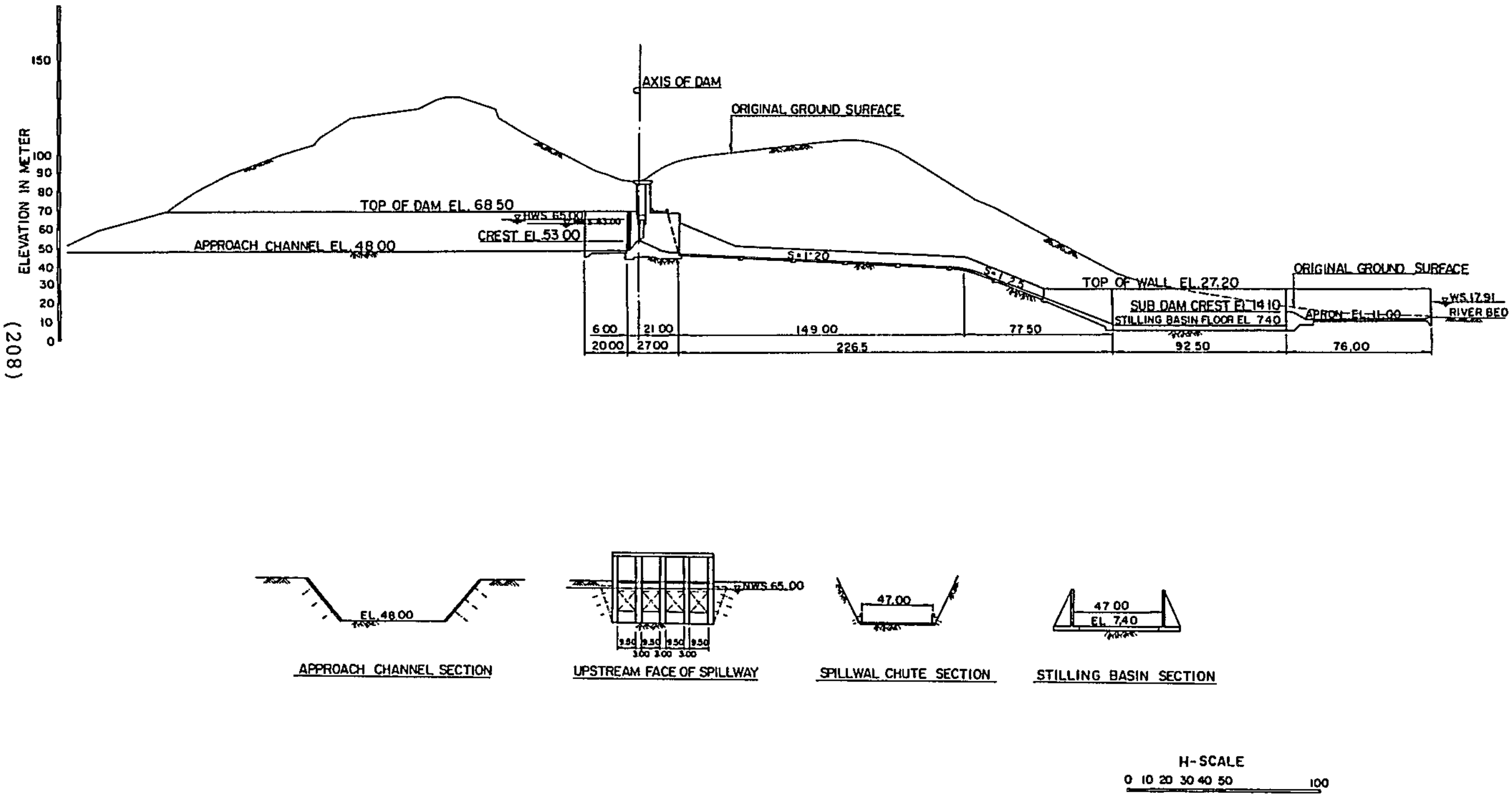


Fig.12.4.5 Profile of Side-channel Spillway

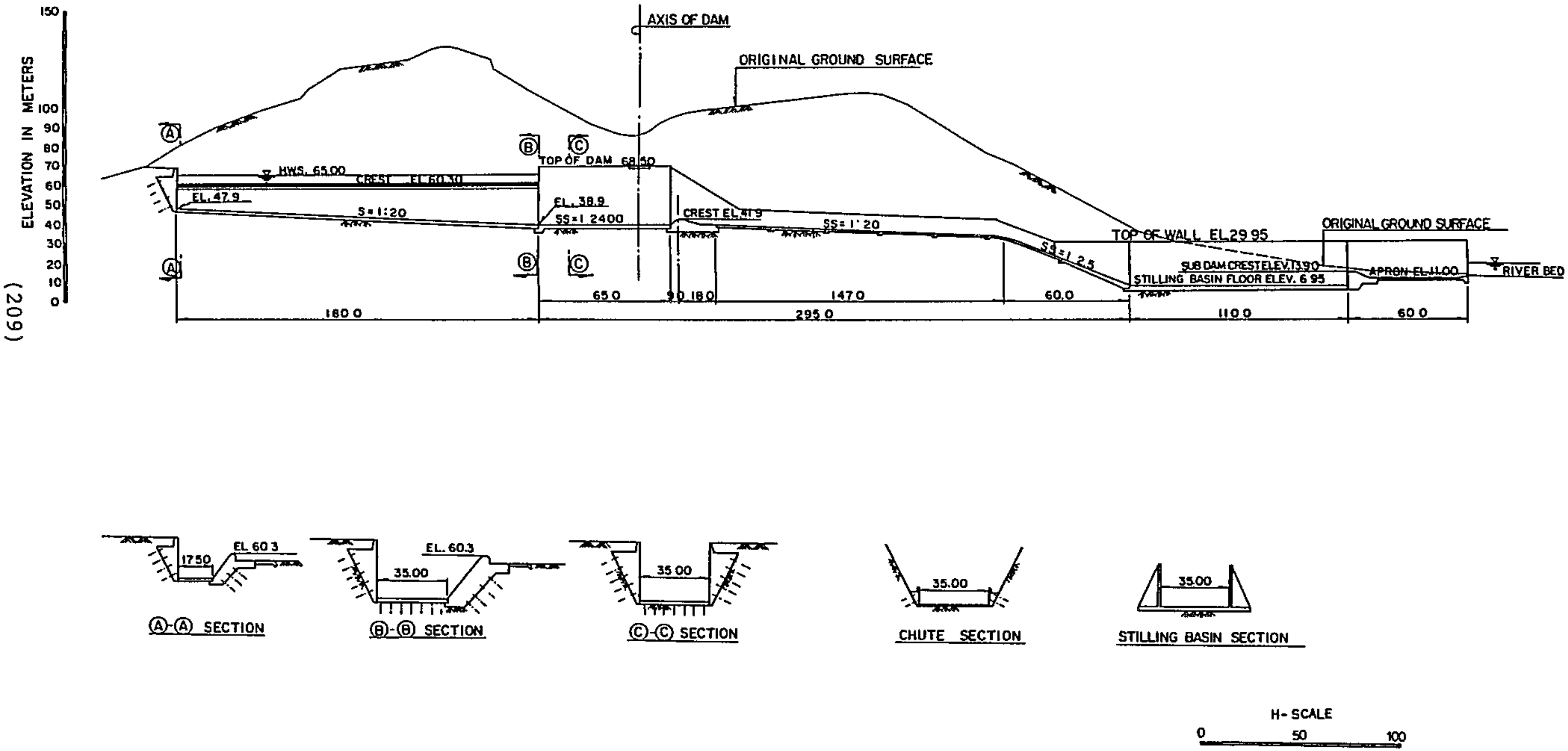


Table 12-4-1 Discharge Over an Uncontrolled Overflow Ogee Crest

Overflow Depth (He)	Dam Water Level (EL)	He/Ho	C/Co	Coefficient of Discharge (C)	Q = $C \cdot L' \cdot He^{3/2}$	V	Ha $0.35 V^2/2G$	Stored Water Level (EL + Ha)
0.7 m	61.0 m	0.159	0.837	1.758	185.327 m^3s	0.412 m/s	0 m	61.0 m
1.2	61.5	0.272	0.867	1.821	430.877	0.798	0.016	61.5
1.7	62.0	0.386	0.890	1.869	745.685	1.184	0.036	62.0
2.7	63.0	0.614	0.935	1.964	1568.409	1.936	0.096	63.1
3.7	64.0	0.840	0.970	2.037	2609.552	2.636	0.178	64.2
4.7	65.0	1.068	1.000	2.100	3851.593	3.292	0.277	65.3
5.7	66.0	1.295	1.025	2.153	5273.838	3.907	0.389	66.4

Ho: Design head over ogee crest

Table 12-4-2 FLOOD ROUTING

TIME (HR)	INFLOW (M**3/S)	OUTFLOW (M**3/S)	WATER LEVEL (M)	STORAGE CAPACITY (M**3)
0.00	20.000	0.000	60.30	264200000.
1.00	40.000	1.753	60.31	264304830.
2.00	50.000	4.670	60.32	264455260.
3.00	60.000	7.958	60.33	264630530.
4.00	75.000	11.840	60.35	264837890.
5.00	90.000	16.454	60.36	265083950.
6.00	120.000	22.384	60.39	265392030.
7.00	150.000	29.757	60.41	265784180.
8.00	220.000	40.042	60.45	266324530.
9.00	360.000	56.419	60.51	267194900.
10.00	480.000	80.415	60.60	268460540.
11.00	560.000	109.327	60.71	269990910.
12.00	630.000	141.282	60.83	271681790.
13.00	670.000	174.763	60.96	273452800.
14.00	690.000	221.461	61.07	275187460.
15.00	710.000	271.601	61.18	276819710.
16.00	730.000	318.563	61.27	278349310.
17.00	740.000	362.183	61.36	279769860.
18.00	745.000	401.787	61.44	281067520.
19.00	750.000	439.936	61.52	282243330.
20.00	755.000	481.343	61.58	283293950.
21.00	760.000	517.927	61.64	284222210.
22.00	765.000	550.323	61.69	285044220.
23.00	770.000	579.087	61.74	285774080.
24.00	775.000	604.392	61.78	286424580.
25.00	780.000	627.344	61.81	287006210.
26.00	782.000	647.720	61.85	287522560.
27.00	785.000	665.724	61.87	287978750.
28.00	787.000	681.682	61.90	288382980.
29.00	790.000	695.843	61.92	288741890.
30.00	792.000	708.467	61.94	289061630.
31.00	795.000	719.457	61.96	289347840.
32.00	797.000	729.631	61.98	289604860.
33.00	800.000	738.787	61.99	289836290.
34.00	805.000	748.246	62.00	290048510.
35.00	810.000	760.347	62.02	290240000.
36.00	820.000	771.558	62.03	290416380.
37.00	840.000	783.471	62.05	290605310.
38.00	850.000	796.038	62.06	290803970.
39.00	870.000	809.081	62.08	291010560.
40.00	885.000	823.029	62.09	291231740.
41.00	900.000	837.227	62.11	291456260.
42.00	920.000	852.091	62.13	291691260.
43.00	930.000	866.979	62.15	291926780.
44.00	935.000	880.437	62.16	292138240.
45.00	940.000	892.162	62.18	292322560.
46.00	945.000	902.506	62.19	292485120.
47.00	950.000	911.746	62.20	292630270.

TIME	INFLOW	OUTFLOW	WATER LEVEL	STORAGE CAPACITY
(HR)	(M**3/S)	(M**3/S)	(M)	(M**3)
48.00	955.000	919.705	62.21	292762620.
49.00	960.000	927.489	62.22	292884480.
50.00	965.000	934.694	62.23	292997380.
51.00	970.000	941.436	62.24	293103100.
52.00	980.000	948.265	62.25	293211390.
53.00	990.000	955.709	62.26	293330180.
54.00	1010.000	965.074	62.27	293472510.
55.00	1020.000	975.218	62.28	293633790.
56.00	1040.000	986.378	62.29	293810690.
57.00	1070.000	1000.288	62.31	294032640.
58.00	1100.000	1017.486	62.33	294306560.
59.00	1120.000	1036.342	62.35	294605570.
60.00	1160.000	1057.897	62.38	294939900.
61.00	1200.000	1082.804	62.41	295334400.
62.00	1240.000	1110.611	62.44	295777790.
63.00	1280.000	1141.267	62.48	296259840.
64.00	1330.000	1174.697	62.52	296788990.
65.00	1380.000	1211.518	62.57	297371650.
66.00	1420.000	1250.020	62.61	297980670.
67.00	1490.000	1291.874	62.66	298643200.
68.00	1560.000	1339.905	62.72	299395840.
69.00	1600.000	1389.003	62.78	300171780.
70.00	1660.000	1438.289	62.84	300950530.
71.00	1720.000	1489.747	62.90	301763840.
72.00	1840.000	1549.038	62.98	302701820.
73.00	2000.000	1623.074	63.05	303904000.
74.00	2120.000	1709.783	63.14	305320700.
75.00	2260.000	1805.087	63.23	306877700.
76.00	2330.000	1902.329	63.32	308466180.
77.00	2400.000	1994.249	63.41	309966340.
78.00	2500.000	2084.754	63.50	311443970.
79.00	2600.000	2177.119	63.59	312952580.
80.00	2800.000	2280.874	63.69	314648060.
81.00	2920.000	2395.782	63.80	316526080.
82.00	3080.000	2516.250	63.91	318484220.
83.00	3240.000	2664.058	64.04	320535550.
84.00	3420.000	2873.042	64.21	322556670.
85.00	3660.000	3082.519	64.38	324580610.
86.00	3990.000	3316.235	64.57	326832640.
87.00	3990.000	3527.323	64.74	328078080.
88.00	3720.000	3630.707	64.82	329871620.
89.00	3240.000	3584.388	64.78	329412350.
90.00	2670.000	3387.040	64.63	327501570.
91.00	2480.000	3131.548	64.42	325038080.
92.00	2240.000	2889.170	64.22	322696700.
93.00	2000.000	2647.569	64.03	320362500.
94.00	1720.000	2477.471	63.87	317833220.
95.00	1560.000	2311.154	63.71	315117570.

TIME (HR)	INFLOW (M**3/S)	OUTFLOW (M**3/S)	WATER LEVEL (M)	STORAGE CAPACITY (M**3)
96.00	1320.000	2138.181	63.55	312292610.
97.00	1220.000	1965.764	63.38	309477380.
98.00	1140.000	1809.662	63.23	306929410.
99.00	1100.000	1672.625	63.10	304693250.
100.00	1040.000	1552.704	62.98	302739460.
101.00	1000.000	1443.411	62.85	301018370.
102.00	940.000	1346.935	62.73	299487740.
103.00	920.000	1261.570	62.63	298140420.
104.00	880.000	1187.527	62.54	296972030.
105.00	860.000	1122.524	62.46	295945730.
106.00	830.000	1065.680	62.39	295048960.
107.00	800.000	1014.373	62.33	294238720.
108.00	760.000	966.405	62.27	293481220.
109.00	740.000	922.103	62.21	292781820.
110.00	720.000	883.148	62.17	292160260.
111.00	680.000	845.638	62.12	291568380.
112.00	630.000	806.684	62.07	290952190.
113.00	560.000	763.448	62.02	290267900.
114.00	520.000	727.450	61.97	289528060.
115.00	440.000	694.719	61.92	288696060.
116.00	340.000	654.484	61.85	287671300.
117.00	240.000	606.333	61.78	286445820.
118.00	160.000	552.638	61.69	285079550.
119.00	120.000	498.050	61.61	283692290.
120.00	90.000	446.027	61.52	282370820.
121.00	70.000	404.544	61.45	281127680.
122.00	50.000	368.440	61.37	279952130.
123.00	45.000	334.802	61.30	278857220.
124.00	40.000	304.160	61.24	277859840.
125.00	40.000	276.457	61.18	276958720.
126.00	35.000	251.405	61.13	276143360.
127.00	33.000	228.610	61.09	275401730.
128.00	30.000	207.950	61.05	274729220.
129.00	27.000	189.366	61.01	274116610.
130.00	25.000	176.965	60.97	273550590.
131.00	20.000	166.789	60.93	273012740.
132.00	20.000	157.247	60.89	272501250.
133.00	20.000	148.190	60.86	272023300.
134.00	20.000	139.735	60.83	271576830.
135.00	20.000	131.833	60.80	271159810.
136.00	20.000	124.448	60.77	270770430.
137.00	20.000	117.560	60.74	270406660.
138.00	20.000	111.125	60.72	270066940.
139.00	20.000	105.110	60.70	269749500.

Table 12-4-3 HIDRAVLIC COMPUTATION FOR SIDE-CHANNEL SPILLWAY

B1 =	17.500 (M)	B2 =	35.000 (M)
Z1 =	0.7000	Z2 =	0.0000
L =	180.000 (M)	N =	0.015
Q =	3630.000 (M**3/S)	ALP =	1.100
Z0 =	0.000 (M)	AI =	1/ 20.00
H0 =	17.700 (M)		
Q0 =	3630.000 (M**3/S)		

NO.	(M)	(M)	(M)	(M**3/S)	(M)	(M)	(M)	(M**2)	(M/S)
0.	0.000	0.000	0.000	3630.000	0.000	17.700	17.700	729.151	4.978
1.	7.200	7.200	0.360	3484.800	0.125	17.825	17.465	705.807	4.937
2.	18.000	10.800	0.900	3267.000	0.199	18.024	17.124	671.985	4.862
3.	36.000	18.000	1.800	2904.000	0.360	18.384	16.584	618.659	4.694
4.	59.940	23.940	2.997	2421.210	0.533	18.917	15.920	553.146	4.377
5.	83.880	23.940	4.194	1938.421	0.584	19.501	15.307	492.941	3.932
6.	107.820	23.940	5.391	1455.631	0.610	20.111	14.720	436.743	3.333
7.	131.760	23.940	6.588	972.841	0.591	20.702	14.114	382.908	2.541
8.	155.700	23.940	7.785	490.052	0.491	21.193	13.408	329.249	1.488
9.	180.000	24.300	9.000	0.002	0.227	21.420	12.420	271.339	0.000

Table 12-4-4 CHUTE HYDRAULIC COMPUTATION

B =	35.000	1:T,	T= 0
Q =	2115.000	TL=	235.000
Z1=	41.900	Z2 =	2.950
N =	0.015	ALFA=	1.100

	Uniform flow			Critical		
Q(M*3/S)	H(M)	A(M*2)	V(M/S)	HC(M)	AC(M*2)	VC(M/S)
2115.000	1.192	41.708	50.709	7.428	259.987	8.135

NO	L(M)	TL(M)	Z(M)	H(M)	A(M*2)	V(M/S)	HV(M)	H+Z(M)	F	B
0	0.000	0.000	41.900	7.428	259.987	8.135	3.714	49.328	0.953	35.000
1	2.000	2.000	40.900	5.598	195.942	10.794	6.539	46.498	1.457	35.000
2	2.000	4.000	39.900	5.031	176.075	12.012	8.098	44.931	1.711	35.000
3	2.000	6.000	38.900	4.654	162.881	12.985	9.463	43.554	1.923	35.000
4	31.800	37.800	37.310	4.281	149.837	14.115	11.182	41.591	2.179	35.000
5	31.800	69.600	35.720	4.011	140.395	15.065	12.737	39.731	2.403	35.000
6	31.800	101.400	34.130	3.803	133.108	15.889	14.169	37.933	2.603	35.000
7	31.800	133.200	32.540	3.636	127.260	16.619	15.501	36.176	2.784	35.000
8	31.800	165.000	30.950	3.498	122.440	17.274	16.746	34.448	2.950	35.000
9	5.000	170.000	28.950	3.297	115.379	18.331	18.858	32.247	3.225	35.000
10	5.000	175.000	26.950	3.130	109.547	19.307	20.920	30.080	3.486	35.000
11	5.000	180.000	24.950	2.989	104.618	20.216	22.937	27.939	3.735	35.000
12	5.000	185.000	22.950	2.868	100.375	21.071	24.917	25.818	3.975	35.000
13	5.000	190.000	20.950	2.762	96.670	21.879	26.864	23.712	4.205	35.000
14	5.000	195.000	18.950	2.669	93.399	22.645	28.779	21.619	4.428	35.000
15	5.000	200.000	16.950	2.585	90.483	23.374	30.663	19.535	4.644	35.000
16	5.000	205.000	14.950	2.510	87.863	24.072	32.520	17.460	4.853	35.000
17	5.000	210.000	12.950	2.443	85.492	24.739	34.349	15.393	5.056	35.000
18	5.000	215.000	10.950	2.381	83.334	25.380	36.151	13.331	5.254	35.000
19	5.000	220.000	8.950	2.325	81.360	25.996	37.926	11.275	5.446	35.000
20	5.000	225.000	6.950	2.273	79.546	26.588	39.675	9.223	5.634	35.000
21	5.000	230.000	4.950	2.225	77.873	27.160	41.398	7.175	5.816	35.000
22	5.000	235.000	2.950	2.181	76.324	27.711	43.096	5.131	5.994	35.000

Table 12-4-5 CHUTE HYDRAULIC COMPUTAION

B =	35.000	1:T, T=	0
Q =	3630.000	TL=	235.000
Z1=	41.900	Z2 =	2.950
N =	0.015	ALFA=	1.100

	Uniform flow			Critical		
Q(M*3/S)	H(M)	A(M*2)	V(M/S)	HC(M)	AC(M*2)	VC(M/S)
3630.000	1.664	58.253	62.314	10.648	372.691	9.740

NO	L(M)	TL(M)	Z(M)	H(M)	A(M*2)	V(M/S)	HV(M)	H+Z(M)	F	B
0	0.000	0.000	41.900	10.648	372.691	9.740	5.324	52.548	0.953	35.000
1	2.000	2.000	40.900	8.389	293.622	12.363	8.578	49.289	1.363	35.000
2	2.000	4.000	39.900	7.654	267.883	13.551	10.305	47.554	1.565	35.000
3	2.000	6.000	38.900	7.154	250.391	14.497	11.795	46.054	1.731	35.000
4	31.800	37.800	37.310	6.636	232.250	15.630	13.710	43.946	1.938	35.000
5	31.800	69.600	35.720	6.251	218.795	16.591	15.448	41.971	2.120	35.000
6	31.800	101.400	34.130	5.948	208.177	17.437	17.064	40.078	2.284	35.000
7	31.800	133.200	32.540	5.699	199.482	18.197	18.584	38.239	2.435	35.000
8	31.800	165.000	30.950	5.491	192.175	18.889	20.024	36.441	2.575	35.000
9	5.000	170.000	28.950	5.210	182.348	19.907	22.241	34.160	2.786	35.000
10	5.000	175.000	26.950	4.974	174.085	20.852	24.402	31.924	2.987	35.000
11	5.000	180.000	24.950	4.771	166.993	21.737	26.519	29.721	3.179	35.000
12	5.000	185.000	22.950	4.594	160.806	22.574	28.599	27.544	3.364	35.000
13	5.000	190.000	20.950	4.438	155.341	23.368	30.646	25.388	3.543	35.000
14	5.000	195.000	18.950	4.299	150.462	24.126	32.666	23.249	3.717	35.000
15	5.000	200.000	16.950	4.173	146.069	24.851	34.661	21.123	3.886	35.000
16	5.000	205.000	14.950	4.060	142.087	25.548	36.631	19.010	4.050	35.000
17	5.000	210.000	12.950	3.956	138.453	26.218	38.579	16.906	4.211	35.000
18	5.000	215.000	10.950	3.861	135.120	26.865	40.505	14.811	4.368	35.000
19	5.000	220.000	8.950	3.773	132.048	27.490	42.412	12.723	4.521	35.000
20	5.000	225.000	6.950	3.692	129.205	28.095	44.299	10.642	4.671	35.000
21	5.000	230.000	4.950	3.616	126.564	28.681	46.166	8.566	4.818	35.000
22	5.000	235.000	2.950	3.546	124.103	29.250	48.016	6.496	4.962	35.000

Table 12-4-6 Hydraulic Specification of the Stilling Basin

Item \ Discharge		$3,630^{m^3/s}$	$2,115^{m^3/s}$
Channel Width	b^{m}	35.00	35.00
Elevation of Stilling Basin	z^{m}	6.950	6.950
Inflow Depth	d_1^{m}	3.692	2.273
Inflow Velocity	$v_1^{m/s}$	28.095	26.588
Fluid Number	F	4.671	5.633
Hydraulic Jump Depth	d_2^{m}	22.612	17.006
Velocity	$v_2^{m/s}$	4.587	3.553
Velocity Head	hv_2^{m}	1.073	0.644
Specific Energy	E_2^{m}	23.256	17.650
Hydraulic Jump	WL_2^{m}	29.562	23.956
Energy	Eh_2^{m}	30.206	24.600

CHAPTER 13

DIVERSION WORKS

13. DIVERSION WORKS

13.1 Tunnel and Temporary Cofferdam

Results of the comparison are presented in Table Table 13.1.1.

13.2 Determination of Tunnel Cross Section and Elevation of Temporary Cofferdam

Design conditions

Diversion flood discharge	Q = 1,500 m^3/sec
Tunnel entrance elevation	EL = 13.20 m
Tunnel outlet elevation	EL = 11.00 m
Tunnel length	L = 770 m
Tunnel cross section	Standard horseshoe type (2R type)
Coefficient of roughness	n = 0.012

It is assumed that a full-flow takes place in the interior of the tunnel, and the hydraulic calculations are carried out by the Manning's Formula.

Elevation of the original point for the hydraulic study at the outlet of the tunnel is assumed to be 0.85 x D (inner diameter of the tunnel).

The specifications of the diversion tunnel are presented in Table 13.2.1 and Table 13.2.2, regarding the cases of 1 tunnel and 2 tunnels, respectively.

Table 13-1-1 Combination of the tunnel diameter

		Case 1	Case 2	Case 3	Case 4	Case 5
Tunnel dia (m)	1	8.5	9.0	10.0	10.4	10.8
	2	8.5	8.3	7.3	6.8	6.2
Velocity (m/s)	1	12.518	12.200	12.141	12.099	12.046
	2	12.518	11.955	11.226	10.867	10.435
Discharge (m^3/s)	1	750	819	1,007	1,085	1,165
	2	750	683	496	417	333
	Total	1,500	1,502	1,503	1,502	1,498
Cost $x10^3$ ₱	1	49,100	53,330	62,370	66,600	71,610
	2	49,100	47,740	40,420	36,780	33,110
	Total	98,200	101,070	102,790	103,380	104,720
$x10^3$ US$		12,275	12,633	12,848	12,922	13,090

Table 13-1-2 The relationship between the diameter of tunnel and the crest elevation of cofferdam

Item	Case 1	Case 2	Case 3	Case 4
Tunnel dia (m)	8.4	8.5	8.6	8.8
Coffer dam crest EL (m)	36.9	36.0	35.4	34.0
Volume of coffer dam embankment (m^3)	$300x10^3$	$260x10^3$	$240x10^3$	$210x10^3$
Construction cost($x10^3$₱)				
Diversion tunnel	97,020	98,200	100,100	103,180
Coffer dam embankment	12,000	10,400	9,600	8,400
Total $x10^3$₱	109,020	108,600	109,700	111,580
$x10^3$US$	13,627	13,575	13,712	13,947

Fig.13.2.1 Longitudinal Section of Tunnel

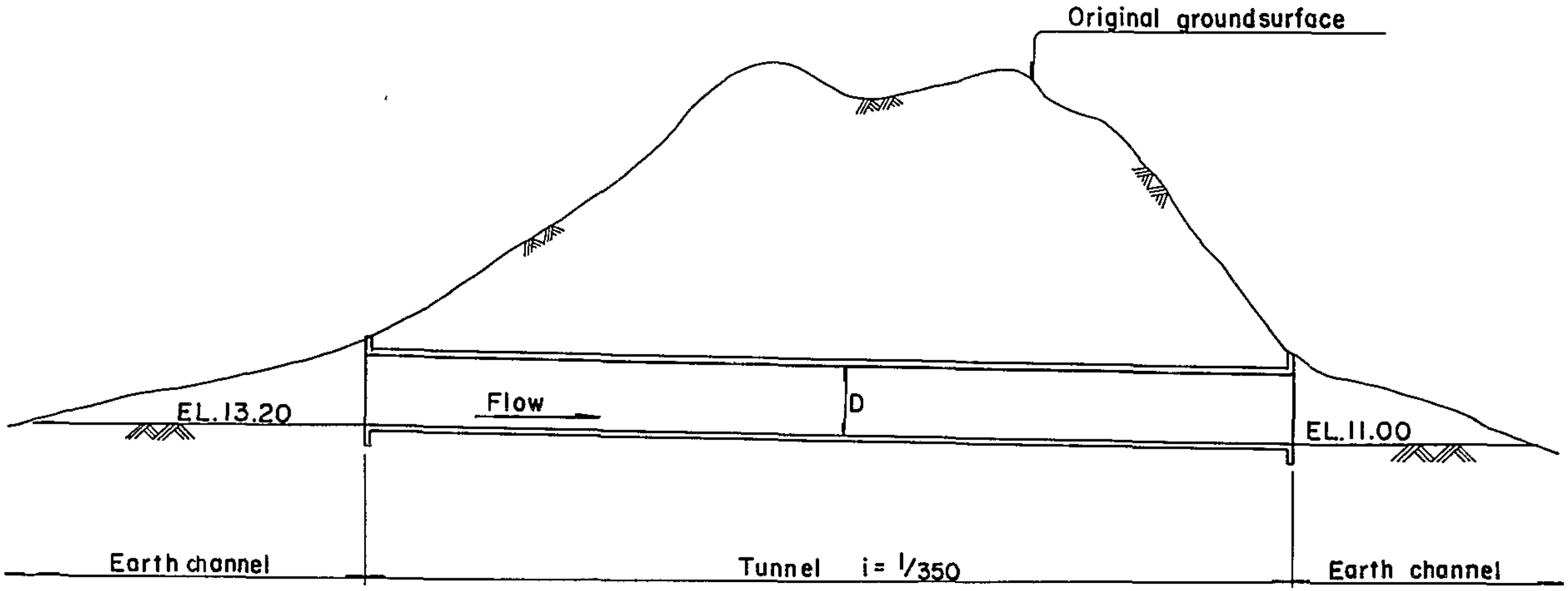


Fig.13.2.2 Profile of Diversion Tunnel

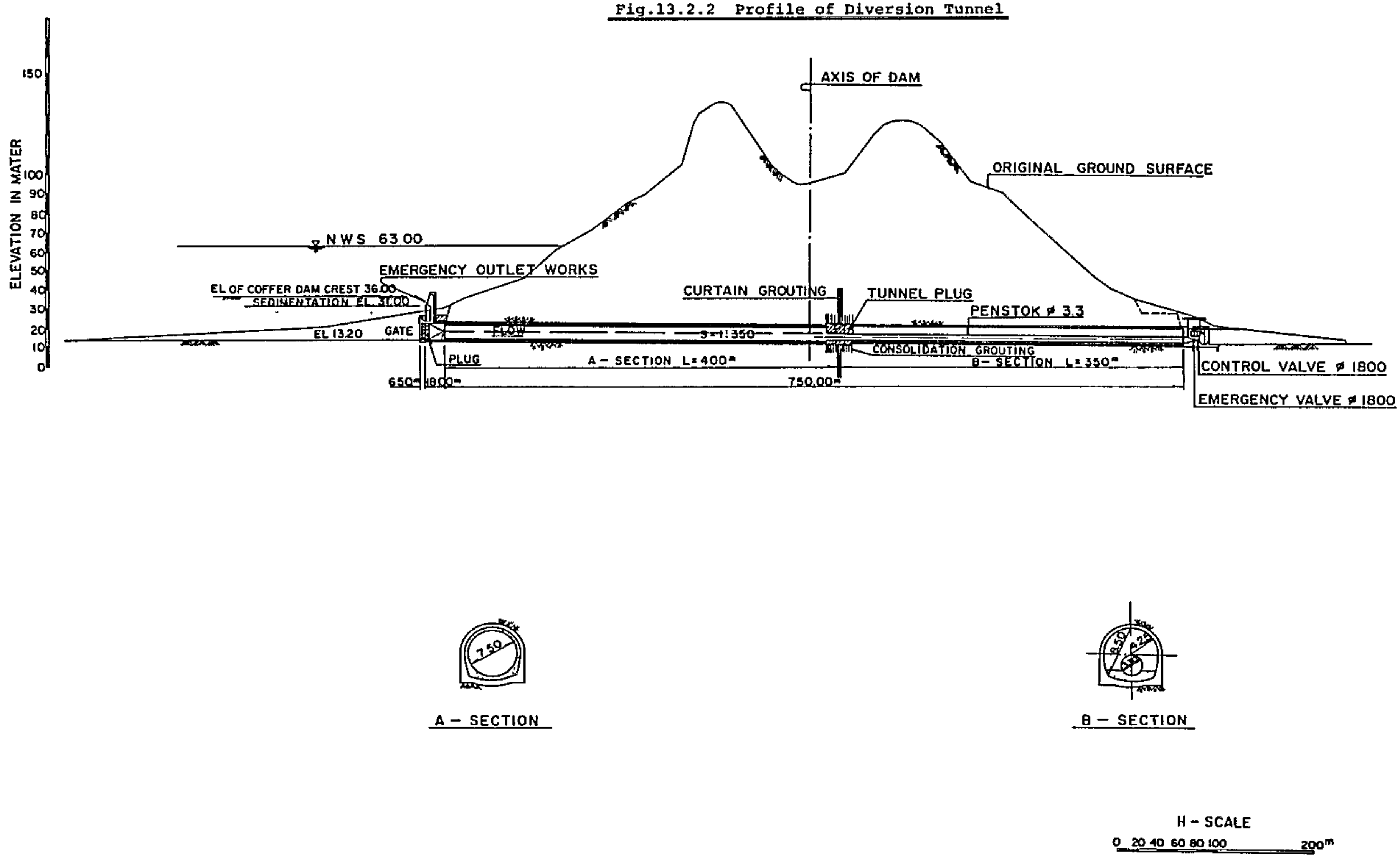


Table 13-2-1 Comparative Specifications of Tunnel Diameters and Temporary Cofferdam

one tunnel $Q=1,500 m^3/s$

Dia	12.0^M	12.2^M	12.4^M
$A = 3.317 (^D/2)^2$	119.412	123.426	127.505
$V = {}^Q/A$	12.562	12.153	11.764
$hv = v^2/19.6$	8.051	7.536	7.061
Downstream Side Elevation Z_o	11.000	11.000	11.000
Elevation at Prescribed Point of Downstream side	21.200	21.370	21.540
$f = 122 \cdot n^2/D^{4/3}$	0.000639	0.000626	0.000612
Fliction Loss $h_f = f \cdot L \cdot hv$	3.964	3.630	3.328
Inflow/Run-off Loss $hi = 1.2 \cdot hv$	9.661	9.043	8.473
Total Loss $\Sigma h = h_f + hi$	13.625	12.673	11.801
Upstream Side Water Level $H = Z + \Sigma h$	34.825	34.043	33.341
Upstream Side Elevation Zi	13.200	13.200	13.200
1.5 x D	18.000	18.300	18.600
High Water Level Condition Hi = Zi+1.5xD	31.200	31.500	31.800
High Water Level Condition of Full Flow	OK	OK	OK
Elevation of Temporary Cofferdam Crest (H+2)	36.9	36.0	35.4

Table 13-2-2 Comparative Specification of Tunnel Diameters and Temporary Cofferdam

two tunnels $Q=\frac{1500}{2}=750 m^3/s$

Dia	8.2^M	8.4^M	8.5^M	8.6^M	8.8^M
$A = 3.317\ (^D/2)^2$	55.759	58.512	59.913	61.331	64.217
$V = {}^Q/A$	13.451	12.818	12.518	12.229	11.679
$hv = v^2/19.6$	9.231	8.383	7.995	7.630	6.959
Downstream Side Elevation Z	11.000	11.000	11.000	11.000	11.000
Elevation at Prescribed Point of Downstream Side	17.970	18.140	18.225	18.310	18.480
$f = 122 \cdot n^2/D^{4/3}$	0.001062	0.001029	0.001013	0.000997	0.000967
Fliction Loss $h_f = f \cdot L \cdot hv$	7.549	6.642	6.235	5.857	5.182
Inflow/Run-off Loss $hi = 1.2 \cdot hv$	11.077	10.060	9.594	9.156	8.351
Total Loss $\Sigma h = h_f + hi$	18.626	16.702	15.829	15.013	13.533
Upstream Side Water Level $H = Z + \Sigma h$	36.596	34.842	34.054	33.323	32.013
Upstream Side Elevation Zi	13.200	13.200	13.200	13.200	13.200
1.5 x D	12.300	12.600	12.750	12.900	13.200
High Water Level Condition Hi = Zi+1.5xD	25.500	25.800	25.950	26.100	26.400
High Water Level Condition of Full Flow	OK	OK	OK	OK	OK
Elevation of Temporary Cofferdam Crest	38.6	36.9	36.0	35.4	34.0

CHAPTER 14

INTAKE FACILITY

14. INTAKE FACILITY

14.1 Type and Structure of the Intake Facility

The tower type and shaft type intake works can be taken into consideration as possible alternatives which might be adopted in the present case. These two alternatives are compared from the economical point of view. The outlines of the two alternatives are presented in Figures 14.1.1 through 14.1.5. The comparison of the construction costs and other quantitative data is presented in Table 14.1.1 and Table 14.1.2.

14.2 Hydraulic Losses

The outline of this intake facility is presented in Figure 14.2.1, while the hydraulic study of the cross section corresponding to the various intake quantities and the losses of various kinds are presented in Tables 14.3.1 through 14.3.6.

The relation between the intake water quantity and loss water head is presented in Figure 14.2.2.

The following losses are taken into consideration in the present calculation.

$$HT = he + hf + hc + hex + hb + hv$$

where, HT: Total loss

he: Inflow loss

hf: Friction loss

hc: Cross section contraction loss

hex: Cross section expansion loss

hb: Bend loss

hv: Valve loss.

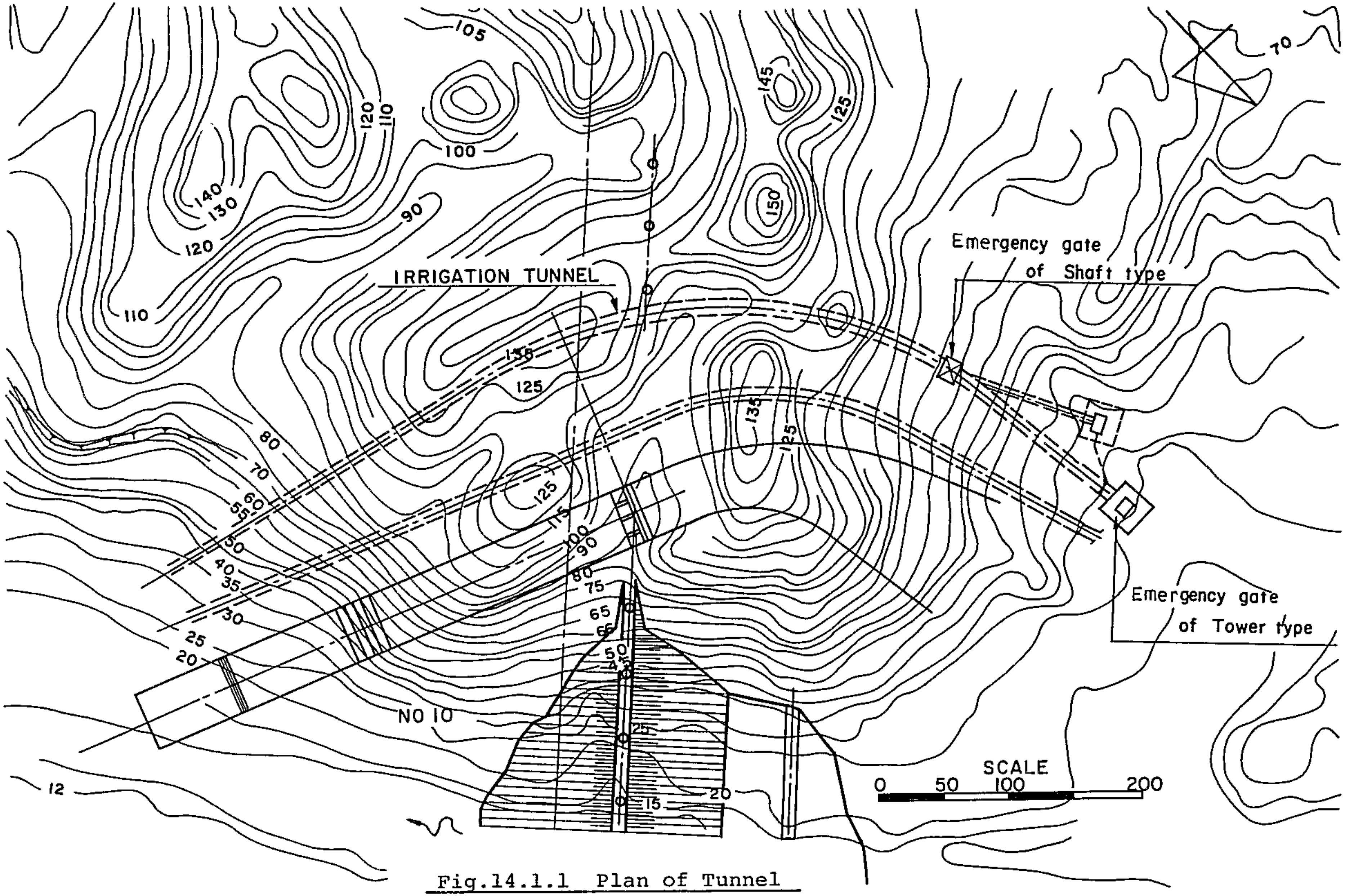


Fig.14.1.1 Plan of Tunnel

Fig.14.1.2 Profile of Tower Type Intake

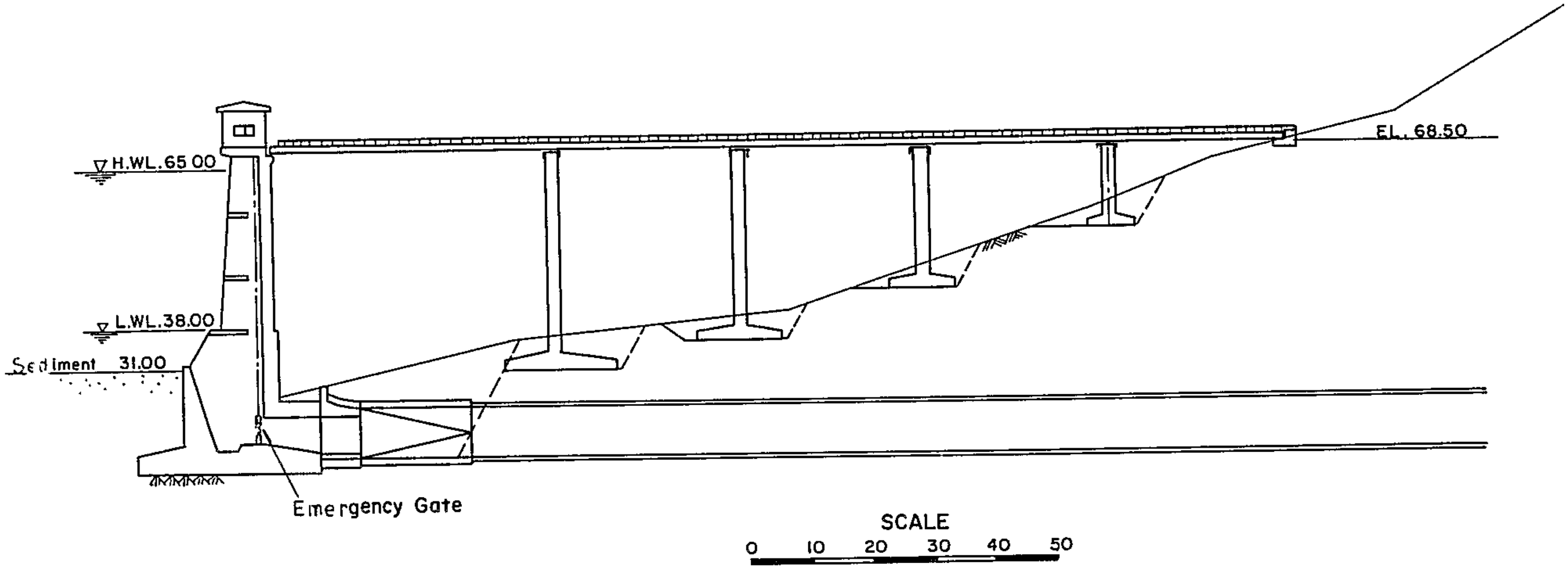


Fig.14.1.3 Tipical Section of Tower Type

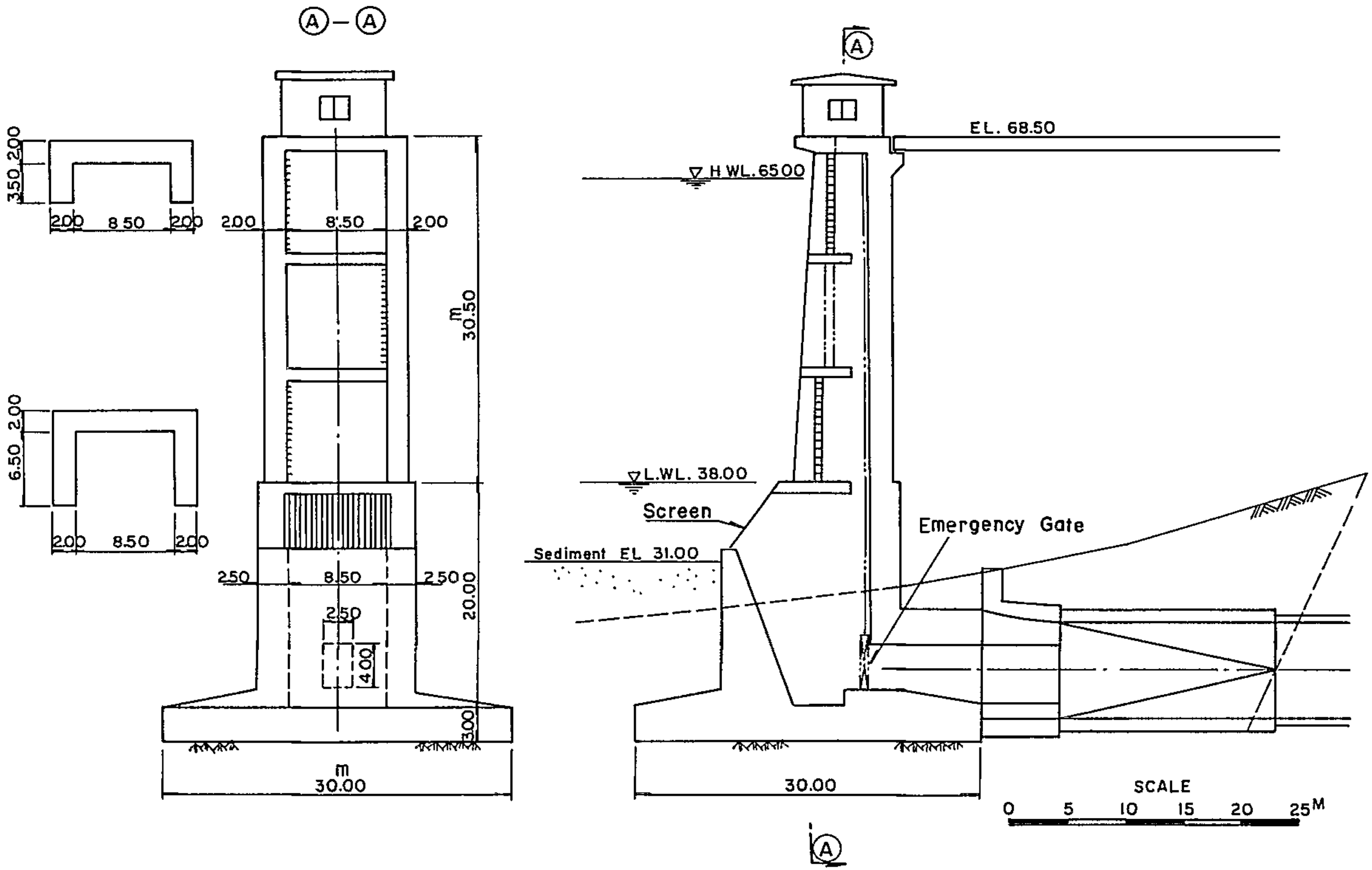


Fig.14.1.4 Profile of Shaft Type Intake

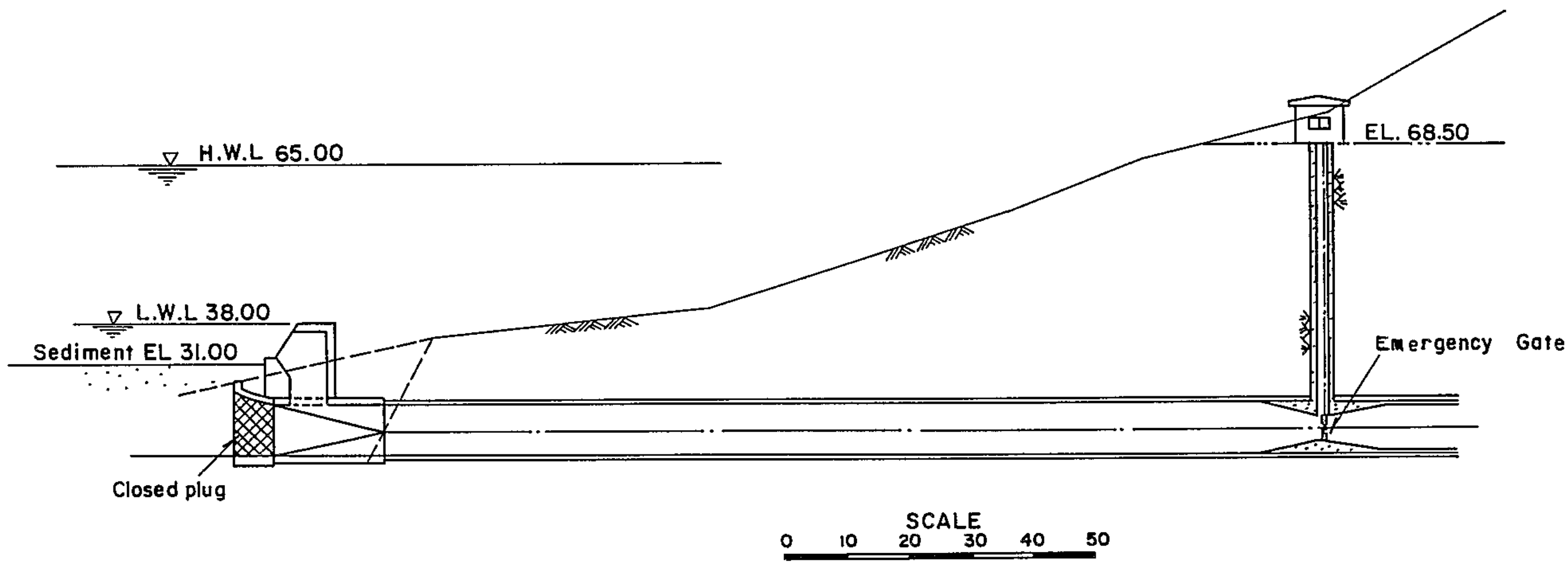


Fig.14.1.5 Typical Section of Shaft Type

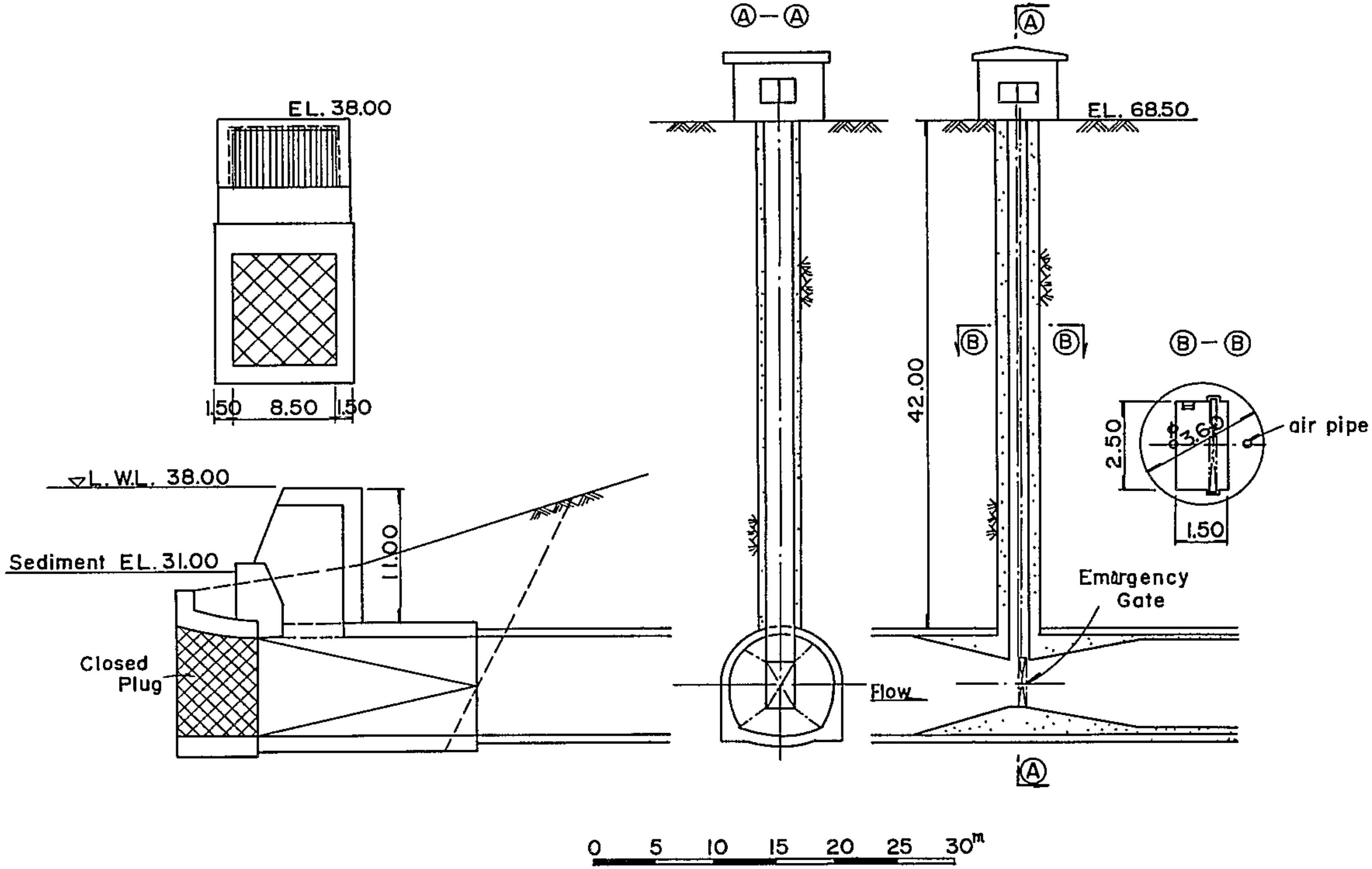


Table 14-1-1 Comparison of Construction Cost

Item	Unit	Unit Cost	Type of Intake			
			Tower Type		Shaft Type	
			Quantity	Amount	Quantity	Amount
Reinforced Concrete	m^3	₱ 600	8,665	x1,000₱ 5,199.0	712	x1,000₱ 427.2
Reinforcing Bars	t	6,500	173	1,124.5	25	162.5
Gate	t	63,000	22	1,386.0	22	1,386.0
Access Bridge	t	20,000	84	1,680.0	-	-
Shaft Excavation	m^3	350	-	-	428	149.8
Shaft Concrete	m^3	750	-	-	270	202.5
Plug Concrete	m	600	-	-	498	298.8
Total				9,415.5		2,626.8

Table 14-1-2 Construction Quantity of Intake Works

Tower type			Shaft type		
Reinforced Concrete	Tower	7,795 m^3	Reinforced Concrete	Intake	460 m^3
	Abutment	870		Gate	252
	Total	8,665		Total	712
Reinforcing Bars		173 ton	Reinforcing Bars		25 ton
Access Bridge		84 ton	Excavation	Shaft	428 m^3
Gate	B x H 2.5 x 4.0	1 LS	Gate	B x H 2.5 x 4.0	1 LS
			Concrete	Shaft	270 m^3
				Plug	498 m^3

Fig.14.2.1 Pictorial Representation of Intake Tunnel

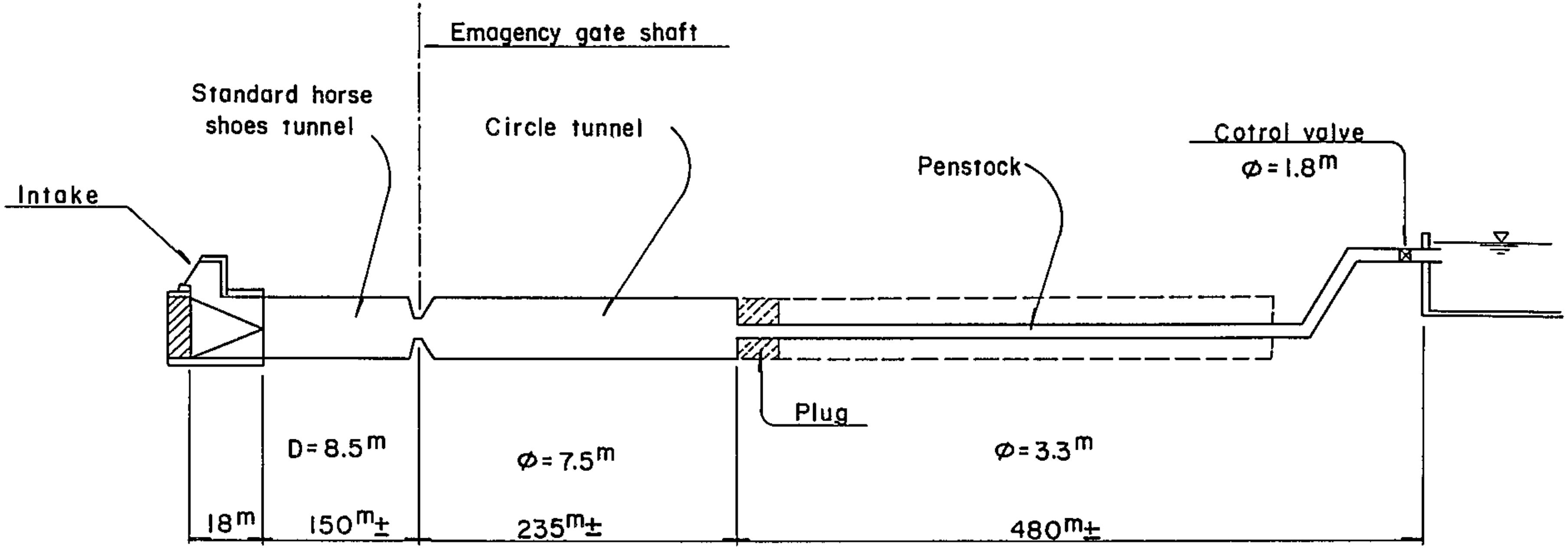


Fig. 14- 2- 2 Discharge - Loss Curve

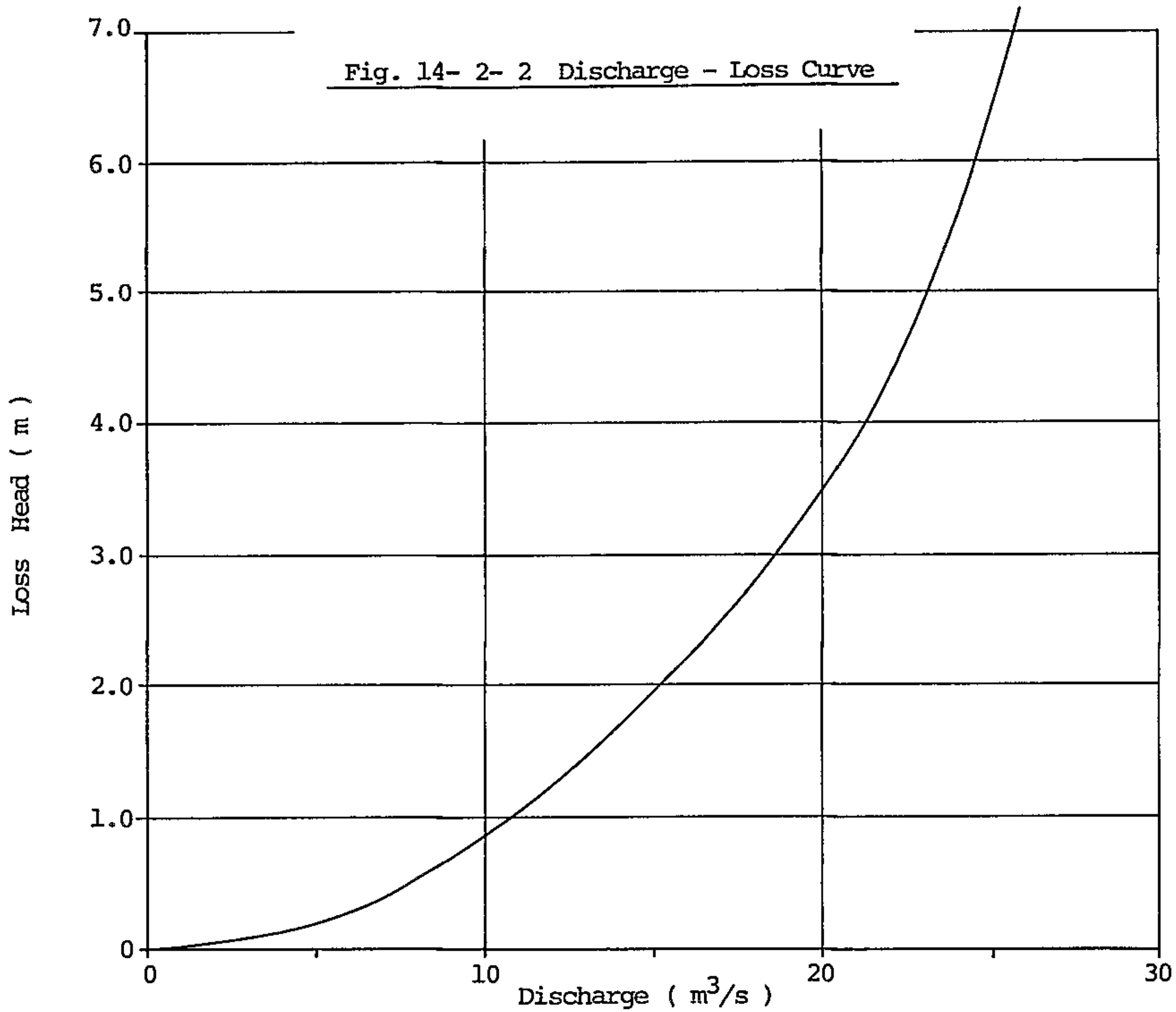


Fig.14.2.3 Profile of Intake Tunnel

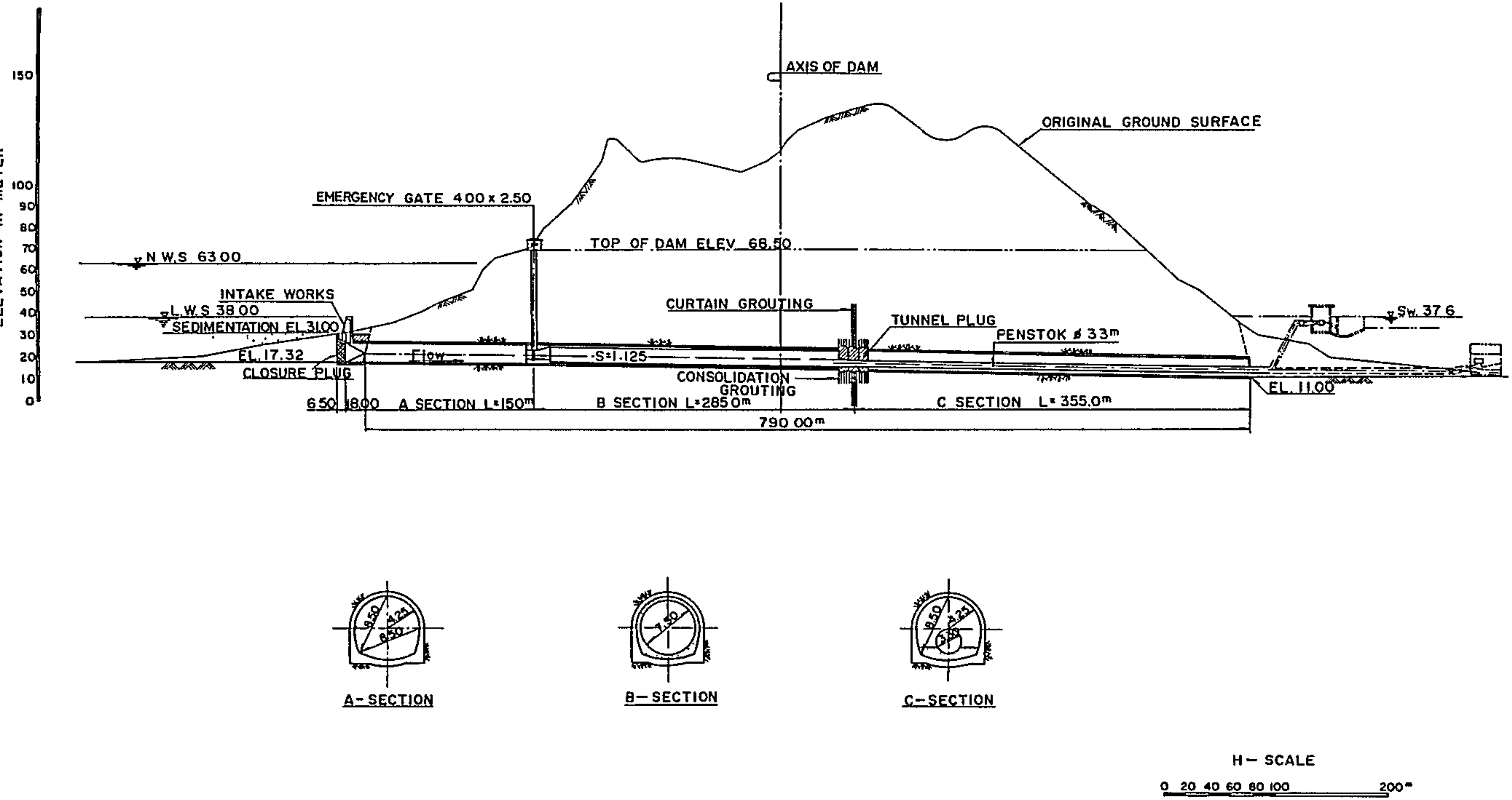


Table 14- 2- 1 Hydraulic Calculation

$Q = 25.31 m^3/s$

Section	n	A	V	hr	R	$S_f = \frac{V^2 \cdot n^2}{R^{4/3}}$
Inlet (8.5x5.0)	0.015	42.50	0.596	0.018	1.574	0.000044
Transition (8.5x8.5)	0.015	72.25	0.350	0.006	2.125	0.000010
H.S Tunnel (D=8.5)	0.015	59.91	0.422	0.009	2.148	0.000014
E.G Gate (4.0x2.5)		10.00	2.531	0.327		
C Tunnel (Ø = 7.5)	0.015	44.18	0.573	0.017	1.875	0.000032
Penstock (Ø = 3.3)	0.013	8.55	2.960	0.447	0.825	0.001916
Valve (Ø = 1.8)		2.54	9.946	5.047		

Head Loss

$Q = 25.31 m^3/s$

Inlet	Entrance	0.5 x 0.018	0.009
Transition	Contruction	0.1 (0.018 - 0.009)	0.001
H.S Tunnel	Friction	0.000044 x 150	0.007
Tunnel-Gate	Contruction Expantion	0.46 (0.327 - 0.009) 0.60 (0.327 - 0.017)	0.146 0.186
C. Tunnel	Friction	0.000032 x 235	0.075
Tunnel-Penstock	Contruction	0.45 (0.447 - 0.017)	0.194
Penstock	Friction	0.001916 x 480	0.920
"	Bend	0.3 x 0.447 x 2	0.268
Penstock-Valve	Contruction	0.41 (5.047 - 0.447)	1.886
Valve		0.38 x 5.047	2.826
Total			6.518

Table 14- 2- 2 Hydraulic Calculation

Q= 20.0 m^3/s

Section	n	A	V	hr	R	$S_f=\frac{V^2 \cdot n^2}{R^{4/3}}$
Inlet (8.5x5.0)	0.015	42.50	0.471	0.011	1.831	0.000027
Transition (8.5x8.5)	0.015	72.25	0.277	0.004	2.125	0.000006
H.S Tunnel (D=8.5)	0.015	59.91	0.334	0.006	2.771	0.000009
E.G Gate (4.0x2.5)		10.00	2.000	0.204		
C Tunnel (Ø = 7.5)	0.015	44.18	0.453	0.010	2.312	0.000020
Penstock (Ø = 3.3)	0.013	8.55	2.339	0.279	0.773	0.001196
Valve (Ø = 1.8)		2.54	7.859	3.151		

Head Loss

Q= 20.0 m^3/s

Inlet	Entrance	0.5 x 0.011	0.006
Transition	Contruction	0.1 (0.006 - 0.004)	0
H.S Tunnel	Friction	0.000009 x 150	0.001
Tunnel-Gate	Contruction Expantion	0.46 (0.204 - 0.006) 0.60 (0.204 - 0.010)	0.091 0.116
C. Tunnel	Friction	0.000020 x 235	0.005
Tunnel-Penstock	Contruction	0.45 (0.279 - 0.010)	0.121
Penstock	Friction	0.001196 x 480	0.574
"	Bend	0.41 (3.151 - 0.279)	1.178
Penstock-Valve	Contruction	0.38 x 3.151	1.197
Valve		0.3 x 0.279 x 2	0.167
Total			3.456

Table 14- 2- 3 Hydraulic Calculation

$Q = 15.0\ m^3/s$

Section	n	A	V	hr	R	$S_f = \frac{V^2 \cdot n^2}{R^{4/3}}$
Inlet (8.5x5.0)	0.015	42.50	0.353	0.006	1.831	0.000015
Transition (8.5x8.5)	0.015	72.25	0.208	0.002	2.125	0.000004
H.S Tunnel (D=8.5)	0.015	59.91	0.250	0.003	2.771	0.000005
E.G Gate (4.0x2.5)		10.00	1.500	0.115		
C Tunnel (Ø = 7.5)	0.015	44.18	0.340	0.006	2.312	0.000011
Penstock (Ø = 3.3)	0.013	8.55	1.754	0.157	0.773	0.000673
Valve (Ø = 1.8)		2.54	5.894	1.772		

Head Loss

$Q = 15.0\ m^3/s$

Inlet	Entrance	0.5 x 0.006	0.003
Transition	Contruction	0.1 x (0.003 - 0.002)	0
H.S Tunnel	Friction	0.000005 x 150	0.001
Tunnel-Gate	Contruction Expantion	0.46 (0.115 - 0.003) 0.60 (0.115 - 0.006)	0.052 0.065
C. Tunnel	Friction	0.000011 x 235	0.003
Tunnel-Penstock	Contruction	0.45 (0.157 x 0.006)	0.068
Penstock	Friction	0.000673 x 480	0.323
"	Bend	0.41 (1.772 - 0.157)	0.662
Penstock-Valve	Contruction	0.38 x 1.772	0.673
Valve		0.3 x 0.177 x 2	0.106
Total			1.955

Table 14- 2- 4 Hydraulic Calculation

Q= 10.0 m^3/s

Section	n	A	V	hr	R	$S_f=\frac{V^2 \cdot n^2}{R^{4/3}}$
Inlet (8.5x5.0)	0.015	42.50	0.235	0.003	1.831	0.000007
Transition (8.5x8.5)	0.015	72.25	0.138	0.001	2.125	0.000002
H.S Tunnel (D=8.5)	0.015	59.91	0.117	0.001	2.771	0.000002
E.G Gate (4.0x2.5)		10.00	1.000	0.051		
C Tunnel (Ø = 7.5)	0.015	44.18	0.226	0.003	2.312	0.000005
Penstock (Ø = 3.3)	0.013	8.55	1.176	0.071	0.773	0.000303
Valve (Ø = 1.8)		2.54	3.929	0.788		

Head Loss

Q= 10.0 m^3/s

Inlet	Entrance	0.5 x 0.003	0.002
Transition	Contruction	0.1 x (0.003 - 0.001)	0
H.S Tunnel	Friction	0.000007 x 150	0.001
Tunnel-Gate	Contruction Expantion	0.46 x (0.051 - 0.001) 0.60 x (0.051 - 0.003)	0.019 0.029
C. Tunnel	Friction	0.000002 x 235	0.001
Tunnel-Penstock	Contruction	0.45 (0.071 - 0.003)	0.031
Penstock	Friction	0.000303 x 480	0.145
"	Bend	0.41 (0.788 - 0.071)	0.294
Penstock-Valve	Contruction	0.38 x 0.788	0.299
Valve		0.3 x 0.0071 x 2	0.043
Total			0.864

Table 14- 2- 5 Hydraulic Calculation

Q= 9.1 m^3/s

Section	n	A	V	hr	R	$S_f=\frac{V^2 \cdot n^2}{R^{4/3}}$
Inlet (8.5x5.0)	0.015	42.50	0.214	0.002	1.831	0.000006
Transition (8.5x8.5)	0.015	72.25	0.126	0.001	2.125	0.000001
H.S Tunnel (D=8.5)	0.015	59.91	0.152	0.001	2.771	0.000002
E.G Gate (4.0x2.5)		10.00	0.910	0.042		
C Tunnel (Ø = 7.5)	0.015	44.18	0.206	0.002	2.312	0.000004
Penstock (Ø = 3.3)	0.013	8.55	1.064	0.058	0.773	0.000248
Valve (Ø = 1.8)		2.54	3.576	0.652		

Head Loss

Q= 9.1 m^3/s

Inlet	Entrance	0.5 x 0.002	0.001
Transition	Contruction	0.1 (0.002 - 0.001)	0
H.S Tunnel	Friction	0.000002 x 150	0
Tunnel-Gate	Contruction Expantion	0.46 x (0.042 - 0.001) 0.60 x (0.042 - 0.002)	0.019 0.024
C. Tunnel	Friction	0.000004 x 235	0.001
Tunnel-Penstock	Contruction	0.45 (0.058 - 0.002)	0.025
Penstock	Friction	0.000248 x 480	0.119
"	Bend	0.41 (0.652 - 0.058)	0.244
Penstock-Valve	Contruction	0.38 x 0.652	0.248
Valve		0.3 x 0.058 x 2	0.035
Total			0.716

Table 14- 2- 6 Hydraulic Calculation

$Q = 5.0\ m^3/s$

Section	n	A	V	hr	R	$S_f = \frac{V^2 \cdot n^2}{R^{4/3}}$
Inlet (8.5x5.0)	0.015	42.50	0.118	0.001	1.831	0.000002
Transition (8.5x8.5)	0.015	72.25	0.069	-	2.125	-
H.S Tunnel (D=8.5)	0.015	59.91	0.083	-	2.771	0.000001
E.G Gate (4.0x2.5)		10.00	0.500	0.013		
C Tunnel (Ø = 7.5)	0.015	44.18	0.113	0.001	2.312	0.000001
Penstock (Ø = 3.3)	0.013	8.55	0.585	0.017	0.773	0.000075
Valve (Ø = 1.8)		2.54	1.965	0.197		

Head Loss

$Q = 5.0\ m^3/s$

Inlet	Entrance	0.5 x 0.001	0.001
Transition	Contruction		0
H.S Tunnel	Friction	0.000001 x 150	0
Tunnel-Gate	Contruction Expantion	0.46 (0.013 - 0) 0.60 (0.013 - 0.001)	0.006 0.007
C. Tunnel	Friction	0.000001 x 235	0
Tunnel-Penstock	Contruction	0.45 (0.017 - 0.001)	0.007
Penstock	Friction	0.000075 x 480	0.036
"	Bend	0.41 (0.197 - 0.017)	0.074
Penstock-Valve	Contruction	0.38 x 0.197	0.075
Valve		0.3 x 0.017 x 2	0.010
Total			0.216

CHAPTER 15

IRRIGATION COMPONENT

15. Irrigation Project

15.1 Comparative Study of the Alternatives of the Driving Channels

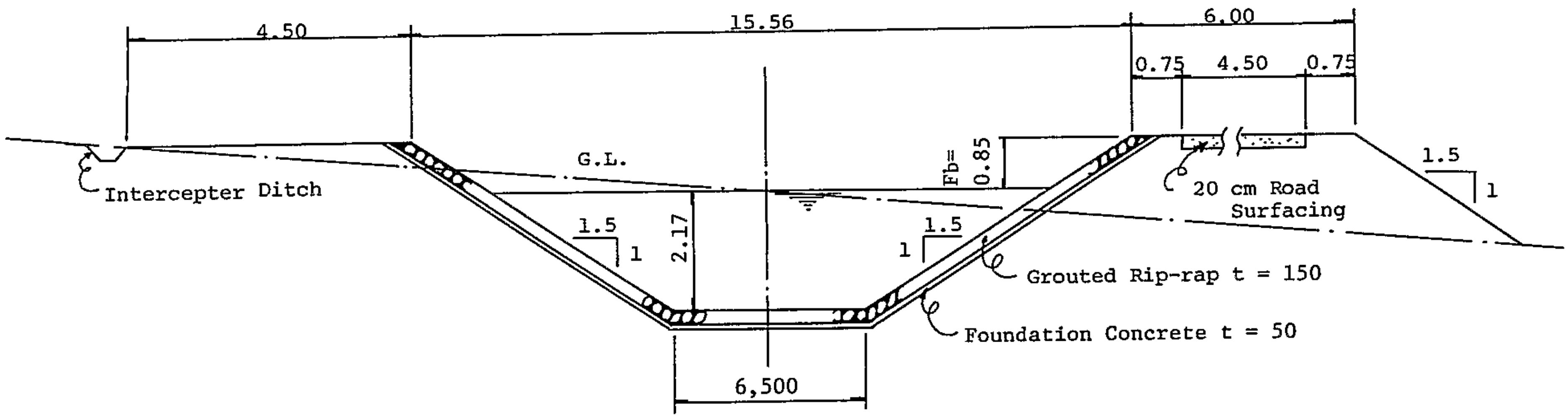
Table 15.1.1 Economical Comparison

	Length (m)	Unit Cost (US$/m)	Cost (US$)
[A] Open Channel Route			
Tunnel Part	750	1,850	1,387,500
Open Channel Part	1,600	250	400,000
Total	2,350		1,787,500
[B] Alternative Route (Tunnel Route)			
Tunnel Part	1,600	1,850	2,960,000
Open Channel Part	160	250	40,000
Total	1,760		3,000,000

15.2 Driving Channel

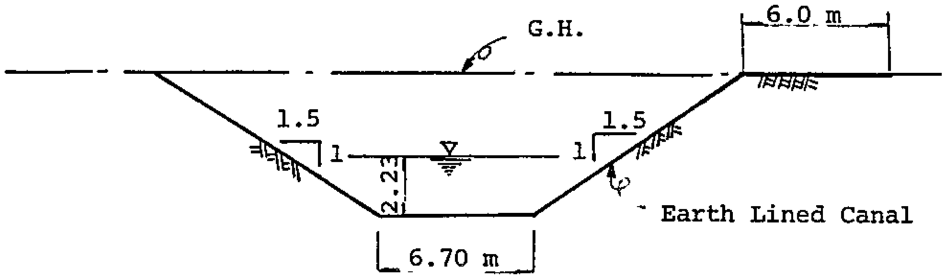
Fig. 15. 2. 1 TYPICAL CROSS SECTION OF DRIVING CHANNEL

$$\begin{cases} Q = 21.666 \ m^3/sec \\ V = 1.04 \ m/sec \\ I = 0.0004 \end{cases}$$



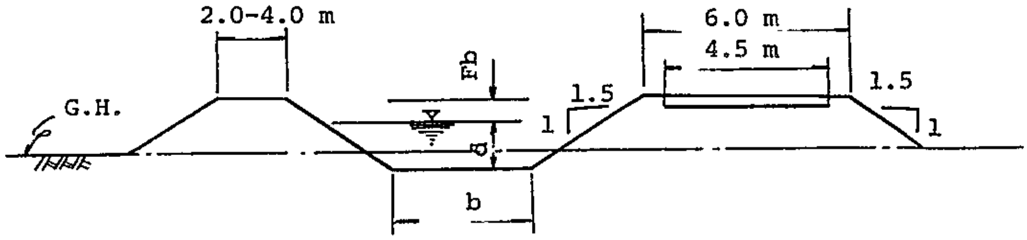
15.3 Main Canal & Lateral Canal

Main Canal Mo



West/East Main Canal

b = 2d $Q<4.0\ m^3/sec$
b = 2.5d $4<Q<9\ m^3/sec$
Fb = 0.4d



Lateral/Sublateral

Fb = 0.4d
b = 2d

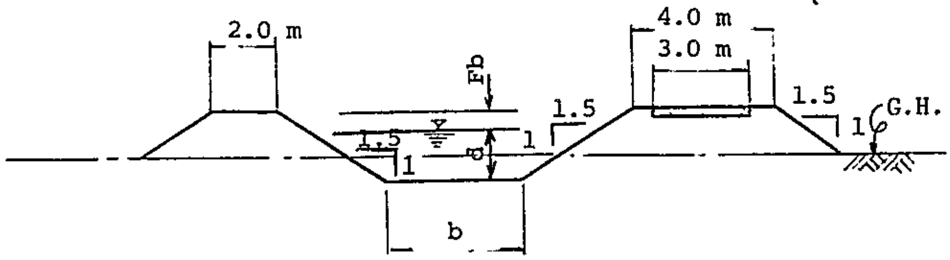
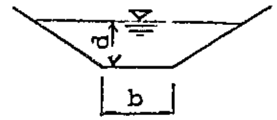


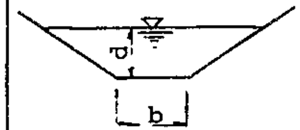
Fig. 15.3.1 Typical Cross Section of Canal

Table 15.3.1 The Approximate Calculation of Head Losses

	Distance (L)	Discharge (Q)	Velocity (V)	Slope of Water Surface	Head Loss (⊿h=I·L)	Water Surface Elevation	Canal Capacity		Remarks
	(km)	(m^3/sec)	(m/sec)	I	(m)	(m)	b(m)	d(m)	
Outlet of Tunnel						37.40			
	4.00	21.666	1.04	0.0004	1.60		6.50	2.17	
Mabini Turnout					0.15	35.80			
	3.00	20.535	0.42	0.0003	0.90		6.70	2.23	
Check Gate E_0					0.15	34.75 34.60			
	0.30	10.117	0.86	0.0004	0.12		4.90	1.63	
" E_1					0.15	34.48 34.33			
	3.70	7.998	0.90	0.0005	1.85		3.80	1.52	
" E_2					0.15	32.48 32.33			
	1.10	7.263	0.93	0.0006	0.66		3.50	1.40	
" E_3					0.15	31.67 31.52			
	2.10	6.378	0.89	0.0006	1.26		3.30	1.32	
" E_4					0.15	30.26 30.11			
	0.30	5.803	0.94	0.0007	0.21		3.20	1.28	
" E_5					0.15	29.90 29.75			
	1.50	5.153	0.94	0.0008	1.20		2.90	1.18	
" E_6					0.15	28.55 28.40			
	5.30	4.682	0.97	0.0009	4.77		2.80	1.12	
" E_7					0.15	23.63 23.48			

	Distance (L)	Discharge (Q)	Velocity (V)	Slope of Water Surface	Head Loss (⊿h=I·L)	Water Surface Elevation	Canal Capacity		Remarks
	(km)	(m^3/sec)	(m/sec)	I	(m)	(m)	b(m)	d(m)	
	0.30	3.947	0.94	0.0009	0.27		2.20	1.10	
" E_8					0.15	23.21 23.06			
	3.50	3.175	0.88	0.0009	3.15		2.00	1.00	
" E_9					0.15	19.91 19.88			
	2.50	1.827	0.80	0.0010	2.50	(16.15)	1.60	0.80	Drop
" E_{10}						13.65			

Table 15.3.2 The Approximate Calculation of Head Losses

	Distance (L)	Discharge (Q)	Velocity (V)	Slope of Water Surface	Head Loss (h=I·L)	Water Surface Elevation	Canal Capacity		Remarks
	(km)	(m^3/sec)	(m/sec)	I	(m)	(m)	b(m)	d(m)	
Outlet of Tunnel						37.40			
	4.00	21.666	1.04	0.0004	1.60		6.50	2.17	
Mabini Turnout					0.15	35.80 35.65			
	3.00	20.536	0.92	0.0003	0.90		6.70	2.23	
Check Gate W_0					0.15	34.75 34.60			
	2.00	10.419	0.87	0.0004	0.80		5.00	1.67	
" W_1					0.15	33.80 33.65			
	6.50	9.628	0.85	0.0004	2.60		4.80	1.60	
" W_2					0.15	31.05 30.90			
	0.50	9.232	0.84	0.0004	0.20		4.70	1.57	
" W_3					0.15	30.70 30.55			
	0.20	6.755	0.78	0.0004	0.08		3.70	1.48	
" W_4					0.15	30.47 30.32			
	0.30	5.596	0.76	0.0004	0.12		3.50	1.40	
" W_5					0.15	30.20 30.05			
	1.10	5.144	0.74	0.0004	0.44		3.40	1.36	
" W_6					0.15	29.61 29.46			
	5.90	4.861	0.78	0.0005	2.95		3.10	1.24	
" W_7					0.15	26.51 26.36			

	Distance (L)	Discharge (Q)	Velocity (V)	Slope of Water Surface	Head Loss (h=I·L)	Water Surface Elevation	Canal Capacity		Remarks
	(km)	(m^3/sec)	(m/sec)	I	(m)	(m)	b(m)	d(m)	
	1.00	3.702	0.74	0.0005	0.50		2.40	1.20	
" W_8					0.15	25.86 25.71			
	3.00	2.967	0.70	0.0005	1.50		2.20	1.10	
" W_9					0.15	24.21 24.06			
	1.80	2.383	0.72	0.0006	1.08		2.00	1.00	
" W_{10}					0.15	22.98 22.83			
	2.50	1.780	0.74	0.0008	2.00		1.70	0.85	
" W_{11}					0.15	20.83 20.68			
	1.50	1.573	0.71	0.0008	1.20		1.60	0.80	
" W_{12}					0.15	19.48 19.33			
	2.60	0.932	0.66	0.0009	2.34		1.30	0.65	
" W_{13}						16.99			

15.4 Related Canal Structures

Table 15.4.1 Estimated Number of Structures on Canal

Name of Canal	Canal Length (km)	Survice Area (Ha)	Numbers of Structure				
			Turnout	Checkgate	B=6m Bridge	Siphon	Culvert
Driving Channel	4.7				1		
Main Canal, Mo	3.0				1	1	-
Mabini Canal	10.0	600	12	5	2	-	-
West Main Canal	28.9	5,530	13	13	4	7	6
East Main Canal	20.6	5,370	10	8	2	2	4
Lateral Canal							
W1	6.0	420	8	3	-	-	3
W2	2.8	210	4	2	-	-	-
W3-1	2.9	300	6	2	1	-	-
W3-2	8.9	510	10	5	-	1	-
W3-3	2.2	240	5	2	-	-	-
W3-4	3.0	265	5	2	-	-	-
W4	8.0	615	12	4	1	-	-
W5	4.1	240	5	2	-	-	-
W6	3.5	150	3	2	-	-	-
W7	5.0	615	12	3	1	-	-
W8	3.0	390	8	2	-	-	-
W9	3.5	310	6	2	1	-	-
W10	3.0	320	6	2	1	-	-
W11	0.7	110	2	1	-	-	-
W12	3.5	340	7	2	1	-	-
W13-1	4.7	325	7	3	-	1	-
W13-2	1.1	170	4	1	-	-	-
Sub Total (ΣWi)			110	40	6	2	3
E1-1	5.7	525	11	3	2	-	-
E1-2	4.5	600	12	3	-	-	-
E2	5.3	390	8	3	2	-	-
E3	4.5	470	10	3	-	-	-
E4	3.5	305	6	2	-	-	-
E5	4.8	345	7	3	1	-	-
E6	4.0	250	5	2	-	-	2
E7	5.3	390	8	3	2	-	1
E8	3.4	410	8	2	1	-	-
E9-1	4.0	315	6	2	1	1	1
E9-2	4.0	400	8	2	2	-	-
E10-1	4.5	480	10	3	1	-	-
E10-2	3.3	290	6	2	-	-	-
E10-3	2.6	200	4	2	-	-	-
Sub Total (ΣEi)			109	35	12	1	4

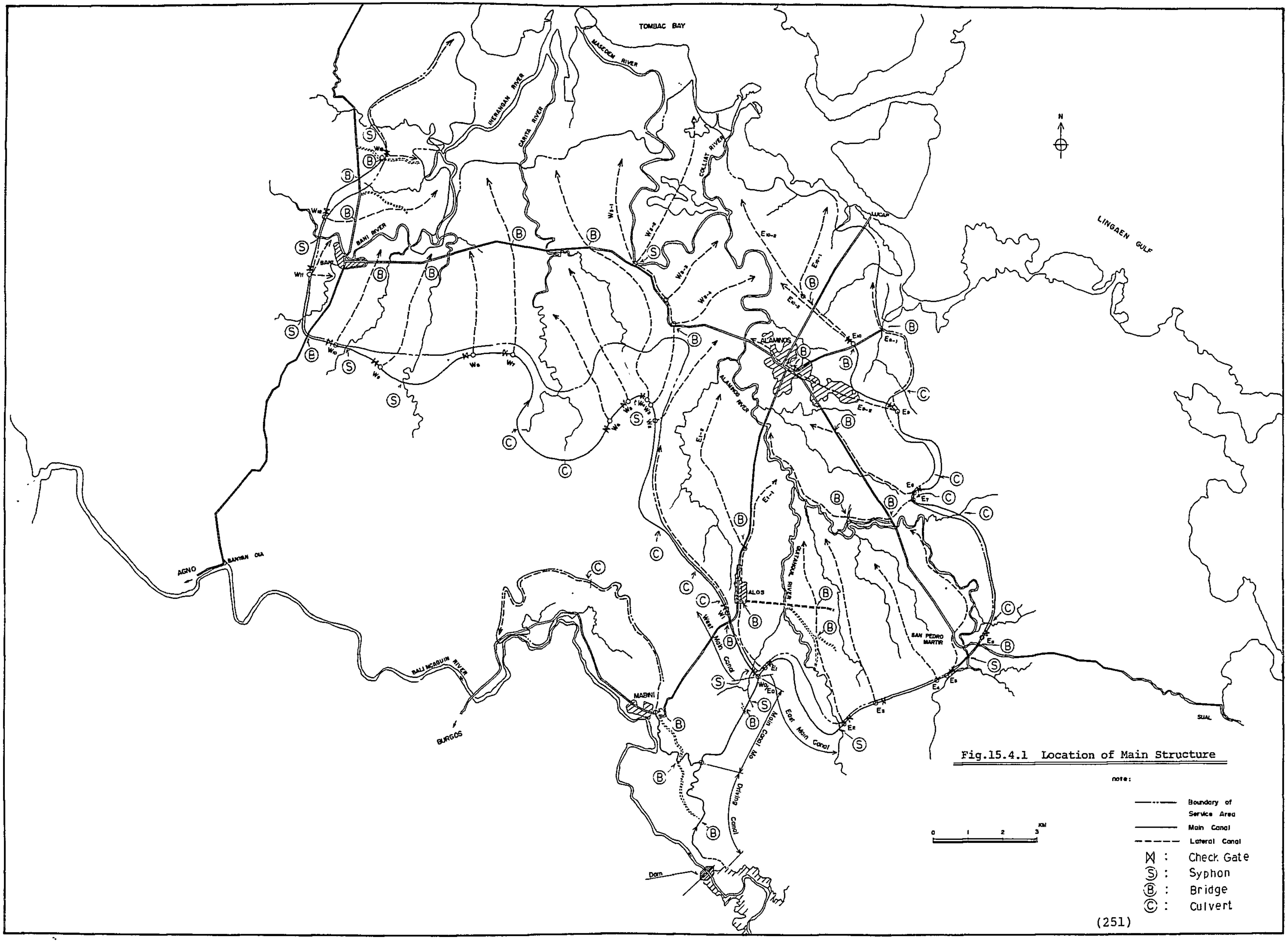
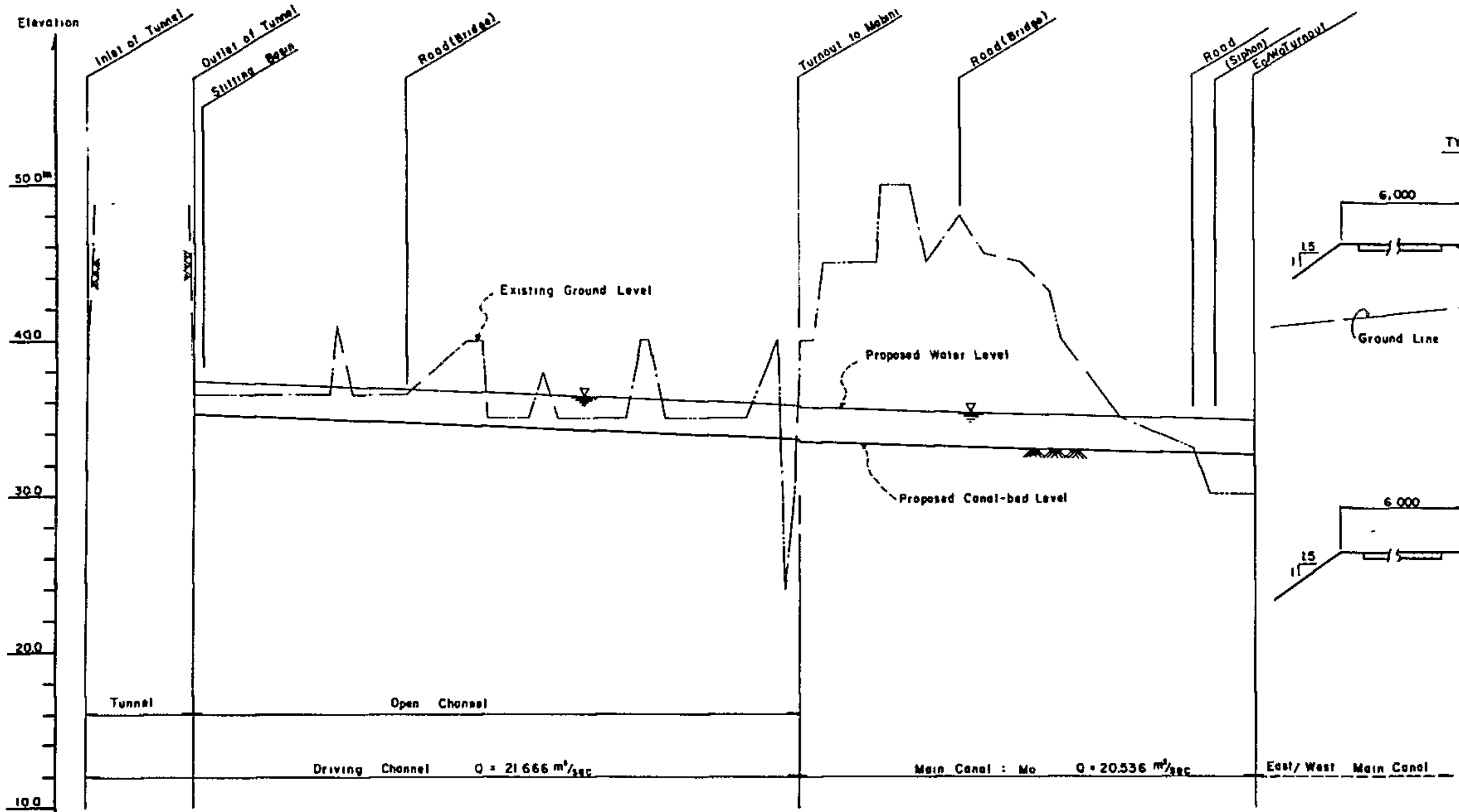
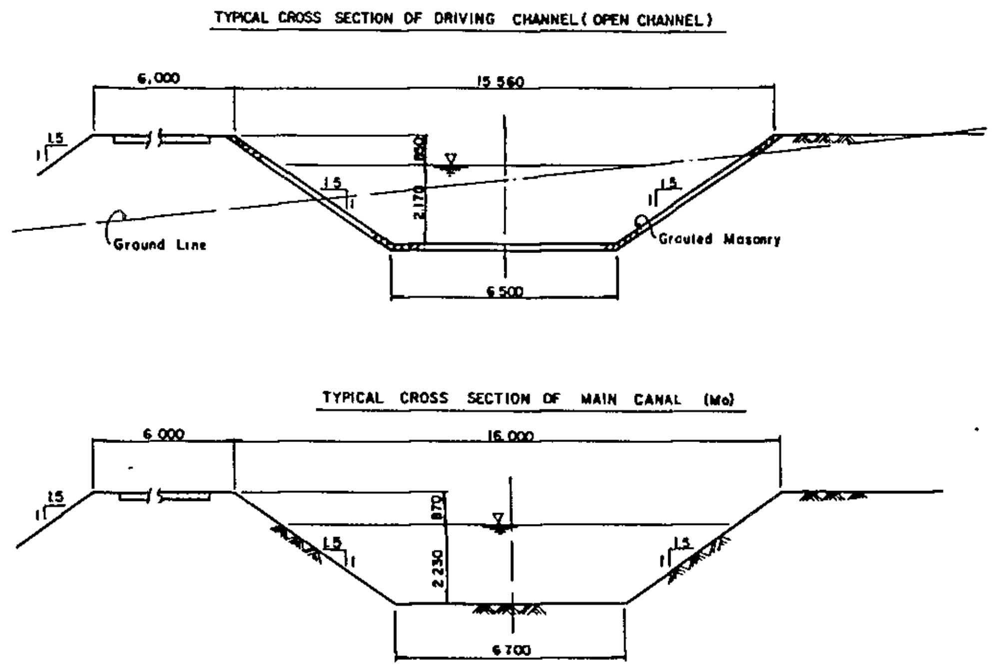


Fig.15.4.1 Location of Main Structure





LONGITUDINAL SLOPE							I = 0.0004								I = 0.0003				
PROPOSED LEVEL [illegible] (m)	38.00	37.57	37.40	37.28	37.08	36.88	36.68	36.48	36.28	36.08	35.88	(35.80 / 35.65	35.56	35.41	35.26	35.11	34.96	34.81	34.75
PROPOSED LEVEL CANAL BED (m)	35.83	35.40	35.23	35.11	34.91	34.71	34.51	34.31	34.11	33.91	33.71	(33.63 / 33.42	33.33	33.16	33.03	32.[illegible]	32.73	32.68	32.52
GROUND LEVEL (m)	38.0	135.0	36.5	36.5	36.5	36.5	40.0	38.0	35.0	35.0	38.8	40.0	45.0	46.8	45.4	39.2	34.2	30.0	30.0
ACCUMULATED DISTANCE (m)	0	500	700	1,000	1,500	2,000	2,500	3,000	3,500	4,000	4,500	4,700	5,000	5,500	6,000	6,500	7,000	7,500	7,700
DISTANCE (m)	0	500	200	300	500	500	500	500	500	500	500	200	300	500	500	500	500	500	200
STATION	NO 0	NO.5		NO10	NO15	NO20	NO25	NO30	NO35	NO40	NO45	.	NO50	NO55	NO60	NO65	NO70	NO75	NO77

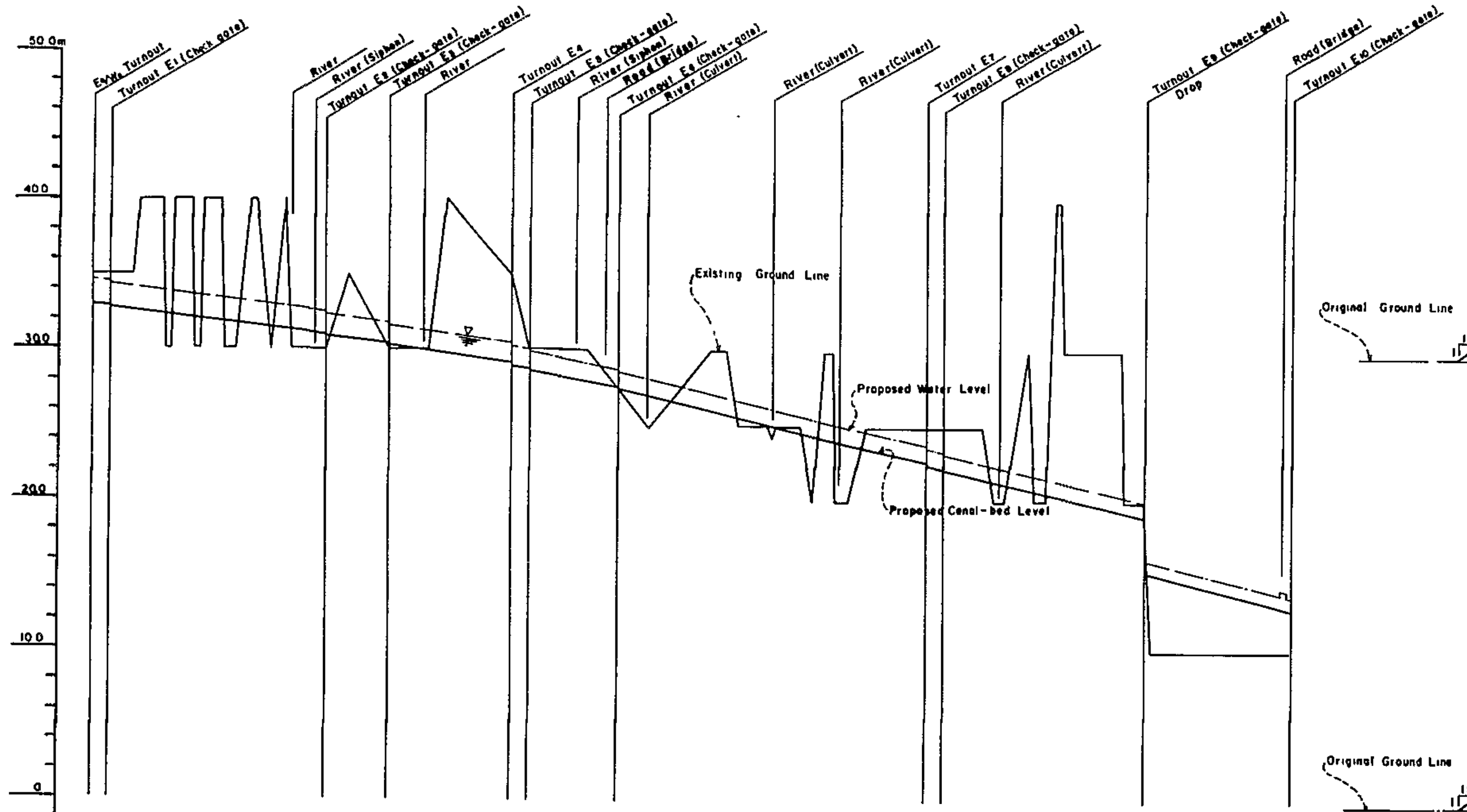
MABINI AGRICULTURAL DEVELOPMENT PROJECT

REPUBLIC OF THE PHILIPPINES

LONGITUDINAL SECTION OF DRIVING CHANNEL, MAIN CANAL (Mo)

Date MARCH 1982 | Fig. 15-4-2

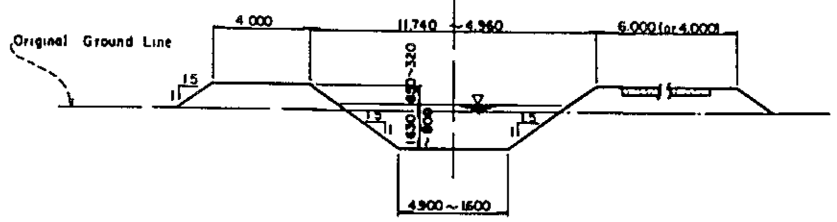
JAPAN INTERNATIONAL COOPERATION AGENCY



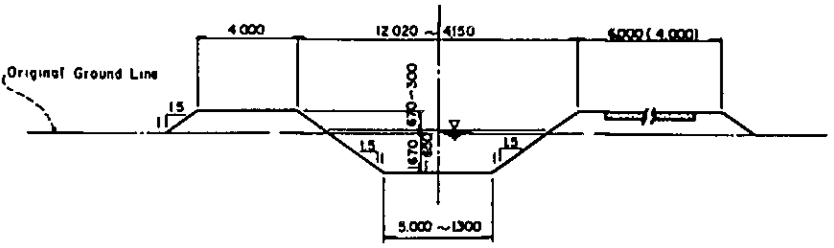
Longitudinal slope
I = 0.0004 · I = 0.0005 · I = 0.0006 · I = 0.0006 · I = 0.0007 · I = 0.0008 · I = 0.0009 · I = 0.0009 · I = 0.0009 · I = 0.0010

Station	Distance (m)	Accumulated distance (m)	Ground level (m)	Proposed level – Canal bed (m)	Proposed level – Water surface (m)
NO.0 / +300	0 / 300	0 / 300	35.0 / 35.0	32.97 (32.85 / 32.81)	34.60 (34.48 / 34.33)
NO 1	700	1,000	40.0	32.46	33.98
NO.2	1,000	2,000	40.0	31.96	33.48
NO.3	1,000	3,000	32.0	31.46	32.98
NO 4	1,000	4,000	30.0	(30.96 / 30.93)	(32.48 / 32.33)
NO 5	1,000 / 100	5,000 / 5,100	31.0 / 30.0	30.33 [illegible] (30.20)	31.73 [illegible] (31.52)
NO.6	900	6,000	37.0	29.66	30.98
NO.7	1,000 / 200 / 300	7,000 / 7,200 / 7,500	36.0 / 35.0 / 30.0	29.06 [illegible] (28.55)	30.38 [illegible]
NO 8	500	8,000	30.0	28.25	29.45
NO 9	1,000	9,000	28.0	(27.35 / 27.28)	(28.55 / 28.40)
NO10	1,000	10,000	23.0	26.38	27.50
NO11	1,000	11,000	27.0	25.48	26.60
NO12	1,000	12,000	25.0	24.58	25.70
NO13	1,000	13,000	20.0	23.68	24.80
NO14	1,000 / 300 / 300	14,000 / 14,300 / 14,600	25.0 / 25.0 / 25.0	22.78 (22.51 [illegible])	23.90 (23.63 [illegible])
NO15	400	15,000	25.0	21.70	22.70
NO16	1,000	16,000	27.0	20.80	21.80
NO17	1,000	17,000	30.0	19.90	20.90
NO 18 / +100	1,000 / 100	18,000 / 18,100	20.0 / 20.0	19.00 (18.91 [illegible])	20.00 (19.91 [illegible])
NO.19	900	19,000	10.0	14.45	15.25
NO 20	1,000	20,000	10.0	13.45	14.25
E. P	600	20,600	10.0	12.85	13.65

CROSS SECTION OF EAST MAIN CANAL



CROSS SECTION OF WEST MAIN CANAL



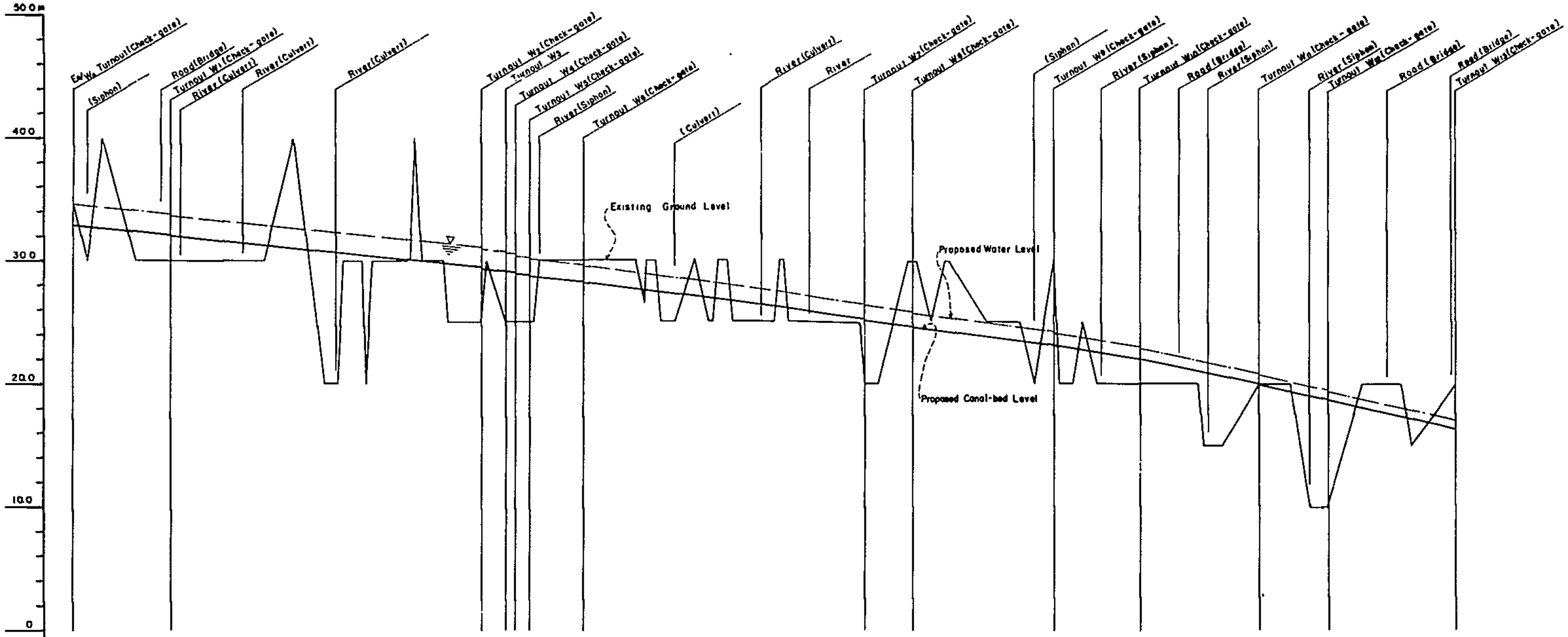
MABINI AGRICULTURAL DEVELOPMENT PROJECT

REPUBLIC OF THE PHILIPPINES

LONGITUDINAL SECTION OF EAST MAIN CANAL

Date MARCH 1982 | Fig 15-4-3

JAPAN INTERNATIONAL COOPERATION AGENCY



LONGITUDINAL SLOPE: I = 0.0004 (NO.0 – NO.2); I = 0.0004 (NO.2 – NO.8+500); I = 0.0004; I = 0.0004; I = 0.0004; I = 0.0004 (to NO.10+600); I = 0.0005; I = 0.0005; I = 0.0005; I = 0.0006; I = 0.0008; I = 0.0008; I = 0.0009 (to E.P.)

STATION	DISTANCE (m)	ACCUMULATED DISTANCE (m)	GROUND LEVEL (m)	PROPOSED LEVEL – CANAL BED (m)	PROPOSED LEVEL – WATER LEVEL (m)
NO.0	0	0	35.0	32.93	34.60
NO.1	1,000	1,000	34.0	32.53	34.20
NO.2	1,000	2,000	30.0	(32.13 / (32.05	(33.80 / (33.65
NO.3	1,000	3,000	30.0	31.65	33.25
NO.4	1,000	4,000	30.0	31.25	32.85
NO.5	1,000	5,000	28.0	30.85	32.45
NO.6	1,000	6,000	30.0	30.45	32.05
NO.7	1,000	7,000	30.0	30.05	31.65
NO.8	1,000	8,000	25.0	29.65	31.25
+500	500	8,500	25.0	(29.45 / (29.33	(31.05 / (30.90
NO.9	500	9,000	25.0	[illegible]	[illegible]
+200	200	9,200	25.0	[illegible]	[illegible]
+500	300	9,500	25.0	[illegible]	[illegible]
NO.10	500	10,000	30.0	28.49	29.85
+600	600	10,600	30.0	(28.25 / (28.22	(29.61 / [illegible]
NO.11	400	11,000	30.0	28.02	29.26
NO.12	1,000	12,000	30.0	27.52	28.76
NO.13	1,000	13,000	28.0	27.02	28.26
NO.14	1,000	14,000	25.0	26.52	27.76
NO.15	1,000	15,000	25.0	26.02	27.26
NO.16	1,000	16,000	25.0	25.52	26.76
+500	500	16,500	20.0	(25.27 / (25.16	(26.51 / (26.36
NO.17	500	17,000	23.0	24.91	26.11
+500	500	17,500	30.0	(24.86 / (24.61	(25.86 / (25.71
NO.18	500	18,000	27.0	24.36	25.46
NO.19	1,000	19,000	25.0	23.86	24.96
NO.20	1,000	20,000	22.0	23.36	24.46
+500	500	20,500	30.0	(23.11 / (23.06	(24.21 / (24.06
NO.21	500	21,000	23.0	22.76	23.76
NO.22	1,000	22,000	20.0	22.16	23.16
+300	300	22,300	20.0	(21.98 / (21.96	(22.98 / (22.83
NO.23	700	23,000	20.0	21.42	22.27
NO.24	1,000	24,000	15.0	20.62	21.47
+800	800	24,800	20.0	(19.98 / [illegible]	(20.83 / [illegible]
NO.25	300	25,000	20.0	19.72	20.32
NO.26	1,000	26,000	10.0	18.92	19.72
+300	300	26,300	10.0	(18.68 / (18.58	(19.48 / (19.33
NO.27	700	27,000	20.0	18.05	18.70
NO.28	1,000	28,000	15.0	17.15	17.80
E.P.	900	28,900	20.0	16.34	16.99

MABINI AGRICULTURAL DEVELOPMENT PROJECT

REPUBLIC OF THE PHILIPPINES

LONGITUDINAL SECTION OF WEST MAIN CANAL

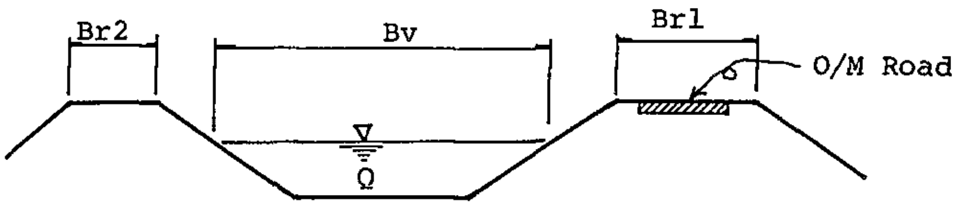
Date MARCH 1982 | Fig 15-4-4

JAPAN INTERNATIONAL COOPERATION AGENCY

15.5 Operation and Maintenance Road

Summary of Design Criteria For Operation and Maintenance Roads in NIA

1) Canal



(a) Br1 = 6.00^{M} in case of Bw 9.00^{M}

Br2 = 4.0^{M}

(b) Br1 = 4.0^{M} in case of Bw 9.0^{M}

Q 0.3 m^3/sec

(c) Br1 = 2.5 m in case of Q 0.3 m^3/sec

Where Br1: O/M Road Width

Br2: Other Side Bank

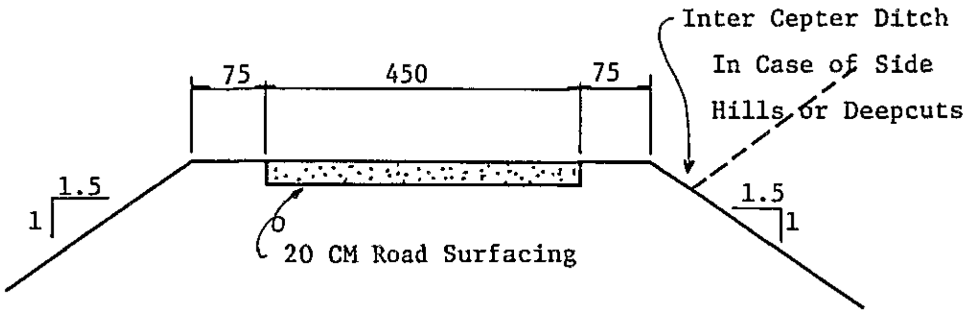
Bw : Normal Water Surface Width

(d) Generally the canal embankment with road surfacing shall be placed at the survice area side of the canal.

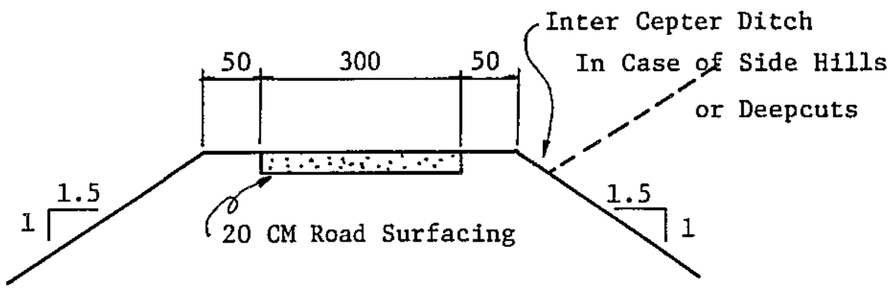
2) Operation Road

Fig. 15. 5. 1 Typical Sections of O & M Road

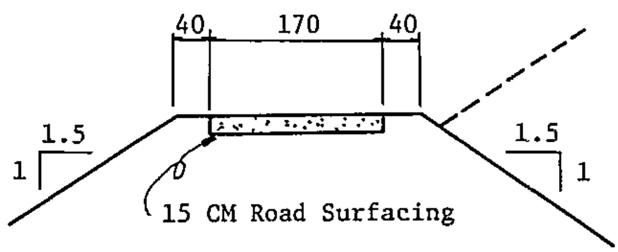
6.00 M ROADWAY



4.00 M ROADWAY



2.50 M ROADWAY



CHAPTER 16

CONSTRUCTION PLAN

16. Construction Plan

16.1 Construction Machinery

The chief items of machinery taken into consideration at the occasion of preparation of the plan for the implementation of the present project are listed in Table 17.1.1.

16.2 Diversion Tunnel

The half-face excavation method will be adopted for the construction of the diversion tunnel, because it has an inner diameter of 8.5 m. The schedule for the construction of the diversion tunnel by means of this method is shown in Figure 17.2.1. The third enlargement is started after completion of the arch lining. The works ranging from the half excavation to the arch timbering are planned to be carried out from the two sides of the tunnel, aiming at shortening the construction period. The cycle time will be as follows, by assuming 10 hours of work with 2 shifts.

Half face excavation	Cycles per team	1.25
	Excavation length	0.9 m
Third enlargement	Cycles per team	1.90
	Excavation length	2.0 m
Dobera excavation	Cycles per team	1.45
	Excavation length	4.5 m

The specifications of the forms are presented below.

Arch sliding form	10.5 m
Side wall sliding form	7.5 m
Side wall centre	6.0 (= 1.5 m x 4)

Table 16.1.1 List of Construction Machine

Slurry trench cutoff wall machine	2
Motor scraper 24 m	
Crawler type dozer 45 ton	
Bulldozer 21 ton	
Sheeps foot roller with Bulldozer 11 ton	
Ripper dozer 45 ton	
Tractor shovel side-dump bucket 1.6 m	2
Concrete mixer truck 3.2 m	
Power generating equipment 50-74 kW	
Crawler drill	
Crawler type tractor shovel 3.2 m	
Tractor shovel 2.3 m	
Dump truck 12 ton	
Vibration roller 6 ton	
Vibration roller 8-15 ton	
Back hoe 0.7-1.0 m	
Dump truck 8 ton	
Grout mixer and agitator	

① Half Face Excavation

② Arch Timbering

③ Arch Lining

④ Third Enlargement

⑤ Dobera Excavation

⑥ Side Wall Concrete

⑦ Invert Concrete

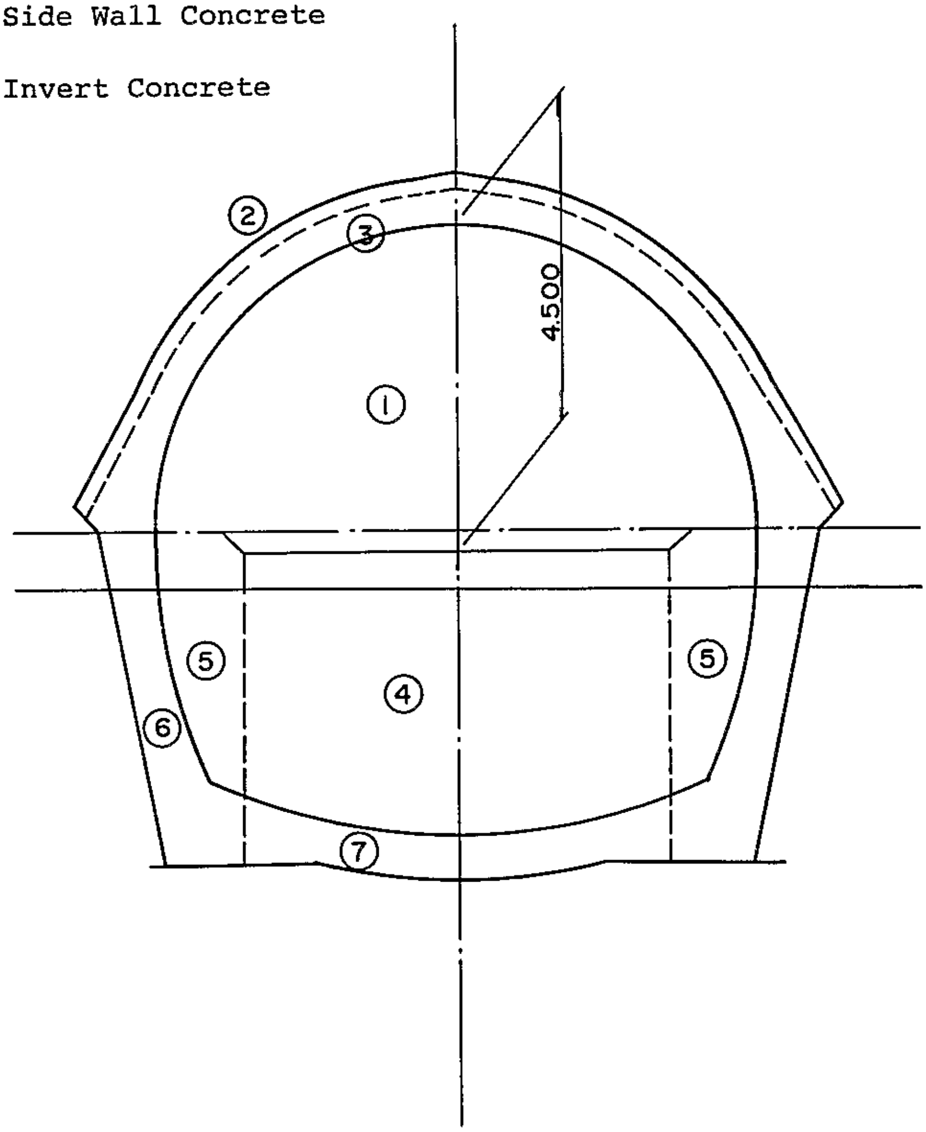


Fig.17.2.1 Half Face Excavation Method