2.2 Study of Alternative Plannings

The main point of the development planning of this project is to irrigate the area through out year, introducing double cropping pattern, by the construction of weir in Selagan river.

Compared with the development area, the quantity of water is quite abundant, but as the construction cost would be comparatively expensive in view of the topographical conditions and the existing features of the benefited area. Thus, the following alternative studies have been carried out.

Alternative - 1 : In the case of an intake without diversion dam in Selagan river.

Alternative - 2 : In the case of construction of weir in the most down-stream of Selagan river.

Alternative - 3 : In the case of construction of pump station in Selagan river.

Alternative - 4 : In the case of getting water resources from small dams in the branches of Selagan river.

Each location of the plannings is shown in the following location map.

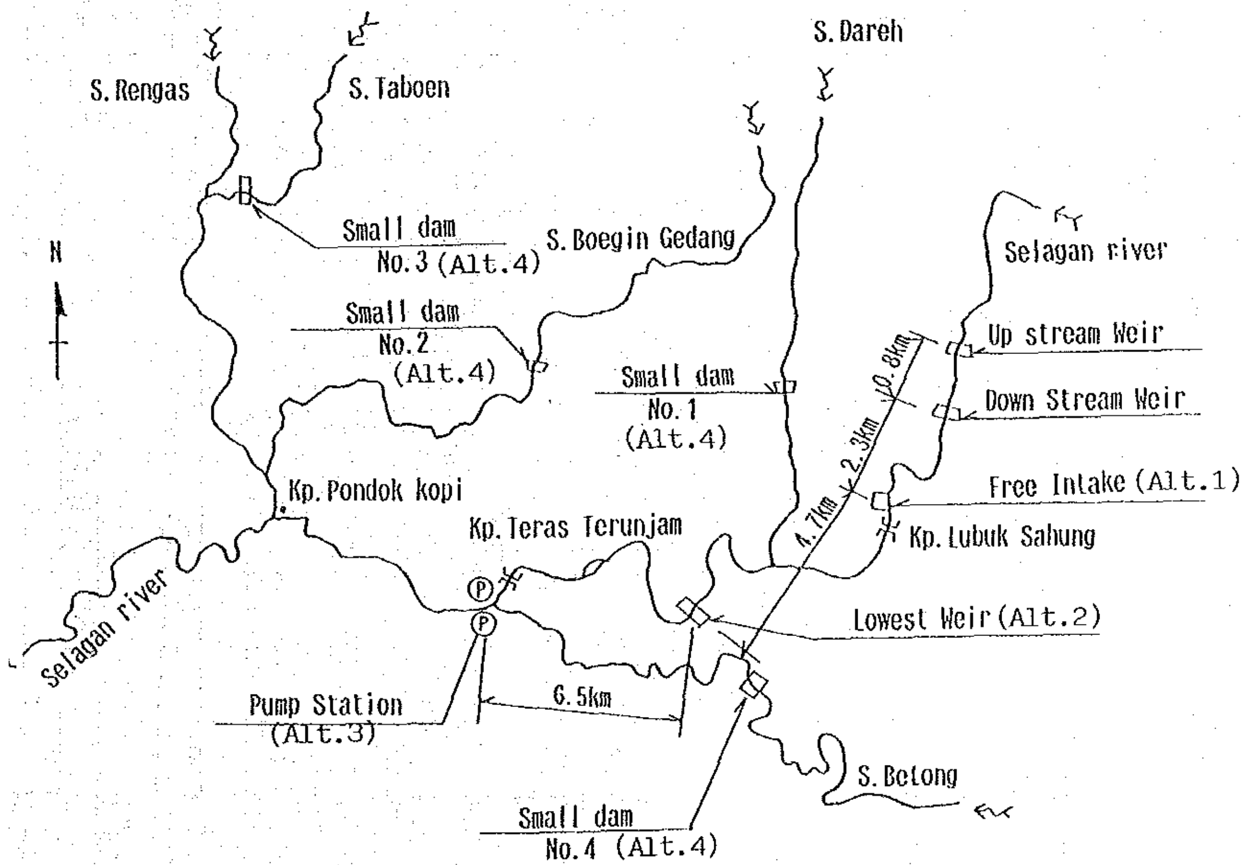
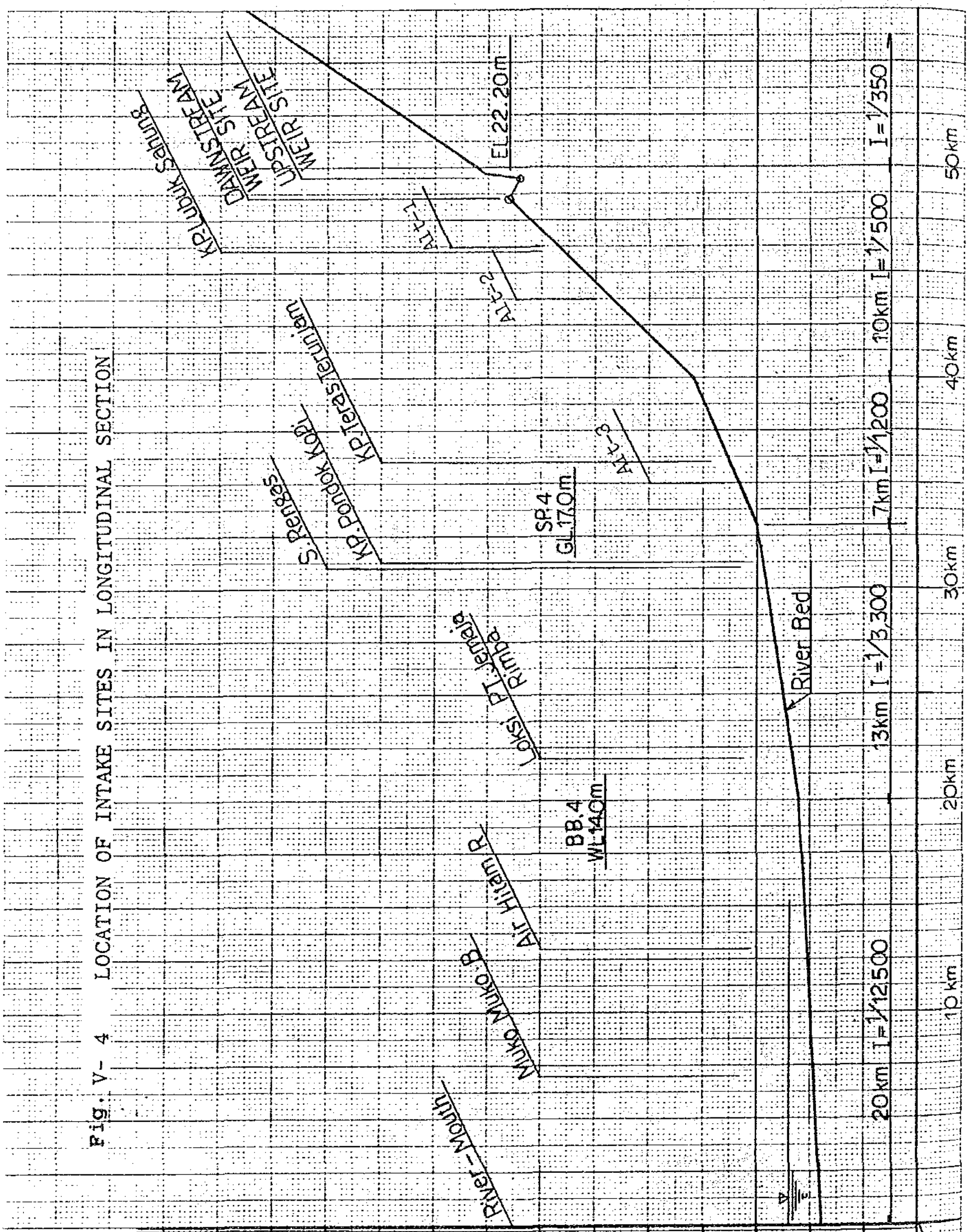


Fig. V- 3 LOCATION OF ALTERNATIVE INTAKE

Fig. V- 4 LOCATION OF INTAKE SITES IN LONGITUDINAL SECTION



2.2.1 The planning of Free intake without diversion dam in Selagan River (Alt.-1)

In the case of the intake without diversion dam, it is practically not possible to take the whole discharge of the river. As the intake capacity of the River is quite small during dry season, the irrigable scale would be much smaller compared with the case of constructing the weir.

With the above consideration, this planning is carried with the condition of constructing a small fixed concrete weir (about 1.0m in height) which is popular in the projects near-by. The location of the intake is selected with the conditions that the flow-line is smooth, the water route is stable, geological condition is good, river-bed is steady, it is easier to connect with the canal, the design discharge is stable for the intake and so on.

There is no such location to fulfill all the conditions. However, the proposed location is selected just the up-stream of Kp. Lubuk Sahung in view of stability of intake. The specific condition of the location is as follows.

a. It is the ending point of the meandering and the water is taken in the right bank where the water route goes straight.

b. There is the shallow bed-rock just in the downstream of the location.

c. The river width is wide (100 m in the width) and the shoal is developed in the center of the River.

d. The elevation of river-bed is EL.16.20m and the elevation of the benefited area is limited in low area.

e. There are houses just in the down-stream of Lubuk Sahung village, the sand blow-off canal is required to pass through the village.

f. The maintenance is not easy, because the back sand is easier to go into the intake from the water route.

g. It is necessary to have a fixed weir of about 1.0m in height through the study of Q-H curve of the cross-section of the existing river.

h. Possible quantity of water intake is about 20% of the river discharge. It is decided by the ratio between the width of intake and the same of the river as natural intake, even if the intake is effective, and irrigable area becomes smaller, and the cultivation ratio also lower in dry season.

As mentioned above, as it is difficult to ensure the design intake water, and to control the inflow of sand in this case, it is decided to be omitted from this planning. But, the approximate dimension in this case is shown as follows:

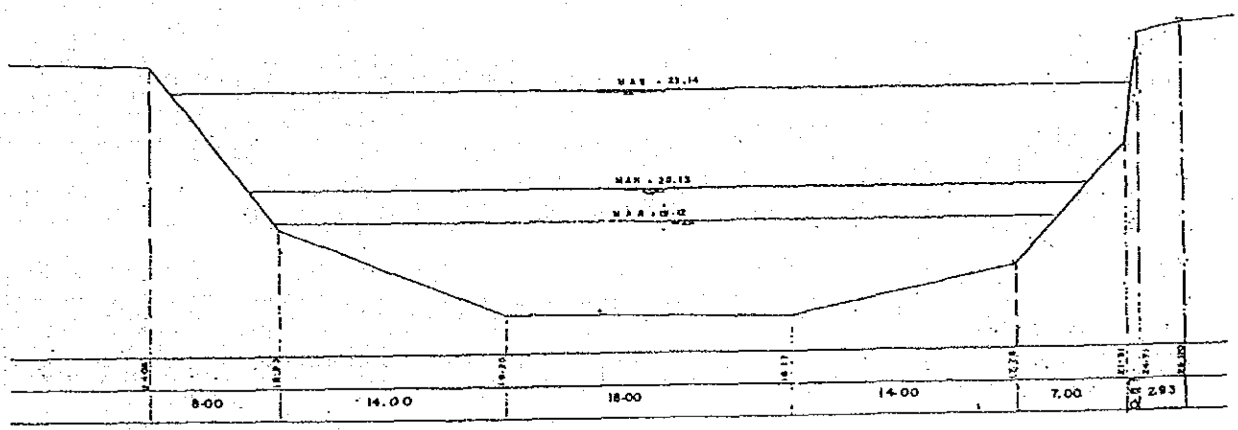
Table V-13 Approximate Dimension of Free Intake Plan

Item	Dimension
Location	Kp. Lubuk Sahung
Distance from the river mouth	46.4 Km
Width of existing river	100.0 m
Elevation of existing river-bed	EL.26.20 m
Slope of existing river-bed	1/540
Catchment area	396 Km^2
Design-flood discharge (one hundred year flood discharge probability)	1,056 m^3/sec
Raising height of water surface	1.00 m
Elevation of design intake bed	EL.17.20 m
Designed intake water level	WS 17.40 m
River discharge	
Rainy season (Jan. - May)	22.8 m^3/sec
Dry season (Jul. - Nov.)	9.7 m^3/sec
Probable quantity of water intake	20%
Designed water intake	
Rainy season	4.6 m^3/sec
Dry season	1.9 m^3/sec
Unit duty of water	
Rainy season	1.36 l/sec/ha
Dry season	1.53 l/sec/ha
Probable irrigation area	
Rainy season	3,300 ha
Dry season	1,200 ha
Cultivation ratio	136%

Fig. V-5 Q - H CURVE AT KP. LUBUK SAHUNG

River bed slope(I)= 1/550

X	Y	N
0.000	24.080	
		0.040
8.000	18.930	
		0.040
22.000	16.260	
		0.040
40.000	16.170	
		0.040
54.000	17.760	
		0.040
61.000	21.310	
		0.040
61.870	24.710	
		0.040
64.800	25.170	
		0.000



Q - H CURVE

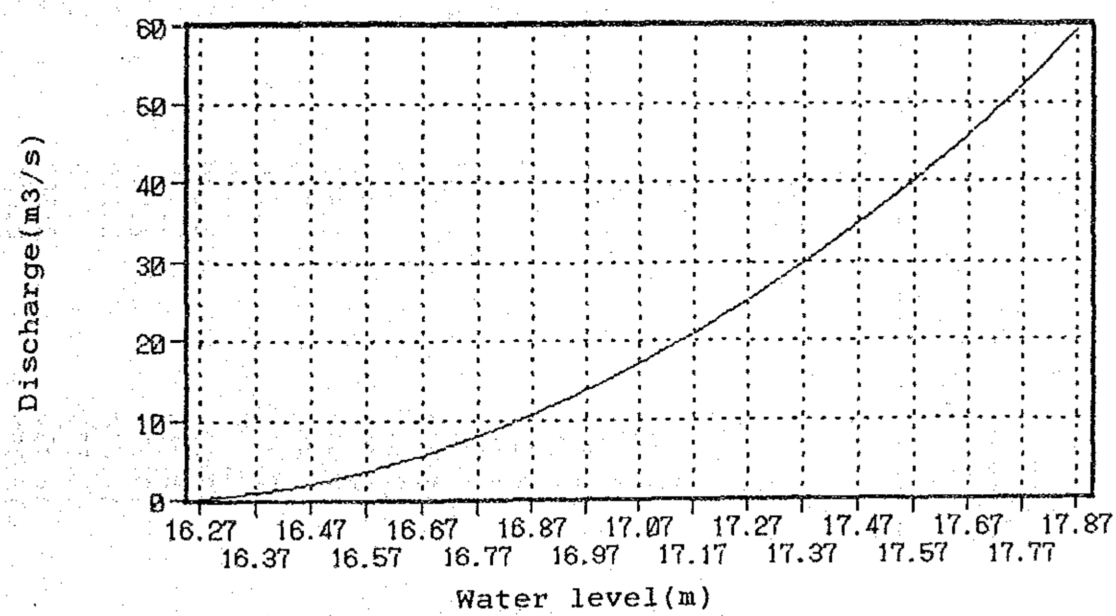


Table V-14 DISCHARGE CALCULATION AT KP. LUBUK SAHUNG

Water level	Water depth	Area	Wetted Perimeter	Hydraulic Radius	Roughness Coefficient	Velocity	Discharge
(m)	(m)	(m^2)	(m)	(m)		(m/s)	(m^3/s)
16.270	0.100	1.034	18.940	0.055	0.040	0.153	0.159
16.370	0.200	2.998	20.360	0.147	0.040	0.297	0.891
16.470	0.300	5.102	21.780	0.234	0.040	0.405	2.067
16.570	0.400	7.346	23.200	0.317	0.040	0.495	3.638
16.670	0.500	9.731	24.620	0.395	0.040	0.574	5.587
16.770	0.600	12.257	26.040	0.471	0.040	0.645	7.906
16.870	0.700	14.923	27.460	0.543	0.040	0.710	10.594
16.970	0.800	17.729	28.880	0.614	0.040	0.770	13,652
17.070	0.900	20.676	30.299	0.682	0.040	0.826	17.084
17.170	1.000	23.764	31.719	0.749	0.040	0.879	20.896
17.270	1.100	26.992	33.139	0.814	0.040	0.930	25.095
17.370	1.200	30.360	34.559	0.878	0.040	0.978	29.686
17.470	1.300	33.869	35.979	0.941	0.040	1.024	34.679
17.570	1.400	37.518	37.399	1.003	0.040	1.068	40.080
17.670	1.500	41.308	38.819	1.064	0.040	1.111	45.898
17.770	1.600	45.238	40.173	1.126	0.040	1.154	52.198
17.870	1.700	49.268	40.928	1.204	0.040	1.206	59.433
17.970	1.800	53.370	41.682	1.280	0.040	1.256	67.084
18.070	1.900	57.544	42.437	1.356	0.040	1.306	75.150
18.170	2.000	61.790	43.192	1.431	0.040	1.353	83.630
18.270	2.100	66.109	43.947	1.504	0.040	1.400	92.521
18.370	2.200	70.499	44.702	1.577	0.040	1.444	101.825
18.470	2.300	74.962	45.457	1.649	0.040	1.488	111.541
18.570	2.400	79.497	46.212	1.720	0.040	1.530	121.669
18.670	2.500	84.104	46.967	1.791	0.040	1.572	132.210
18.770	2.600	88.783	47.722	1.860	0.040	1.613	143.166
18.870	2.700	93.534	48.476	1.929	0.040	1.652	154.536

2.2.2 The planning of construction of weir in the most down-stream of Selagan River (Alt.-2)

In this case, the location of weir is selected in the nearest point from the benefited area (6.5 Km up-stream from Kp. Teras Terunjam).

The specific character of the location can be estimated as follows:

a. The river-bed can be 8.5m lower in elevation.

b. The elevation of water intake can be WS 16.40m as the maximum back water of weir can be 8.0m by the double closing system.

c. As the elevation of water intake is low, the benefited area can be as small as Alternative-1 compared with the other alternatives.

d. Cultivation ratio can be 200%.

e. The structural scale is bigger with 8.0 m of raising water height and 100m in the width. Thus, though construction cost of the weir is expensive, total construction cost is rather cheaper as the irrigated area is smaller. But the unit cost per benefited area is higher and the investment ratio is low.

f. The main benefited area is not possible to be irrigated. (Over the elevation of 19.0m in SP-IV).

As the unit cost ratio is higher than the other alternatives, this planning is canceled.
(Unit construction : 1.07)

The approximate dimension in this case is shown as follows:

Table V-15 Approximate Dimension of the most down-stream Weir Plan

Item	Dimension
Location	6.5 Km up-stream from Kp. Teras Terunjam
Width of existing river-bed	100.0 m
Elevation of existing river-bed	GH 8.50 m
Slope of existing river	1/500
Catchment area	418 Km^2
Design-flood discharge	1,115 m^3/sec
Raising height of water surface	8.00 m
Designed intake water level	WS 16.40 m
Elevation of the benefited area (SP-IV)	GH 11.20 m
Actual irrigable area	About 3,300 ha
Main canal	24.5 Km
Secondary canal	31.2 Km
Cultivation ratio	200%

Table V-16 COMPARISON OF APPROXIMATE CONSTRUCTION COST IN EACH LOCATION OF INTAKE

Kind of Construction Works	Unit	Unit Cost	Plan-3		Plan-3		Alt-1		Alt-2	
			Weir in Up-stream		Weir in Down-stream		Gravity Intake		Weir in the Most Down-stream	
		10x3Rp.	Quantity	Cost	Quantity	Cost	Quantity	Cost	Quantity	Cost
				Mill.Rp		Mill.Rp		Mill.Rp		Mill.Rp
1. Weir	m^3	253	16,400	4,149	15,700	3,972	6,000	1,518	22,200	5,617
2. Main Canal										
Up-stream	m	900	800	720	-	-	-	-	-	-
Middle-stream	m	600	14,600	8,760	14,600	8,760	12,300	7,380	7,600	4,560
Down-stream	m	495	14,500	7,178	14,500	7,178	14,500	7,178	14,500	7,178
Sub-total				16,658		15,938		14,558		11.738
3. Secondary Canal	m	332	39,700	13,180	39,700	13,180	31,200	10,358	31,200	10,358
4. Tertiary Canal	ha	1,000	4,200	3,150	4,200	3,150	3,300	2,475	3,300	2,475
Total				37,137		36,240		28,909		30,188
Cost Ratio of Whole Works				1.02		1.00		0.80		0.83
Cost per Hectare		US$/ha		4,792		4,677		4,748		4,958
Cost Ratio per Hectare				1.02		1.00		1.02		1.07

Adopted

2.2.3 The planning of construction of pump station (Alt.3)

It is examined to apply to construction of pump station in the down-stream without depending on the gravity irrigation.

1) Selection of the location for the pump station

The location is selected with the consideration of the following matters.

- The location where the main canal is the shortest in economic view.
- The location where the pump station can be set in higher place to avoid the influence of the flood.
- The location to avoid the tidal influence.
- The location where is nearer from the existing road.
- The location where it is smoother for the connection with the main canal by pipe line.

Due consideration of the above matters, it is planned to set two (2) pump stations on the both banks in 800m down-stream from Kp. Teras Terunjam.

2) Dimension of Pump

i) Pump Station at Right Bank

a. Gross Pump Head Ha = 16.6m

Control Elevation (B.B.4)	:	EL.14.00m
Length of Main Canal	:	12.5Km
Average Slope of Main Canal	:	1/2,500

b. Total Pump Head H = 18.5m

Length of Pipe-line	:	L = 1,000m
Maximum Pumping Water	:	Q = 2.57m^3/sec (A = 1,680ha)
Diameter of Delivery Pipe	:	ø 1,500mm

c. Selection of Pump

Unit Requirement of Pump Water	:	Q = 2.57/3 = 51m/min/pump
Mixed Flow Pump (High Head and Vertical Shaft Type)	:	ø 700 x 3nos.
Generating Power for Motor	:	260Kw x 3nos.

ii) Pump Station at Left Bank

a. Gross Pump Head Ha = 18.6m

Control Elevation (SP-IV)	: EL.19.0m
Length of Main Canal	: 6.2Km
Average Slope of Main Canal	: 1/2,500

b. Total Pump Head H = 20.4m

Length of Pipe-line	: L = 800m
Maximum Pumping Water	: Q = 3.46m^3/sec (A = 2,260ha)
Diameter of Delivery Pipe	: ø 1,650mm

c. Selection of Pump

Unit Requirement of Pump Water	: Q = 69m^3/min/pump
Mixed Flow Pump (High Head and Vertical Shaft Type)	: ø 800 x 3nos.
Generating Power for Motor	: 370Kw x 3nos.

3) Silt Basin and Suction Tank

It is necessary to build a silt basin in front of a suction tank, as the Plan is to take the intake water from natural river, and for the suction tank to have the type that the motors are not inundated by flood water.

4) Construction Method

The Pump Station is planned to construct by Copure Method at the right bank of existing meander utilizing the meander part of the River.

5) Outline of the pump stations

	Left bank	Right bank
a) Designed duty of water	3.46m^3/sec (208m^3/min)	2.57m^3/sec (154m^3/min)
b) Kind of the pump	High head mixed flow pump with vertical shaft	The same
c) Total pump head	20.4m	18.5m
d) Diameter & number of pumps	ø800mm x 3nos.	ø700mm x 3nos.
e) Horse power of engine	503 Ps x 3Nos.	354 Ps x 3Nos.
f) Form of station house	Double stories	The same
g) Method of construction	Copure method	The same

6) Approximate Construction Cost

Table V-17 APPROXIMATE CONSTRUCTION COST OF PUMP PLAN

Unit : Rp.1,000

	Left Bank	Right Bank	Total
a) Pump Station			
i) Civil Works	135,600	158,400	294,000
ii) Earth Works by Copure Method	2,890,000		2,890,000
iii) Pump House	261,900	306,000	567,000
iv) Equipment Cost	3,266,000	2,613,000	5,630,900
Sub-total			9,630,900
b) Main Canal			
i) Pipe-line	1,800m x 15,000 Rp/m		27,000
ii) Middle-Stream	5,000m x 414,000 Rp/m		2,070,000
iii) Down-Stream	17,000m x 332,000 Rp/m		5,644,000
Sub-total			7,741,000
c) Secondary Canal	37,200m x 332,000 Rp/m		12,350,400
d) Tertiary Canal	3,940ha x 750,000 Rp/ha		2,955,000
Total			32,677,300

7) Fuel Cost for Pump Operation

Pump operation cost is estimated by the following formula.

$$Q = P_E \cdot B_E \cdot 1/45 \ (l/hr)$$

where,

P_E : Pump Horse Power
B_E : Consumption Ration of Fuel (0.22)
rt : Specific Gravity of Fuel (0.85 kg/l)

The fuel consumption per hour for the maximum irrigation requirement is as follows.

Right Bank Qk : 354 x 0.22 x 1/0.85 x 3nos. = 275 l/hr
Left Bank QL : 503 x 0.22 x 1/0.85 x 3nos. = 391 l/hr
Total = 666 l/hr

In the other hand, the annual water requirement is estimated 21,663 m^3/ha/year based on the proposed cropping pattern.

As the cultivation area is 3,940 ha in this pump plan, the annual total water requirement (Q) is as follows.

$$Q = 21,663 \times 3,940 \text{ ha} = 85,352,200 \text{ m}^3$$

It becomes 45% of 190,105,300 m^3 which is the annual total water quantity calculated by the maximum irrigation requirement all through a year.

Thus, the quantity of annual fuel consumption is estimated as follows.

666 l/hr x 24 ha x 365 days x 0.45 = 2,625,000 l/year

And the annual cost is,

2,625,000 l/year x Rp.240/l = Rp.630,000,000.

8) Economic Comparison with Weir Plan

For the economic comparison with Weir Plan, the cost of Pump Plan is converted to the annual cost because the Pump Plan requires the fuel cost for operation and has difference upon the durable period.

Durable periods for the equipment and the structure are assumed as follows.

Motor for Pump	20 years	Irrigable area (Weir Plan) 4,200 ha
Gate	30 years	Irrigable area (Pump Plan) 3,940 ha
Civil Structure Works	50 years	

Table V-18 COMPARISON OF WEIR & PUMP COST

Unit : Rp.1,000

Construction Works	Weir Plan		Pump Plan	
	Const.Cost	Annual Cost	Const.Cost	Annual Cost
I. Weir				
I-1 Weir & Intake	3,472,842	69,457	-	-
I-2 Gate	499,950	16,665	-	-
II. Pump Station				
II-1 Civil Works	-	-	294,000	5,880
II-2 Canal Works	-	-	2,890,000	57,800
II-3 Pump House	-	-	567,900	11,358
II-4 Equipment	-	-	5,879,000	293,950
II-5 Fuel Cost	-	-	-	644,160
III. Main Canal				
III-1 Canal	15,828,000	316,560	7,637,810	152,756
III-2 Attached Gates	110,000	3,667	103,190	3,440
IV. Secondary Canal				
IV-1 Canal	13,080,000	261,600	12,256,590	245,132
IV-2 Attached Gates	100,000	3,333	93,810	3,127
V. Tertiary System	3,150,000	63,000	2,955,000	59,100
Total	36,240,792	734,282	32,677,300	1,476,703
Cost per Hectare	8,629		8,294	
Annual Cost Ratio		1.00		2.01

As the result of the above study. Annual Cost of Pump Plan is about double from the same of Weir Plan and Pump Plan is not applied.

2.2.4 Planning of a group of small dams (Alt.-4)

It is examined for the planning to ensure the water recourses from a group of small dams near the benefited area for economizing the construction cost of the water resource facilities and the canals. In this case, there is no intake water from the main river of Selagan river.

It is possible to select the locations in Rengas River for the right-side benefited area, and in Betong river for the left-side benefited area.

1) Plan of a group of small dams

Elevational control points for the location are WL 14.0m in water level at BB.4, SP-VI in the transmigrated area for the right-bank benefited area, and GH 19.0m in the ground height at SP-IV for the left bank benefited area.

Each independent river system is required for the construction of small dams since the benefited areas are distributed in the both banks of the Selagan River.

Actual paddy cultivation areas are 1,800 ha in the right bank and 2,400 ha in the left bank of the Selagan River. Hence, it is necessary to construct three (3) small dams in the right bank and one (1) small dam in the left bank as those water resources.

The dimension of each dam is as follows:

Table V-19 APPROXIMATE DIMENSION OF SMALL DAMS

No.	Location	Name of River	Catchment area	El. of crest of dam	Length of crest	Height of dam	Embankment volume
			km^2	EL m	m	m	m^3
1	Right-side	S. Dereh	11	28.0	100	6	20,000
2	Right-side	S. Boegingedang	19	25.0	100	6	20,000
3	Right-side	S. Taboen	35	23.0	200	8	50,000
4	Left-side	S. Betoeng	66	25.0	300	15	120,000

2) Approximate Water Balance

i) Right Bank Side

- Benefited Area : 1,800 ha
- Annual Cultivation Ratio : 200%
- Annual Storage Capacity for unit Catchment Area: 500,000 m^3/km^2
- Unit Water Requirement
  - Rainy Season : 1.36 l/s/ha] 21,663 m^3/ha Annual Water Requirement
  - Dry Season : 1.53 l/s/ha
- Irrigation Period
  - Rainy Season : 105 days
  - Dry Season : 105 days

Total annual storage capacity of three (3) days by the above condition is,

$$V = 500{,}000\ m^3/km^2 \times (11 + 19 + 35\ Km^2) = 32{,}500{,}000\ m^3$$

Annual Water Requirement:

$$Q = 21{,}663 \times 1{,}800ha = 38{,}993{,}400\ m^3$$

Accordingly, about 17% of water scarcity is assumed by the resources of right-bank side

ii) Left Bank Side

- Benefited area : 2,400 ha
- The other conditions are the same with the Right Bank Side

o Annual Storage Capacity

$$V = 500{,}000\ m^3/Km^2 \times 66\ Km^2 = 33{,}000{,}000\ m^3$$

o Annual Water Requirement

$$Q = 21{,}663\ m^3/ha \times 2{,}400\ ha = 51{,}991{,}200\ m^3$$

Hence, about 37% of water scarcity is assumed by the resources of right-bank side.

3) Approximate Construction Cost

Table V-20 APPROXIMATE CONSTRUCTION COST

Construction Works	Unit	Appr. Quantity	Unit Cost	Cost	Remarks
			Rp.	Rp.1,000	
1. Dam					
1-1 Dam Body	m^3	210,000	12,000	2,520,000	for 4 dams.
1-2 Intake	m^3	4,500	310,000	1,395,000	"
Sub-total				3,915,000	
2. Main Canal					
2-1 Down-Stream	m	10,500	332,000	3,486,000	
2-2 Middle-Stream	m	21,000	444,000	8,694,000	
2-3 Leading Canal	m	8,000	495,000	3,960,000	
Sub-total				16,140,000	
3. Secondary Canal	m	39,700	332,000	13,180,400	
4. Tertiary System	ha	4,200	750,000	3,150,000	
Total				36,385,400	

4) Conclusion

Specific character of this plan is as follows:

- It is unavoidable to reduce the benefited area due to the scarcity of the quantity of river discharge in the both banks and cultivation ratio is estimated as 143%.
- There occurs 37 percent of the scarcity of water in the left bank, and it has to be constructed a dam or reservoir inside of Concession area.
- Diversion works for the dams and canals will be in many numbers and it is complicated for the operation and maintenance.
- It is not clear to say that the construction cost of this plan is cheaper than the plan of construction of weir.

2.3 Study on Canal Factors for Muko-Muko Left Bank Area

Based on the data listed below, the study on canal factors for Muko-Muko left bank area irrigation development including the effective use of the existing secondary canal (S.S. Baru) in Air Selagan project area will be analyzed :

a) Laporan/Studi Analisa Kebutuhan Dan Keseimbangan Air/Wilayah Sungai Air Manjuto Kanan/Propinsi Bengkulu, 1985/86

b) Gambar/Skema Sub Proyek Irigasi Muko-Muko

c) Booklet/Proyek Irigasi Muko-Muko/Propinsi Bengkulu

d) Second Provincial Irrigation Development Project/ANNEX 1/Action Plan for Muko-Muko Scheme/Executive Summary

e) Gambar/Kerja Sub Proyek Irigasi Muko-Muko/Paket /XIII, XIV, XVI, XVII, XIX

(1) Target Area to be developed

Muko-Muko left bank area	:	For paddy	:	6,768 ha
Muko-Muko right bank area	:	"	:	4,919 ha
Silauto area	:	"	:	5,000 ha
Total				16,687 ha

As mentioned below in detail, the above each area is decided based on the river discharge of Air Manjuto. At present, the development for left bank area, that is, the extension works for the development of about 1,000 ha based on the revision of planning for the existing secondary canal (S.S. Baru) on the left bank area, is on going under the Provincial Government for the political settlement to the urgent transmigrants into the Kedung Ombo area.

On the other hand, however, it is clear that the project will have a shortage of irrigation water requirement after developing the above total area (16,687 ha).

In order to recover such the shortage of irrigation water requirement on the coming developed stage, therefore, it is planned by the Provincial Public Works that the necessary irrigation water for the area commanded by the diversion B.B.3 of S.S. Baru (Existing secondary canal) should be supplied from the Air Selagan area.

(2) River Discharge of Air Manjuto and Intake Discharge Plan

The river discharge of Air Manjuto and intake discharge at the Air Manjuto headworks have already been studied in the report shown in the above data a).

The cropping plan stipulated in the above data a). is as follows:

Table V-21 IRRIGATION PLAN OF MUKO-MUKO PROJECT

Crop	Left Bank Area	Right Bank Area	Total
Paddy (Dry Season)	5,247 ha	7,690 ha	12,937 ha
Upland (Dry Season)	6,768	9,919	16,687
Paddy (Wet Season)	6,768	9,919	16,687

Table V-22 MONTHLY RIVER AND INTAKE DISCHARGE FOR MUKO-MUKO PROJECT

Month	River Discharge	Unit Water Requirement	(Left Bank) Discharge (Left Bank)	(Right Bank) Discharge (Right Bank)	Total Intake Discharge	Remarks
	m^3/s	$m^3/s/ha$	m^3/s	m^3/s	m^3/s	
1	18.20	0.78	4.09	6.00	10.09	Cropping in Dry season
2	20.70	1.60	8.40	12.30	20.70	"
3	13.40	1.00	5.25	7.69	12.94	"
4	22.90	0.56	2.94	4.31	7.25	"
5	12.00	-	-	-	-	"
6	16.80	0.23	1.56	2.28	3.84	Cropping in Upland
7	12.90	0.37	2.50	3.67	6.17	"
8	17.50	-	-	-	-	"
9	13.60	0.74	5.01	7.34	12.35	Cropping in Wet season
10	17.70	1.00	6.77	9.92	16.69	"
11	22.20	0.14	0.95	1.39	2.34	"
12	20.80	0.02	0.14	0.20	0.34	"

(3) Tertiary Network for S.S. Baru

The tertiary network for the existing secondary canal (S.S. Baru) planned by the Provincial Public Works is shown in the following sketch:

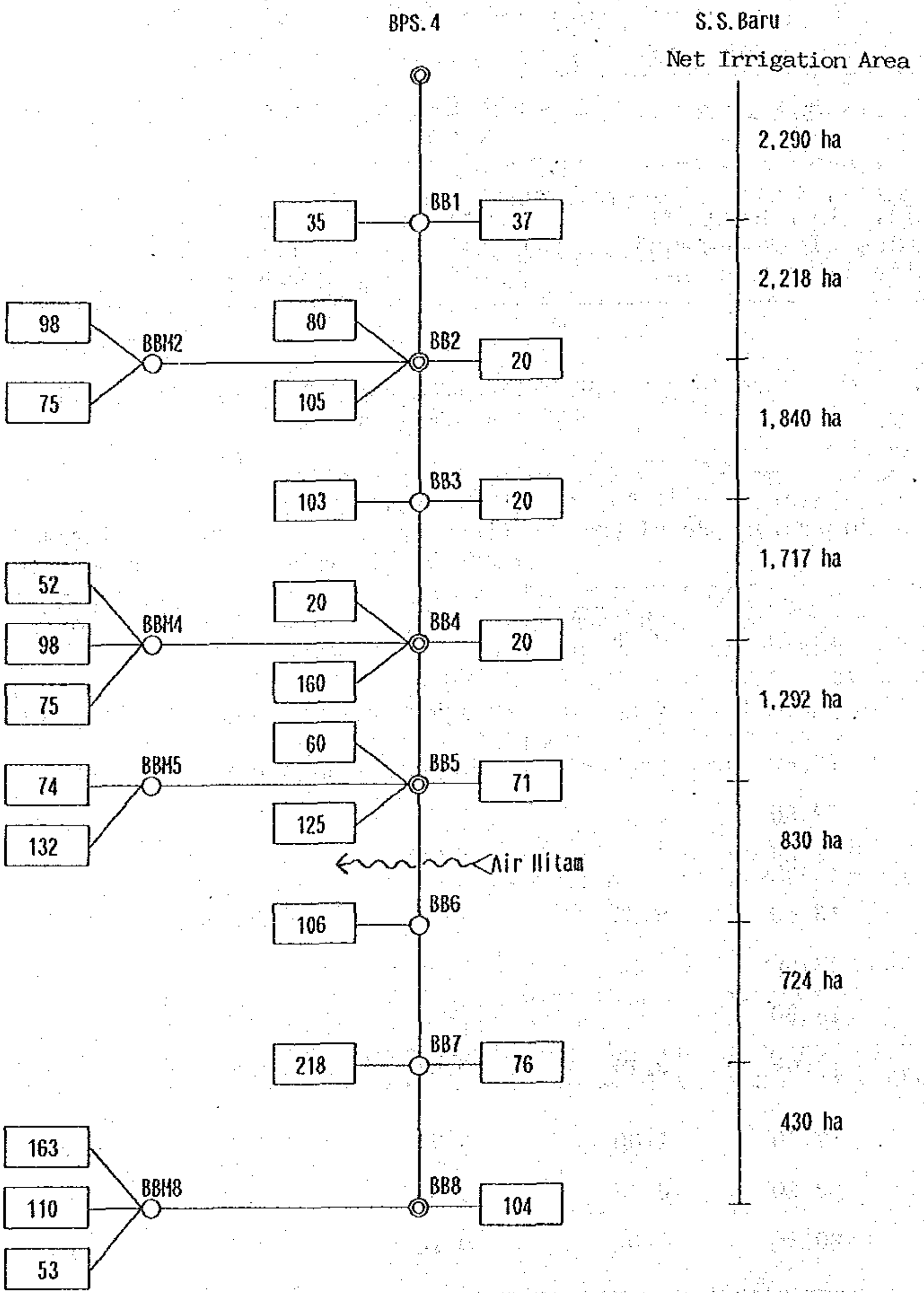


Fig. V-6 PRESENT TERTIARY SCHENE OF S.S.BARU

(4) Present Condition of Canal Plan & Profile (S.S. Baru)

The following table shows the present condition of canal plan & profile for S.S. Baru indicated in the as build drawings for the canal construction:

Table V-23 PRESENT CANAL SCALE OF S.S. BARU

Canal Name	No. of Diversion	Sectional Length	Design Discharge	Canal Bed Elevation	Normal Water Surface	Slope
		m	m^3/s	m	m	
S.S. Baru	BB.1	0	-	14.41	15.41	-
	BB.2	1,606	1.83	13.96	14.96	1/3,650
	BB.3	1,780	1.33	23.34	14.24	1/3,100
	BB.4	1,260	1.08	12.92	13.67	1/3,500
	BB.5	1,767	0.85	12.29	13.04	1/2,900
	BB.6	758	1.33	11.82	12.72	1/2,900
	W43	146	1.33	11.77	12.67	1/3,100
	BB.7	1,661	1.16	11.21	12.06	1/3,000
	BB.8	1,467	0.73	6.75	7.45	1/2,750
	Total:	10,445				
S. Muka	BB.2			13.98	14.88	
	BB.2M	978	1.33	10.58	11.40	1/7,100
	BB.4			23.77	13.40	
	BB.4M	1,187	0.39	12.35	12.98	1/2,850
	BB.5			12.41	13.04	
	BB.5M	777	0.38	8.17	8.80	1/3,300
	BB.8			6.74	7.44	
	BB.8M	726	0.57	6.46	7.11	1/3,000

(5) Present Capacity for Canal Water Discharge

The irrigation area commanded by each diversion under the present planning conditions, taking into account the unit water requirement for the development of Muko-Muko left bank area to be 1.0 (1/s/ha) in wet season and 1.6 (1/s/ha) in dry season and also the capacity for canal water supply from the diversion B.B.3 through the diversion B.B.5, is indicated in the following table:

Table V-24 PRESENT CAPACITY OF S.S. BARU

Name of Canal	Diversion	Canal Water Discharge	Cropping in Wet Season	Cropping in Dry Season
		$m^3/2$	ha	ha
S.S. Baru	B.B.1			
	B.B.2	1.83	1,830	1,144
	BB.3	1.33	1,330	831
	BB.4	1.08	1,080	675
	BB.5	0.85	850	531
	BB.6	0.85	850	531
	BB.7	0.85	850	531
	BB.8	0.73	730	456
S. Muka	BB.2M	1.33	1,330	831
	BB.4M	0.39	390	244
	BB.5M	0.38	1,380	238
	BB.8M	0.57	570	356

From the comparison of commanded area between the above table and the table shown in (3) of this chapter, the shortage of canal water requirement is pointed out:

1) In case of cropping in rainy season : Shortage of water for 442 ha at BB.5 diversion

2) In case of cropping in dry season : Shortage of water for 1,042 ha at BB.2 diversion

(6) Rehabilitation Plan for Secondary Canal (S.S. Baru)

In the future, the canal water from the Muko-Muko left bank area is supplied up to B.B.3 diversion of S.S. Baru.

On the other hand, the downstream area from B.B.4 diversion is included into the Selagan area. Therefore, the canal water for such area is supplied through B.B.4 diversion from the Air Selagan area.

As for the planning for this area, the following matters were well employed:

a) Soil conditions and ground surface elevations for the project area should be reviewed, and then the re-study for irrigation area to be available should be made.

b) The canal water supply for the left bank area of S.S. Baru having rather high elevations shall be made by the application of direct supply from the re-alignment canal.

On the other hand, the existing canal water supply shall be applied for the right bank area.

c) Siphon structure is recommendable for the canal structure crossing Air Hitam.

In order to minimize the capacity for the siphon structure and the canal cross sections for the downstream area, the re-alignment canal should be connected to B.B.4 and B.B.6 diversions.

d) The same unit water requirement employed for the Air Selagan area should be applied to this plan, too.

e) The planning ratio for paddy field shall be 200 %.

(7) Gross Irrigable Area

Table V-25 REVISED PLAN OF GROSS IRRIGABLE AREA

Canal concerned	Air Hitam Left Bank	Air Hitam Right Bank	Total
	(ha)	(ha)	(ha)
Re-alignment Canal	233	143	376
S.S. Baru	226	455	681
TOTAL	459	598	1,057

(8) Tertiary Network (Rehabilitation Plan)

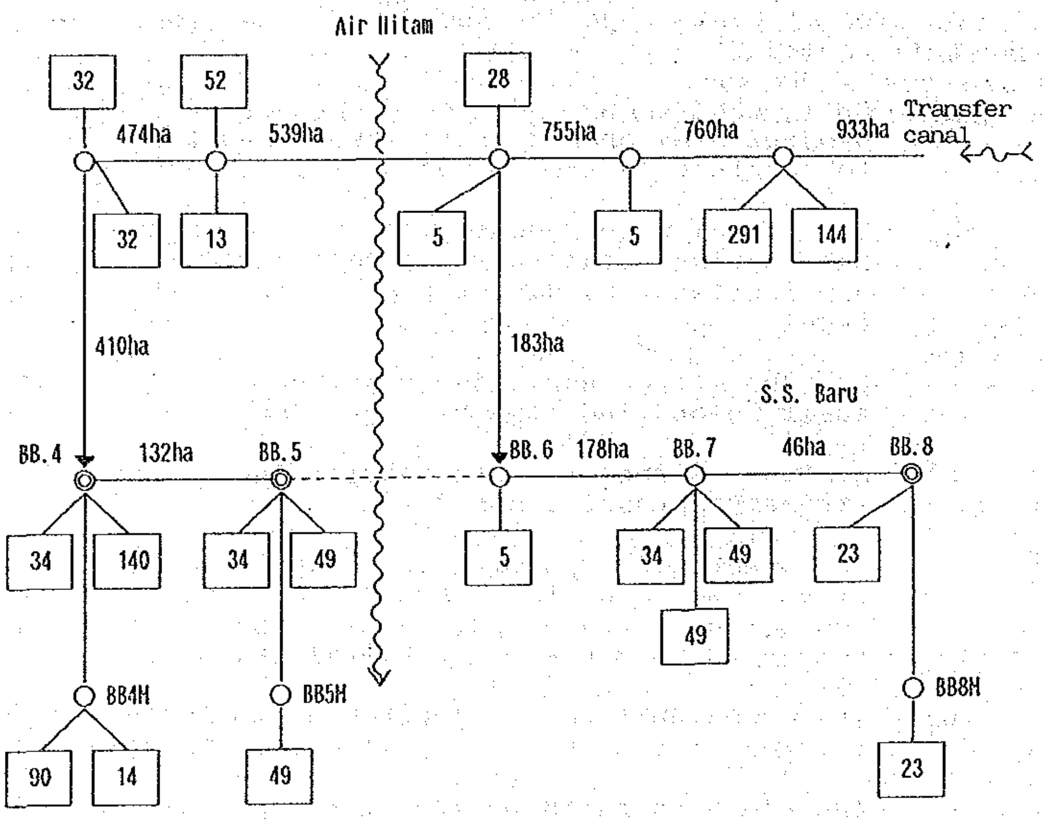


Fig. V-7 RELATION BETWEEN S.S.BARU AND TRANSFER CANAL

Accordingly, the irrigable area for the downstream from B.B.4 diversion is revised as follows:

1) Original Plan	:	1,717 ha
2) Rehabilitation Plan	:	933 ha
Reduction	:	784 ha

It is judged that the above area of 784 ha reduced from the original plan is inadequate for the use of paddy field.

Therefore, such area will be used for upland field, home yard, oil palm plantation and so on.

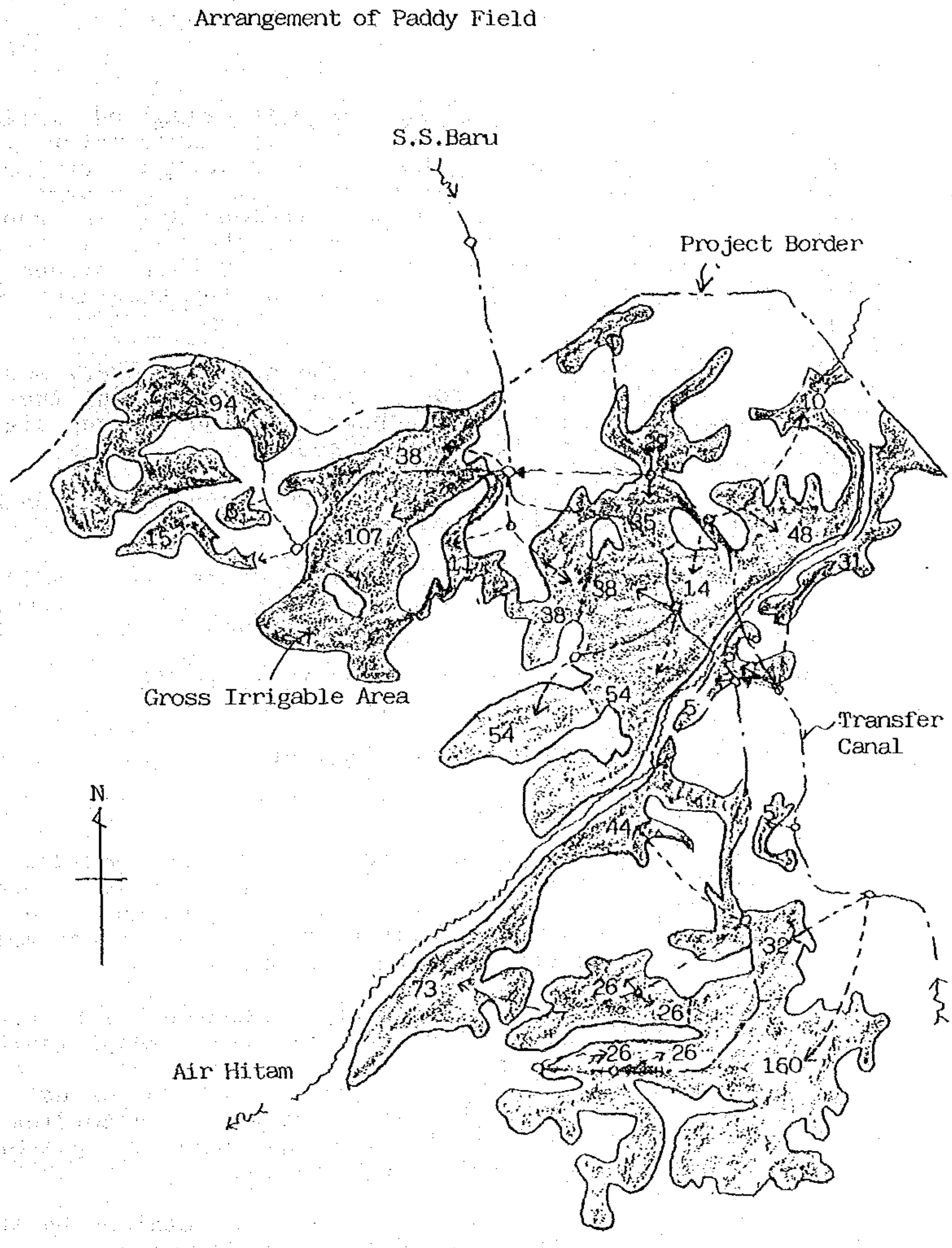


Fig. V-8 REVISED IRRIGATION SYSTEM FOR S.S. BARU

CHAPTER 3 IRRIGATION WATER REQUIREMENT

3.1 Ten Day River Discharge

In comparing the river discharge with required irrigation water by months, the Air Selagan can fully supply water to the benefited area. However, in the case when they are compared by 10 days, there are cases when the river discharge is so insufficient that the area cannot be irrigated. In addition, effective storage is not expected because the weir is proposed for taking irrigation water. Therefore, 10 day discharge is estimated by the following methods to be on the safe side for the project:

a. The average of ten day discharges from every month is calculated first for 8 years, and then the ratio between the ten day discharge and the average discharge is calculated all the year round.

b. The monthly discharge of 1/5 non-exceedance probability is calculated every month.

c. The designed ten day river discharge is obtained by multiplying the monthly discharge of 1/5 non-exceedance probability into the ratio mentioned in a.

The calculation results are given in Table V-26.

3.2 Cropping Pattern and Crop Coefficient

a. Planning of cropping pattern

The growing period of paddy should be decided after doing comparative study of variety selection, meteorological condition, and river discharge. In this report, IR-64 which had been prevailing extensively in Indonesia is studied, and is adopted.

In comparison of the river discharge with required irrigation water, the river can fully supply water to the benefited area, which has little effect on the planting period. Therefore, the following three cropping patterns are assumed according to meteorological condition and the cropping pattern is studied based on water requirements.

Case-1 : Maximum potential yield is obtained by the use of radiation data. Commencement of planting is to be on October 11th.

Case-2 : Harvesting period is to be from May to June which are in less rainfall, January 1st being commencement day of planting.

Case-3 : Considering nursery period, October 1st is to be commencement of planting.

The combination of cropping pattern is to be paddy-paddy in a year in the above cases.

b. The crop coefficient of paddy is decided as in the following Table V-27 and Fig. V-9 based on the design standard KP-01 published by Directorate of Irrigation:

Table V-26 Planning 10 Days Discharge

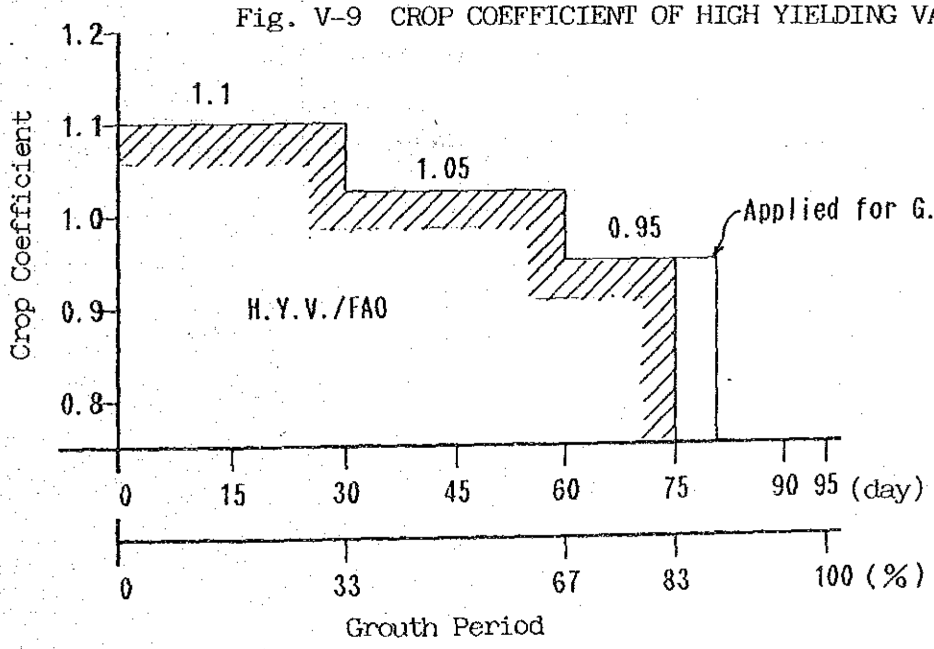
CA=375 km2

Month	10 days	Average 10 days Discharge	1/5 Probable Monthly Dischar.	Planning 10 days Discharge
Jan.	1	66.24 m3/s	m3/s	51.67 m3/s
	2	43.99		34.31
	3	44.35		34.59
	Average	51.53	40.19	40.19
Feb.	1	50.70		35.31
	2	32.22		22.44
	3	36.78		25.62
	Average	39.90	27.79	27.79
Mar.	1	50.70		39.68
	2	56.49		44.21
	3	52.11		40.79
	Average	53.10	41.56	41.56
Apr.	1	40.76		28.06
	2	34.10		23.48
	3	53.95		37.14
	Average	42.94	29.56	29.56
May	1	33.72		25.53
	2	28.47		21.56
	3	24.89		18.85
	Average	29.03	21.98	21.98
Jun.	1	23.21		15.50
	2	25.83		17.24
	3	17.15		11.45
	Average	22.06	14.73	14.73
Jul.	1	21.86		16.61
	2	21.67		16.47
	3	25.64		19.48
	Average	23.06	17.52	17.52
Aug.	1	31.34		16.66
	2	17.33		9.21
	3	24.15		12.83
	Average	24.27	12.90	12.90
Sep.	1	38.14		27.69
	2	36.48		26.48
	3	50.71		36.82
	Average	41.78	30.33	30.33
Oct.	1	41.50		27.23
	2	46.04		30.21
	3	46.59		30.57
	Average	44.71	29.34	29.34
Nov.	1	54.42		32.37
	2	40.76		24.25
	3	60.65		36.08
	Average	51.94	30.90	30.90
Dec.	1	43.15		30.81
	2	52.87		37.75
	3	55.78		39.83
	Average	50.60	36.13	36.13

Table V-27 CROP COEFFICIENT

Growth	Periol	95 days
Month	5 days	
1st	1	1.1
	2	1.1
	3	1.1
	4	1.1
	5	1.1
	6	1.1
2nd	1	1.05
	2	1.05
	3	1.05
	4	1.05
	5	1.05
	6	1.05
3rd	1	0.95
	2	0.95
	3	0.95
	4	0.95
	5	0.0
	6	0.0
4th	1	0.0

Fig. V-9 CROP COEFFICIENT OF HIGH YIELDING VARAETY(HYV)



3.3 Irrigation Water Requirements

Irrigation water requirements are estimated using the meteorological data at Pondok Panjang, and rainfall data at Pondok Kopi.

The meteorological data which are rearranged in ten days are indicated in Table V-29.

a. Evapotranspiration

Crop evapotranspiration is obtained using Modified Penman Method as follows:

Table V-28 TEN DAY EVAPOTRANSPIRATION

Period	Evapotrans-piration	Period	Evapotrans-piration	Period	Evapotrans-piration
Jan. 1	43 mm	May 1	44 mm	Sep. 1	41 mm
2	43	2	44	2	42
3	50	3	44	3	39
Feb. 1	44	Jun. 1	40	Oct. 1	42
2	48	2	39	2	39
3	36	3	46	3	45
Mar. 1	40	Jul. 1	45	Nov. 1	41
2	44	2	44	2	43
3	49	3	50	3	41
Apr. 1	44	Aug. 1	44	Dec. 1	41
2	39	2	43	2	45
3	47	3	47	3	49

(Note: South latitude 2° 35')

The detailed calculation is shown in the Table V-30.

Table V-29 TEN-DAY METEOROLOGICAL DATA

Month	Temper -rature (C)	Rel. Humidity (%)	Sunshine dur (%)	Wind Vel. (Km/day)
JAN 1	31.1	92	39	27.3
2	31.2	93	38	27.2
3	31.3	92	44	29.0
FEB 1	31.1	92	40	29.1
2	31.6	91	46	29.0
3	32.1	90	40	28.3
MAR 1	31.4	92	30	32.3
2	31.8	93	38	29.4
3	31.6	92	40	29.1
APR 1	31.9	91	43	28.0
2	31.7	91	32	29.1
3	31.9	92	49	26.5
MAY 1	32.0	91	49	25.0
2	32.1	91	48	25.7
3	31.9	91	40	25.2
JUN 1	31.9	92	43	25.6
2	32.1	93	41	25.1
3	31.8	94	59	25.4
JUL 1	31.7	96	54	25.7
2	31.8	95	51	25.7
3	32.0	93	55	25.3
AUG 1	31.8	94	46	27.9
2	31.8	92	43	30.5
3	31.7	92	43	33.9
SEP 1	31.2	91	34	36.2
2	31.1	93	37	34.9
3	30.9	95	33	34.4
OCT 1	31.0	94	37	34.4
2	31.1	94	30	32.4
3	30.7	93	35	30.4
NOV 1	31.0	94	35	31.1
2	31.1	93	39	30.8
3	31.0	94	36	29.7
DEC 1	31.1	93	37	29.6
2	31.2	93	45	33.4
3	31.3	93	43	30.3

Table V-30.1 CONSUMPTIVE WATER USE OF CROP BY MODIFIED PENMAN METHOD(1/2)

*** CONSUMPTIVE WATER USE OF CROP BY MODIFIED PENMAN METHOD *** (1)

ITEM	JAN.			FEB.			MAR.			APR.			MAY.			JUN.		
	1	2	3	1	2	3	1	2	3	1	2	3	1	2	3	1	2	3
DATA:																		
LATITUDE:2'35" SOUTH																		
TEMPARATURE (°C)	31.1	31.2	31.3	31.1	31.6	32.1	31.4	31.8	31.6	31.9	31.7	31.9	32.0	32.1	31.9	31.9	32.1	31.8
REL. HUMIDITY (%)	92	93	92	92	91	90	92	93	92	91	91	92	91	91	91	92	93	94
SUN DURATION (%)	39	38	44	40	46	40	30	38	40	43	32	49	49	48	40	43	41	59
WIND VELOCITY (M/S)	0.32	0.31	0.34	0.34	0.34	0.33	0.37	0.34	0.34	0.32	0.34	0.31	0.29	0.30	0.29	0.30	0.29	0.29
CALCULATION:																		
1. F(TAI)/100	9.73	9.75	9.76	9.73	9.80	9.86	9.77	9.82	9.80	9.84	9.81	9.84	9.85	9.86	9.84	9.84	9.86	9.82
2. DELTA*100/L.HEAT	3.32	3.34	3.36	3.32	3.41	3.49	3.37	3.44	3.41	3.46	3.42	3.46	3.47	3.49	3.46	3.46	3.49	3.44
3. SAT.VAPOUR PRESS.	33.91	34.10	34.29	33.91	34.88	35.89	34.49	35.28	34.88	35.48	35.08	35.48	35.68	35.89	35.48	35.48	35.89	35.28
4. GAMMA+DELTA	2.41	2.42	2.43	2.41	2.46	2.51	2.44	2.48	2.46	2.49	2.47	2.49	2.50	2.51	2.49	2.49	2.51	2.48
5. WATER VAPO.PRESS.	31.20	31.71	31.55	31.20	31.74	32.30	31.73	32.81	32.09	32.29	31.92	32.64	32.47	32.66	32.29	32.64	33.38	33.16
6. F(TOP)	0.040	0.036	0.037	0.040	0.036	0.032	0.036	0.029	0.034	0.032	0.035	0.030	0.031	0.030	0.032	0.030	0.025	0.027
7. PWS-PWZ	2.71	2.39	2.74	2.71	3.14	3.59	2.76	2.47	2.79	3.19	3.16	2.84	3.21	3.23	3.19	2.84	2.51	2.12
8. RF(U2)	0.115	0.115	0.117	0.117	0.117	0.116	0.120	0.117	0.117	0.115	0.117	0.115	0.113	0.114	0.113	0.114	0.113	0.113
9. GAMMA*EVAPORAT.	0.31	0.27	0.32	0.32	0.37	0.42	0.33	0.29	0.33	0.37	0.37	0.33	0.36	0.37	0.36	0.32	0.28	0.24
10. SHORT WAVE ANGOT.	8.79	8.79	8.79	8.98	8.98	8.98	8.92	8.92	8.92	8.54	8.54	8.54	8.01	8.01	8.01	7.69	7.69	7.69
11. ASH*F(IR)	0.369	0.365	0.389	0.373	0.397	0.373	0.334	0.365	0.373	0.385	0.342	0.408	0.408	0.404	0.373	0.385	0.377	0.447
12. SHORT WAVE RAD.	3.24	3.21	3.42	3.35	3.57	3.35	2.98	3.26	3.33	3.29	2.92	3.48	3.27	3.24	2.99	2.96	2.90	3.44
13. F(M)	0.61	0.60	0.64	0.62	0.65	0.62	0.55	0.60	0.62	0.64	0.56	0.67	0.67	0.67	0.62	0.64	0.62	0.74
14. LONG WAVE RAD.	0.24	0.21	0.23	0.24	0.23	0.20	0.19	0.17	0.21	0.20	0.19	0.20	0.20	0.20	0.20	0.19	0.15	0.20
15. NET RADIATION	3.00	3.00	3.19	3.11	3.34	3.15	2.79	3.09	3.12	3.09	2.73	3.28	3.07	3.04	2.79	2.77	2.75	3.24
16. DELTA*NET RAD.	9.96	10.02	10.72	10.33	11.39	10.99	9.40	10.63	10.64	10.69	9.34	11.35	10.65	10.61	9.65	9.58	9.60	11.15
17. DELTA*NET.R.+G*EA	10.27	10.29	11.04	10.65	11.76	11.41	9.73	10.92	10.97	11.06	9.71	11.68	11.01	10.98	10.01	9.90	9.88	11.39
18. EO (MM/DAY)	4.26	4.25	4.54	4.42	4.78	4.55	3.99	4.40	4.46	4.44	3.93	4.69	4.40	4.37	4.02	3.98	3.94	4.59
19. EO (MM/10DAY)	43	43	50	44	48	36	40	44	49	44	39	47	44	44	44	40	39	46
20. EO (MM/MONTH)			136			128			133			130			132			125

Table V-30.2 CONSUMPTIVE WATER USE OF CROP BY MODIFIED PENMAN METHOD(2/2)

*** CONSUMPTIVE WATER USE OF CROP BY MODIFIED PENMAN METHOD *** (2)

ITEM	JUL.			AUG.			SEP.			OCT.			NOV.			DEC.		
	1	2	3	1	2	3	1	2	3	1	2	3	1	2	3	1	2	3
DATA:																		
LATITUDE:2'35" SOUTH																		
TEMPARATURE (°C)	31.7	31.8	32.0	31.8	31.8	31.7	31.2	31.1	30.9	31.0	31.1	30.7	31.0	31.1	31.0	31.1	31.2	31.3
REL. HUMIDITY (%)	96	95	93	94	92	92	91	93	95	94	94	93	94	93	94	93	93	93
SUN DURATION (%)	54	51	55	46	43	43	34	37	33	37	30	35	35	39	36	37	45	43
WIND VELOCITY (M/S)	0.30	0.30	0.29	0.32	0.35	0.39	0.42	0.40	0.40	0.40	0.38	0.35	0.36	0.36	0.34	0.34	0.39	0.35
CALCULATION:																		
1. F(TAI)/100	9.81	9.82	9.85	9.82	9.82	9.81	9.75	9.73	9.71	9.72	9.73	9.68	9.72	9.73	9.72	9.73	9.75	9.76
2. DELTA*100/L.HEAT	3.42	3.44	3.47	3.44	3.44	3.42	3.34	3.32	3.29	3.31	3.32	3.26	3.31	3.32	3.31	3.32	3.34	3.36
3. SAT.VAPOUR PRESS.	35.08	35.28	35.68	35.28	35.28	35.08	34.10	33.91	33.52	33.71	33.91	33.14	33.71	33.91	33.71	33.91	34.10	34.29
4. GAMMA+DELTA	2.47	2.48	2.50	2.48	2.48	2.47	2.42	2.41	2.40	2.41	2.41	2.38	2.41	2.41	2.41	2.41	2.42	2.43
5. WATER VAPO.PRESS.	33.68	33.52	33.18	33.16	32.46	32.27	31.03	31.54	31.84	31.69	31.88	30.82	31.69	31.54	31.69	31.54	31.71	31.89
6. F(TDP)	0.023	0.024	0.026	0.027	0.031	0.033	0.041	0.038	0.036	0.037	0.035	0.043	0.037	0.038	0.037	0.038	0.036	0.035
7. PWS-PWZ	1.40	1.76	2.50	2.12	2.82	2.81	3.07	2.37	1.68	2.02	2.03	2.32	2.02	2.37	2.02	2.37	2.39	2.40
8. RF(U2)	0.114	0.114	0.113	0.115	0.118	0.122	0.125	0.123	0.123	0.123	0.121	0.118	0.119	0.119	0.117	0.117	0.122	0.118
9. GAMMA*EVAPORAT.	0.16	0.20	0.28	0.24	0.33	0.34	0.38	0.29	0.21	0.25	0.25	0.27	0.24	0.28	0.24	0.28	0.29	0.28
10. SHORT WAVE ANGOT.	7.79	7.79	7.79	8.24	8.24	8.24	8.70	8.70	8.70	8.89	8.89	8.89	8.79	8.79	8.79	8.68	8.68	8.68
11. ASH*F(IR)	0.428	0.416	0.432	0.397	0.385	0.385	0.350	0.362	0.346	0.362	0.334	0.354	0.354	0.369	0.358	0.362	0.393	0.385
12. SHORT WAVE RAD.	3.33	3.24	3.37	3.27	3.17	3.17	3.05	3.15	3.01	3.22	2.97	3.15	3.11	3.24	3.15	3.14	3.41	3.34
13. F(M)	0.71	0.69	0.71	0.65	0.64	0.64	0.58	0.60	0.57	0.60	0.55	0.58	0.58	0.61	0.59	0.60	0.65	0.64
14. LONG WAVE RAD.	0.16	0.16	0.18	0.17	0.19	0.21	0.23	0.22	0.20	0.22	0.19	0.24	0.21	0.23	0.21	0.22	0.23	0.22
15. NET RADIATION	3.17	3.08	3.19	3.10	2.98	2.96	2.82	2.93	2.81	3.00	2.78	2.91	2.90	3.01	2.94	2.92	3.18	3.12
16. DELTA*NET RAD.	10.84	10.60	11.07	10.66	10.25	10.12	9.42	9.73	9.24	9.93	9.23	9.49	9.60	9.99	9.73	9.69	10.62	10.48
17. DELTA*NET.R.+G*EA	11.00	10.80	11.35	10.90	10.58	10.46	9.80	10.02	9.45	10.18	9.48	9.76	9.84	10.27	9.97	9.97	10.91	10.76
18. EO (MM/DAY)	4.45	4.35	4.54	4.40	4.27	4.23	4.05	4.16	3.94	4.22	3.93	4.10	4.08	4.26	4.14	4.14	4.51	4.43
19. EO (MM/10DAY)	45	44	50	44	43	47	41	42	39	42	39	45	41	43	41	41	45	49
20. EO (MM/MONTH)			139			134			122			126			125			135

b. Effective Rainfall

Effective rainfall for rice is assumed by the following equation using rainfall data at Pondok Kopi for 9 years during the period from 1981 to 1989 (See Table V-31).

$$Re = 0.7 \times Rm$$

where, Re : Effective rainfall mm/month
Rm : Monthly rainfall with 1/5 non-exceedance probability mm/month

The planning ten day effective rainfall is distributed in proportion to the ratio of ten day rainfall to the above monthly rainfall of 1/5 non-exceedance probability.

c. Percolation

The measurement of percolation was carried out at 3 places in the newly developed paddy field of the transmigrated area called SP-III and IV, during the period from February 13 to February 14, 1990.

The required water for the paddy fields in SP-III relies on the water supplied from the swampy area. On the other hand, the required water for the paddy fields in SP-IV is supplied through the connecting canal (BB4M) of Muko-Muko Irrigation Project. The same measuring result of 1.0 mm/day was obtained in each place.

The following percolation data were collected as a supplemental data for the Project:

- Air Manjuto Project, 1985/86:
  Design Value P = 3.00 mm/day
- Air Lais Project, 1981 :
  Design Value P = 2.00 mm/day
- Air Kutahun Project, 1980 :
  Field Survey P = 0.98 mm/day
- Air Bengkulu & Musi. 1975 :
  Design Value P = 2 - 6mm/day
- Air Buku/Dusun Curup, 1983 :
  Field Survey P = 2.8 mm/day

Comparing with the above collected data, the value of 1.0 mm/day resulted in the actual field survey is rather low. However, it is a result obtained under the field condition that the measured paddy field is laid on a low-lying area having rather clayey soil. Taking into account the above field condition, therefore, it is assumed that the value of 3.0 mm/day, which is adopted in this Study, is acceptable as a safety value for the Project.

d. Water Requirement for land preparation

In general, peak water requirements occur at the time of land preparation. Therefore, for lessening water requirements, land preparation period at each paddy field is staggered, and made longer. If rotation system is employed, time lag of land preparation is restricted by labour force and water availability.

In this project, 55 days are adopted for land preparation period.

Irrigation requirements at field level are calculated by the method of Van de Goor & Zijlstra.

Condition:

Presaturation requirements	for wet paddy	: S = 275 mm
"	for dry paddy	: S = 275 mm
Land preparation period		: T = 55 days
Percolation		: P = 3.5 mm/day

$$IR = M.e^{k}/(e^{k}-1)$$

where,

IR : Irrigation requirement at field level, mm/day

M : Water requirement to compensate for evaporation and percolation of the fields already saturated.
$M = Eo + P$

Eo : Open water evaporation taken at 1.1 x ETo during land preparation, mm/day

K : M.T/S

S : Presaturation requirement from below items

. Required water depth above soil surface after puddling : 150 mm
. Saturation requirement : 90-140mm (mean 115mm)
. Nursery requirement : 5 mm
. Losses : 5 mm

S : 275 mm

Table V-31 Planning 10 Day Effective Rainfall

Month	10 days	Average 10 days Rainfall	1/5 Probable Monthly Rainfall	Effective Rainfall
Jan.	1	130.8 mm	mm	68 mm
	2	116.3		61
	3	126.4		66
	Total	373.5	278.3	195
Feb.	1	114.6		59
	2	96.6		50
	3	57.0		29
	Total	268.2	196.8	138
Mar.	1	139.2		60
	2	154.3		67
	3	144.4		62
	Total	437.9	269.6	189
Apr.	1	113.4		42
	2	97.9		37
	3	144.6		54
	Total	355.9	190.6	133
May	1	99.4		52
	2	75.2		39
	3	69.9		36
	Total	244.5	181.7	127
Jun.	1	65.4		27
	2	73.2		30
	3	44.2		19
	Total	182.8	107.9	76
Jul.	1	63.5		24
	2	87.1		32
	3	81.9		30
	Total	232.5	123.6	86
Aug.	1	54.6		21
	2	66.2		25
	3	66.8		26
	Total	187.6	103.1	72
Sep.	1	50.2		29
	2	102.6		58
	3	188.8		107
	Total	341.6	277.6	194
Oct.	1	97.4		38
	2	117.6		45
	3	159.9		62
	Total	374.9	206.5	145
Nov.	1	96.9		37
	2	111.9		43
	3	174.3		67
	Total	383.1	209.3	147
Dec.	1	106.6		61
	2	134.2		77
	3	118.3		68
	Total	359.1	294.9	206
Grand total		3741.6	2439.9	1708

Table V-32 *** PUDDLING WATER REQUIREMENT ***(T= 45 days, S= 275 mm)

Period (1)	ETO mm/d (2)	EO mm/d (3)	P mm/d (4)	M (5) 3+4	k (6)	MS*e (7)	eS-1 (8)	IR- 275 mm/d (9) 7/8
JAN. 1	4.3	4.7	3.0	7.7	1.260	27.143	2.525	10.7
2	4.3	4.7	3.0	7.7	1.260	27.143	2.525	10.7
3	4.5	5.0	3.0	8.0	1.309	29.616	2.702	11.0
FEB. 1	4.4	4.8	3.0	7.8	1.276	27.940	2.582	10.8
2	4.8	5.3	3.0	8.3	1.358	32.270	2.888	11.2
3	4.6	5.1	3.0	8.1	1.325	30.472	2.762	11.0
MAR. 1	4.0	4.4	3.0	7.4	1.211	24.842	2.357	10.5
2	4.4	4.8	3.0	7.8	1.276	27.940	2.582	10.8
3	4.5	5.0	3.0	8.0	1.309	29.616	2.702	11.0
APR. 1	4.4	4.8	3.0	7.8	1.276	27.940	2.582	10.8
2	3.9	4.3	3.0	7.3	1.195	24.119	2.304	10.5
3	4.7	5.2	3.0	8.2	1.342	31.381	2.827	11.1
MAY. 1	4.4	4.8	3.0	7.8	1.276	27.940	2.582	10.8
2	4.4	4.8	3.0	7.8	1.276	27.940	2.582	10.8
3	4.0	4.4	3.0	7.4	1.211	24.842	2.357	10.5
JUN. 1	4.0	4.4	3.0	7.4	1.211	24.842	2.357	10.5
2	3.9	4.3	3.0	7.3	1.195	24.119	2.304	10.5
3	4.6	5.1	3.0	8.1	1.325	30.472	2.762	11.0
JUL. 1	4.5	5.0	3.0	8.0	1.309	29.616	2.702	11.0
2	4.4	4.8	3.0	7.8	1.276	27.940	2.582	10.8
3	4.5	5.0	3.0	8.0	1.309	29.616	2.702	11.0
AUG. 1	4.4	4.8	3.0	7.8	1.276	27.940	2.582	10.8
2	4.3	4.7	3.0	7.7	1.260	27.143	2.525	10.7
3	4.2	4.6	3.0	7.6	1.244	26.364	2.469	10.7
SEP. 1	4.1	4.5	3.0	7.5	1.227	25.583	2.411	10.6
2	4.2	4.6	3.0	7.6	1.244	26.364	2.469	10.7
3	3.9	4.3	3.0	7.3	1.195	24.119	2.304	10.5
OCT. 1	4.2	4.6	3.0	7.6	1.244	26.364	2.469	10.7
2	3.9	4.3	3.0	7.3	1.195	24.119	2.304	10.5
3	4.1	4.5	3.0	7.5	1.227	25.583	2.411	10.6
NOV. 1	4.1	4.5	3.0	7.5	1.227	25.583	2.411	10.6
2	4.3	4.7	3.0	7.7	1.260	27.143	2.525	10.7
3	4.1	4.5	3.0	7.5	1.227	25.583	2.411	10.6
DEC. 1	4.1	4.5	3.0	7.5	1.227	25.583	2.411	10.6
2	4.5	5.0	3.0	8.0	1.309	29.616	2.702	11.0
3	4.4	4.8	3.0	7.8	1.276	27.940	2.582	10.8

e. Water layer replacement

According to the Indonesian Design Standard, 2 replacement, each of 50mm (3.3 mm/day for 1/2 month) at about 1 month and 2 months after transplanting for fertilizer application.

A schematic cropping pattern with the layer replacement is shown as below.

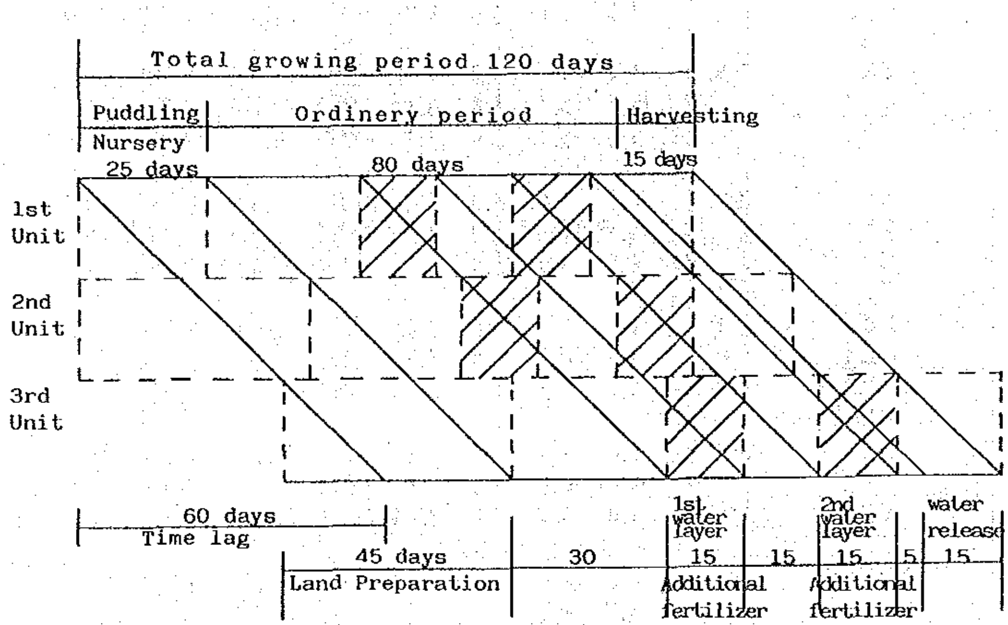


Fig. V-10 MODEL OF PADDY CULTIVATION

Remarks: 1. Rectangular shapes show actual farming period at each unit.

2. Inclined line is representative farming period for a whole area.

3. Commencement period of pudding is staggered in two months, and paddy cultivation is performed in three groups.

f. Irrigation Efficiency

It is very difficult to have effective irrigation for water saving on irrigation development in new land reclamation and transmigration areas. It is found the case of 0.55 in overall irrigation efficiency in Jawa island where the development has been advanced.

For this project the irrigation efficiency is adopted to be 55% in overall considering unlined canal system. The efficiency is divided into as follows:

Main & secondary canals : 80%
Tertiary system : 70%

g. Other coefficients

Water requirements for each group are calculated according to the above groups. The conditions for land preparation period, nursery period, crop coefficient, and water layer are given in the following table:

h. Results of calculation

In the above three cases, two cases are considered in which repairing period for canals is established before the commencements of wet paddy cultivation and of dry paddy cultivation. Therefore, water requirements are calculated in six cases in total. The peak water requirements for each case are indicated in Table V-33 and Fig. V-11. As the river discharge is affluent, the case which shows less water requirements in dry season is favorable. Subsequently, the case 2-2 is adopted taking into account of the potential maximum yield. (Refer to the Appendix IV, Clause 3.3.3 "Basic Condition for Settlement of Alternative Cropping Pattern".)

Table V-33 COMPARISON OF UNIT WATER REQUIREMENT

Case	Crop Season	1st Date of Puddling	Max. Unit	Month of
			1/s/ha	
Case 1-1	Wet	Oct. 11	0.97	Nov.
	Dry	Mar. 26	1.69	Jun.
Case 1-2	Wet	Oct. 11	0.97	Nov.
	Dry	Apr. 26	1.72	Jun.
Case 2-1	Wet	Jan. 1	1.36	Feb.
	Dry	Jun. 16	1.53	Jul., Aug.
Case 2-2 (adopted)	Wet	Jan. 1	1.36	Feb.
	Dry	Jul. 16	1.53	Aug.
Case 3-1	Wet	Oct. 1	1.14	Nov.
	Dry	Mar. 16	1.46	Jun.
Case 3-2	Wet	Oct. 1	1.14	Nov.
	Dry	Apr. 16	1.72	Jun.

Fig. V-11 **COMPARISON OF CROPPING PATTERN**

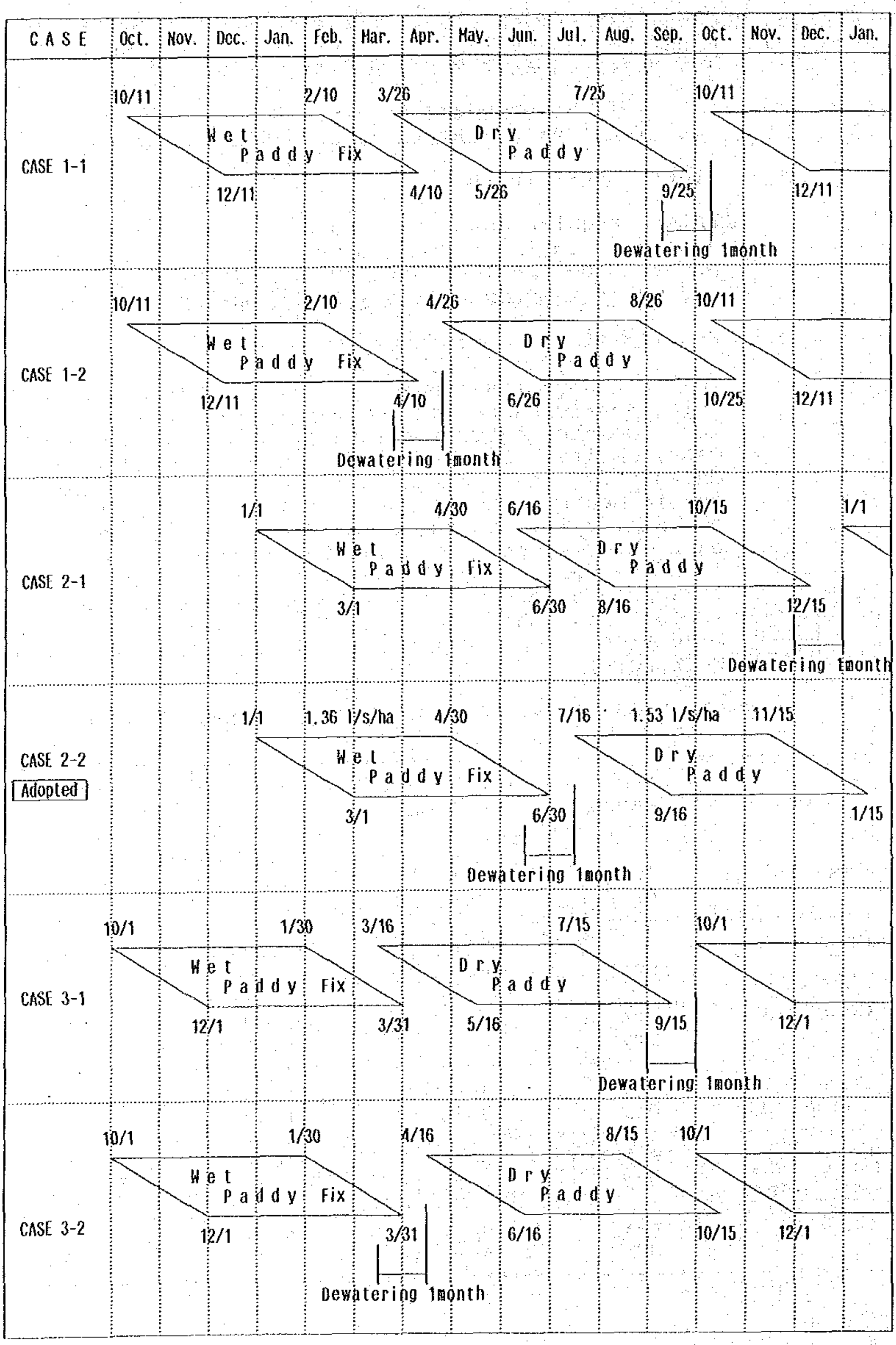


Table V-34

PUDDLING PERIOD, CROP COEFFICIENT AND WATER LAYER

MONTH	5 DAYS PERIOD	C1	C2	C3	LP AREA	C	CROP AREA	WLR1 mm/d	WLR2 mm/d	WLR3 mm/d	MEAN WLR mm/d
1	2	3	4	5	6	7	8	9	10	11	12
1st	1	LP	LP		2/3						
	2	LP	LP		2/3						
	3	LP	LP		2/3						
	4	LP	LP		2/3						
	5	LP	LP		2/3						
	6	1.1	LP		1/3	1.1	1/3				
2nd	1	1.1	LP		1/3	1.1	1/3				
	2	1.1	LP		1/3	1.1	1/3				
	3	1.1	LP	LP	2/3	1.1	1/3				
	4	1.1	1.1	LP	1/3	1.1	2/3				
	5	1.1	1.1	LP	1/3	1.1	2/3				
	6	1.05	1.1	LP	1/3	1.08	2/3	3.3			1.1
3rd	1	1.05	1.1	LP	1/3	1.08	2/3	3.3			1.1
	2	1.05	1.1	LP	1/3	1.08	2/3	3.3			1.1
	3	1.05	1.1	LP	1/3	1.08	2/3				
	4	1.05	1.05	LP	1/3	1.05	2/3		3.3		1.1
	5	1.05	1.05	LP	1/3	1.05	2/3		3.3		1.1
	6	0.95	1.05	1.1		1.03	1	3.3	3.3		2.2
4th	1	0.95	1.05	1.1		1.03	1	3.3			1.1
	2	0.95	1.05	1.1		1.03	1	3.3			1.1
	3	0.95	1.05	1.1		1.03	1				
	4	0.00	0.95	1.1		1.03	2/3		3.3		1.1
	5	0.00	0.95	1.1		1.03	2/3		3.3		1.1
	6	0.00	0.95	1.05		1.00	2/3		3.3	3.3	2.2
5th	1	-	0.95	1.05		1.00	2/3			3.3	1.1
	2	-	0.00	1.05		1.05	1/3			3.3	1.1
	3	-	0.00	1.05		1.05	1/3				
	4	-	0.00	1.05		1.05	1/3				
	5	-	-	1.05		1.05	1/3				
	6	-	-	0.95		0.95	1/3			3.3	1.1
6th	1	-	-	0.95		0.95	1/3			3.3	1.1
	2	-	-	0.95		0.95	1/3			3.3	1.1
	3	-	-	0.95		0.95	1/3				
	4	-	-	0.00		0.00					
	5	-	-	0.00		0.00					
	6	-	-	0.00		0.00					

Table V-35.1 Comparison of Water Requirement(1/3)

Unit: lit./s/ha

	Case 1-1	Case 1-2		Case 1-1	Case 1-2
Period	Oct.11 Mar.26	Oct.11 Apr.26	Period	Oct.11 Mar.26	Oct.11 Apr.26
Jan.1	0.60	0.60	Jul.1	1.33	1.60
2	0.60	0.60	2	1.10	1.37
3	0.51	0.51	3	0.84	1.38
4	0.51	0.51	4	0.84	1.38
5	0.34	0.34	5	1.14	1.50
6	0.46	0.46	6	0.91	1.27
Feb.1	0.46	0.46	Aug.1	0.61	1.37
2	0.67	0.67	2	0.38	1.14
3	0.63	0.63	3	0.35	0.93
4	0.44	0.44	4	0.35	0.93
5	0.29	0.29	5	0.55	1.14
6	0.29	0.29	6	0.55	0.91
Mar.1	0.08	0.08	Sep.1	0.51	0.54
2	0.29	0.29	2	0.28	0.31
3	0.26	0.26	3	0.00	0.11
4	0.26	0.26	4	0.00	0.11
5	0.12	0.12	5	0.00	0.00
6	0.76	0.00	6	0.00	0.00
Apr.1	0.93	0.00	Oct.1	0.00	0.45
2	0.93	0.00	2	0.00	0.22
3	0.96	0.00	3	0.85	0.85
4	0.96	0.00	4	0.85	0.85
5	0.59	0.00	5	0.70	0.70
6	0.59	0.80	6	0.70	0.70
May.1	0.57	0.79	Nov.1	0.97	0.97
2	0.97	0.79	2	0.74	0.74
3	1.03	0.97	3	0.68	0.68
4	1.03	0.97	4	0.68	0.68
5	1.30	0.78	5	0.61	0.61
6	1.30	0.78	6	0.39	0.39
Jun.1	1.42	0.87	Dec.1	0.51	0.51
2	1.19	1.43	2	0.73	0.73
3	1.33	1.13	3	0.48	0.48
4	1.33	1.13	4	0.48	0.48
5	1.69	1.72	5	0.54	0.54
6	1.46	1.72	6	0.75	0.75

Table V-35.2 Comparison of Water Requirement(2/3)

Unit: lit./s/ha

Period	Case 2-1 Jan. 1 Jun.16	Case 2-2 Jan. 1 Jul.16	Period	Case 2-1 Jan. 1 Jun.16	Case 2-2 Jan. 1 Jul.16
Jan.1	0.55	0.55	Jul.1	1.21	0.00
2	0.55	0.55	2	1.21	0.00
3	0.65	0.65	3	0.85	0.00
4	0.65	0.65	4	0.85	1.07
5	0.70	0.70	5	0.94	1.17
6	0.48	0.48	6	1.53	1.17
Feb.1	0.48	0.48	Aug.1	1.41	1.23
2	0.48	0.48	2	1.41	1.23
3	1.10	1.10	3	1.53	0.93
4	0.89	0.89	4	1.53	0.93
5	1.14	1.14	5	1.53	0.94
6	1.36	1.36	6	1.30	1.53
Mar.1	0.73	0.73	Sep.1	1.39	1.18
2	0.73	0.73	2	1.39	1.18
3	0.43	0.43	3	0.78	0.82
4	0.65	0.65	4	0.55	0.82
5	0.91	0.91	5	0.00	0.00
6	0.89	0.89	6	0.00	0.00
Apr.1	0.93	0.93	Oct.1	0.73	1.22
2	0.93	0.93	2	0.73	1.22
3	0.70	0.70	3	0.80	0.99
4	0.70	0.70	4	0.57	0.76
5	0.58	0.58	5	0.35	0.57
6	0.79	0.79	6	0.12	0.34
May.1	0.54	0.54	Nov.1	0.25	0.73
2	0.40	0.40	2	0.25	0.73
3	0.26	0.26	3	0.42	0.89
4	0.26	0.26	4	0.42	0.65
5	0.27	0.27	5	0.25	0.27
6	0.47	0.47	6	0.01	0.04
Jun.1	0.52	0.52	Dec.1	0.00	0.08
2	0.52	0.52	2	0.00	0.08
3	0.26	0.26	3	0.00	0.20
4	1.06	0.00	4	0.00	0.20
5	1.28	0.00	5	0.00	0.30
6	1.28	0.00	6	0.00	0.07

Table V-35.3 Comparison of Water Requirement(3/3)

Unit: lit./s/ha

	Case 3-1	Case 3-2		Case 3-1	Case 3-2
Period	Oct. 1 Mar.16	Oct. 1 Apr.16	Period	Oct. 1 Mar.16	Oct. 1 Apr.16
Jan.1	0.36	0.36	Jul.1	0.97	1.58
2	0.36	0.36	2	0.97	1.58
3	0.28	0.28	3	1.06	1.37
4	0.42	0.42	4	0.82	1.14
5	0.46	0.46	5	0.58	1.27
6	0.67	0.67	6	0.35	1.04
Feb.1	0.44	0.44	Aug.1	0.38	1.00
2	0.35	0.35	2	0.38	1.00
3	0.21	0.21	3	0.55	1.14
4	0.21	0.21	4	0.55	0.91
5	0.29	0.29	5	0.55	0.58
6	0.49	0.49	6	0.32	0.35
Mar.1	0.29	0.29	Sep.1	0.00	0.31
2	0.29	0.29	2	0.00	0.31
3	0.03	0.03	3	0.00	0.31
4	0.58	0.00	4	0.00	0.31
5	0.76	0.00	5	0.00	0.00
6	0.76	0.00	6	0.00	0.00
Apr.1	0.93	0.00	Oct.1	0.97	0.97
2	0.93	0.00	2	0.97	0.97
3	0.72	0.00	3	0.85	0.85
4	0.72	0.96	4	0.85	0.85
5	0.59	0.80	5	0.70	0.70
6	1.00	0.80	6	0.48	0.48
May.1	0.76	0.79	Nov.1	0.74	0.74
2	0.76	0.79	2	0.74	0.74
3	1.25	0.75	3	1.14	1.14
4	1.25	0.75	4	0.93	0.93
5	1.30	0.78	5	0.39	0.39
6	1.07	1.30	6	0.60	0.60
Jun.1	1.41	1.20	Dec.1	0.73	0.73
2	1.41	1.20	2	0.73	0.73
3	1.31	1.35	3	0.25	0.25
4	1.08	1.35	4	0.46	0.46
5	1.46	1.72	5	0.75	0.75
6	1.23	1.49	6	0.74	0.74

Table V-36.1 Unit Water Requirement (Paddy) Case2-2(1/2)

Period	ETo mm/d	P mm/d	Re mm	WLR mm/d	LP Area	ETc (LP) mm/d	Crop Area	Mean c.f	ETc (c) mm/d	NFR (LP) mm/d	NFR (c) mm/d	NFR Total mm/d	DR l/s/ha
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10) 2*9	(11) (7-4) *6	(12) (10+3-4) *8	(13) 11+12 +5	(14) 13/(0.55* 8.64)
Jan.1	4.3	3.0	6.8		0.67	10.7				2.6		2.6	0.55
2	4.3	3.0	6.8		0.67	10.7				2.6		2.6	0.55
3	4.3	3.0	6.1		0.67	10.7				3.1		3.1	0.65
4	4.3	3.0	6.1		0.67	10.7				3.1		3.1	0.65
5	4.5	3.0	6.0		0.67	11.0				3.4		3.4	0.70
6	4.5	3.0	6.0		0.33	11.0	0.33	1.10	5.0	1.7	0.6	2.3	0.48
Feb.1	4.4	3.0	5.9		0.33	10.8	0.33	1.10	4.8	1.6	0.6	2.3	0.48
2	4.4	3.0	5.9		0.33	10.8	0.33	1.10	4.8	1.6	0.6	2.3	0.48
3	4.8	3.0	5.0		0.67	11.2	0.33	1.10	5.3	4.2	1.1	5.2	1.10
4	4.8	3.0	5.0		0.33	11.2	0.67	1.10	5.3	2.0	2.2	4.2	0.89
5	4.6	3.0	3.6		0.33	11.0	0.67	1.10	5.1	2.4	3.0	5.4	1.14
6	4.6	3.0	3.6	1.1	0.33	11.0	0.67	1.08	5.0	2.4	2.9	6.5	1.36
Mar.1	4.0	3.0	6.0	1.1	0.33	10.5	0.67	1.08	4.3	1.5	0.9	3.5	0.73
2	4.0	3.0	6.0	1.1	0.33	10.5	0.67	1.08	4.3	1.5	0.9	3.5	0.73
3	4.4	3.0	6.7		0.33	10.8	0.67	1.08	4.8	1.4	0.7	2.1	0.43
4	4.4	3.0	6.7	1.1	0.33	10.8	0.67	1.05	4.6	1.4	0.6	3.1	0.65
5	4.5	3.0	5.6	1.1	0.33	11.0	0.67	1.05	4.7	1.8	1.4	4.3	0.91
6	4.5	3.0	5.6	2.2		11.0	1.00	1.03	4.6		2.0	4.2	0.89
Apr.1	4.4	3.0	4.2	1.1		10.8	1.00	1.03	4.5		3.3	4.4	0.93
2	4.4	3.0	4.2	1.1		10.8	1.00	1.03	4.5		3.3	4.4	0.93
3	3.9	3.0	3.7			10.5	1.00	1.03	4.0		3.3	3.3	0.70
4	3.9	3.0	3.7	1.1		10.5	0.67	1.03	4.0		2.2	3.3	0.70
5	4.7	3.0	5.4	1.1		11.1	0.67	1.03	4.8		1.6	2.7	0.58
6	4.7	3.0	5.4	2.2		11.1	0.67	1.00	4.7		1.5	3.7	0.79
May.1	4.4	3.0	5.2	1.1		10.8	0.67	1.00	4.4		1.5	2.6	0.54
2	4.4	3.0	5.2	1.1		10.8	0.33	1.05	4.6		0.8	1.9	0.40
3	4.4	3.0	3.9			10.8	0.33	1.05	4.6		1.2	1.2	0.26
4	4.4	3.0	3.9			10.8	0.33	1.05	4.6		1.2	1.2	0.26
5	4.0	3.0	3.3			10.5	0.33	1.05	4.2		1.3	1.3	0.27
6	4.0	3.0	3.3	1.1		10.5	0.33	0.95	3.8		1.2	2.3	0.47
Jun.1	4.0	3.0	2.7	1.1		10.5	0.33	0.95	3.8		1.4	2.5	0.52
2	4.0	3.0	2.7	1.1		10.5	0.33	0.95	3.8		1.4	2.5	0.52
3	3.9	3.0	3.0			10.5	0.33	0.95	3.7		1.2	1.2	0.26
4	3.9	3.0	3.0			10.5						0.0	0.00
5	4.6	3.0	1.9			11.0						0.0	0.00
6	4.6	3.0	1.9			11.0						0.0	0.00

Table V-36.2 Unit Water Requirement (Paddy) Case2-2(2/2)

Period	ETo mm/d	P mm/d	Re mm	WLR mm/d	LP Area	ETc (LP) mm/d	Crop Area	Mean c.f	ETc (c) mm/d	NFR (LP) mm/d	NFR (c) mm/d	NFR Total mm/d	DR l/s/ha
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10) 2*9	(11) (7-4)*6	(12) (10+3-4)*8	(13) 11+12+5	(14) 13/(0.55*8.64)
Jul.1	4.5	3.0	2.4			11.0						0.0	0.00
2	4.5	3.0	2.4			11.0						0.0	0.00
3	4.4	3.0	3.2			10.8						0.0	0.00
4	4.4	3.0	3.2		0.67	10.8				5.1		5.1	1.07
5	4.5	3.0	2.7		0.67	11.0				5.6		5.6	1.17
6	4.5	3.0	2.7		0.67	11.0				5.6		5.6	1.17
Aug.1	4.4	3.0	2.1		0.67	10.8				5.8		5.8	1.23
2	4.4	3.0	2.1		0.67	10.8				5.8		5.8	1.23
3	4.3	3.0	2.5		0.33	10.7	0.33	1.10	4.7	2.7	1.7	4.4	0.93
4	4.3	3.0	2.5		0.33	10.7	0.33	1.10	4.7	2.7	1.7	4.4	0.93
5	4.2	3.0	2.4		0.33	10.7	0.33	1.10	4.6	2.7	1.7	4.5	0.94
6	4.2	3.0	2.4		0.67	10.7	0.33	1.10	4.6	5.6	1.7	7.3	$\boxed{1.53}$
Sep.1	4.1	3.0	2.9		0.33	10.6	0.67	1.10	4.5	2.5	3.1	5.6	1.18
2	4.1	3.0	2.9		0.33	10.6	0.67	1.10	4.5	2.5	3.1	5.6	1.18
3	4.2	3.0	5.8	1.1	0.33	10.7	0.67	1.08	4.5	1.6	1.2	3.9	0.82
4	4.2	3.0	5.8	1.1	0.33	10.7	0.67	1.08	4.5	1.6	1.2	3.9	0.82
5	3.9	3.0	10.7	1.1	0.33	10.5	0.67	1.08	4.2	-0.1	-2.3	-1.3	*0.00
6	3.9	3.0	10.7		0.33	10.5	0.67	1.08	4.2	-0.1	-2.3	-2.4	*0.00
Oct.1	4.2	3.0	3.8	1.1	0.33	10.7	0.67	1.05	4.4	2.3	2.4	5.8	1.22
2	4.2	3.0	3.8	1.1	0.33	10.7	0.67	1.05	4.4	2.3	2.4	5.8	1.22
3	3.9	3.0	4.5	2.2		10.5	1.00	1.03	4.0		2.5	4.7	0.99
4	3.9	3.0	4.5	1.1		10.5	1.00	1.03	4.0		2.5	3.6	0.76
5	4.1	3.0	5.6	1.1		10.6	1.00	1.03	4.2		1.6	2.7	0.57
6	4.1	3.0	5.6			10.6	1.00	1.03	4.2		1.6	1.6	0.34
Nov.1	4.1	3.0	3.7	1.1		10.6	0.67	1.03	4.2		2.4	3.5	0.73
2	4.1	3.0	3.7	1.1		10.6	0.67	1.03	4.2		2.4	3.5	0.73
3	4.3	3.0	4.3	2.2		10.7	0.67	1.00	4.3		2.0	4.2	0.89
4	4.3	3.0	4.3	1.1		10.7	0.67	1.00	4.3		2.0	3.1	0.65
5	4.1	3.0	6.7	1.1		10.6	0.33	1.05	4.3		0.2	1.3	0.27
6	4.1	3.0	6.7			10.6	0.33	1.05	4.3		0.2	0.2	0.04
Dec.1	4.1	3.0	6.1			10.6	0.33	1.05	4.3		0.4	0.4	0.08
2	4.1	3.0	6.1			10.6	0.33	1.05	4.3		0.4	0.4	0.08
3	4.5	3.0	7.7	1.1		11.0	0.33	0.95	4.3		-0.1	1.0	0.20
4	4.5	3.0	7.7	1.1		11.0	0.33	0.95	4.3		-0.1	1.0	0.20
5	4.4	3.0	6.2	1.1		10.8	0.33	0.95	4.2		0.3	1.4	0.30
6	4.4	3.0	6.2			10.8	0.33	0.95	4.2		0.3	0.3	0.07

Table V-37 Unit Water Requirement for Each Block

Unit: lit./s/ha

	C1	C2	C3		C1	C2	C3
Period	Jan. 1 Jul.16	Jan. 1 Jul.16	Feb.11 Aug.26	Period	Jan. 1 Jul.16	Jan. 1 Jul.16	Feb.11 Aug.26
Jan.1	0.82	0.82	0.00	Jul.1	0.00	0.00	0.00
2	0.82	0.82	0.00	2	0.00	0.00	0.00
3	0.97	0.97	0.00	3	0.00	0.00	0.00
4	0.97	0.97	0.00	4	1.60	1.60	0.00
5	1.05	1.05	0.00	5	1.75	1.75	0.00
6	0.41	1.05	0.00	6	1.75	1.75	0.00
Feb.1	0.41	1.03	0.00	Aug.1	$\boxed{1.83}$	$\boxed{1.83}$	0.00
2	0.41	1.03	0.00	2	$\boxed{1.83}$	$\boxed{1.83}$	0.00
3	0.69	1.30	1.30	3	1.10	1.73	0.00
4	0.69	0.69	1.30	4	1.10	1.73	0.00
5	0.94	0.94	1.56	5	1.10	1.75	0.00
6	1.58	0.94	1.56	6	1.10	1.75	$\boxed{1.75}$
Mar.1	0.95	0.29	0.95	Sep.1	0.97	0.97	1.62
2	0.95	0.29	0.95	2	0.97	0.97	1.62
3	0.19	0.24	0.86	3	1.03	0.38	1.03
4	0.19	0.89	0.86	4	1.03	0.38	1.03
5	0.45	1.14	1.14	5	0.00	0.00	0.00
6	1.05	1.14	0.49	6	0.00	0.00	0.00
Apr.1	1.32	0.72	0.77	Oct.1	0.76	1.45	1.45
2	1.32	0.72	0.77	2	0.76	1.45	1.45
3	0.63	0.71	0.76	3	1.16	1.24	0.59
4	0.00	1.33	0.76	4	1.16	0.55	0.59
5	0.00	1.13	0.58	5	0.97	0.36	0.40
6	0.00	1.13	1.23	6	0.27	0.36	0.40
May.1	0.00	0.42	1.20	Nov.1	0.00	1.37	0.80
2	0.00	0.00	1.20	2	0.00	1.37	0.80
3	0.00	0.00	0.78	3	0.00	1.28	1.37
4	0.00	0.00	0.78	4	0.00	0.59	1.37
5	0.00	0.00	0.82	5	0.00	0.00	0.82
6	0.00	0.00	1.43	6	0.00	0.00	0.13
Jun.1	0.00	0.00	1.56	Dec.1	0.00	0.00	0.25
2	0.00	0.00	1.56	2	0.00	0.00	0.25
3	0.00	0.00	0.78	3	0.00	0.00	0.61
4	0.00	0.00	0.00	4	0.00	0.00	0.61
5	0.00	0.00	0.00	5	0.00	0.00	0.90
6	0.00	0.00	0.00	6	0.00	0.00	0.21

3.4 Diversion Requirement

Three basic plans for irrigation are studied in which water is delivered to the existing settlers, to the existing settlers plus new settlers, and to the settlers for oil palm plantations. Finally the last one (Plan-3) is adopted as mentioned in the Clause 2.1 (7), Chapter 2.

Intake discharges and remaining river discharges for three cases are given the following table.

Table V-38 COMPARISON OF REMAINING RIVER DISCHARGE

	Plan-1	Plan-2	Plan-3
Period	Aug. last 10 days	Aug.mid. 10 days	Aug.mid. 10 days
Maximum Intake Discharge	3.33 m^3/s	6.43 m^3/s	6.45 m^3/s
River Discharge	9.21	9.21	9.21
The Remaining River Discharge	7.19	5.30	5.28

For the above Plan-3, domestic water of 0.02 m^3/s is included. The objective supply of domestic water is based on about 3,000 households, 15,000 persons along main canal. The quantity of water supply is planned to be 10 lit/day/person.

Furthermore the maximum irrigable area is confirmed using the above relationship between river discharge and intake discharge in the Table V-34. As the result, the maximum dry paddy area becomes 8,373 ha if the field can be prepared.

Table V-39 RIVER DISCHARGE & DIVERSION REQUIREMENT

Case-1: Irrigable area 2,175 ha
Case-2: Irrigable area 4,200 ha
Case-3: Irrigable area 4,200 ha

			Case-1		Case-2		Case-3	
Period	Unit Water Req.	River Dis.	Diversion Req.	Surplus Dis.	Diversion Req.	Surplus Dis.	Diversion Req.	Surplus Dis.
	l/s/ha	m3/s	m3/s	m3/s	m3/s	m3/s	m3/s	m3/s
Jan.1	0.55	51.67	1.20	50.47	2.31	49.36	2.33	49.34
2	0.65	34.31	1.41	32.90	2.73	31.58	2.75	31.56
3	0.70	34.59	1.52	33.07	2.94	31.65	2.96	31.63
Feb.1	0.48	35.31	1.04	34.27	2.02	33.29	2.04	33.27
2	1.10	22.44	2.39	20.05	4.62	17.82	4.64	17.80
3	1.36	25.62	2.96	22.66	5.71	19.91	5.73	19.89
Mar.1	0.73	39.68	1.59	38.09	3.07	36.61	3.09	36.59
2	0.65	44.21	1.41	42.80	2.73	41.48	2.75	41.46
3	0.91	40.79	1.98	38.81	3.82	36.97	3.84	36.95
Apr.1	0.93	28.06	2.02	26.04	3.91	24.15	3.93	24.13
2	0.70	23.48	1.52	21.96	2.94	20.54	2.96	20.52
3	0.79	37.14	1.72	35.42	3.32	33.82	3.34	33.80
May 1	0.54	25.53	1.17	24.36	2.27	23.26	2.29	23.24
2	0.26	21.56	0.57	20.99	1.09	20.47	1.11	20.45
3	0.47	18.85	1.02	17.83	1.97	16.88	1.99	16.86
Jun.1	0.52	15.50	1.13	14.37	2.18	13.32	2.20	13.30
2	0.26	17.24	0.57	16.67	1.09	16.15	1.11	16.13
3	0.00	11.45	0.00	11.45	0.00	11.45	0.02	11.43
Jul.1	0.00	16.61	0.00	16.61	0.00	16.61	0.02	16.59
2	1.07	16.47	2.33	14.14	4.49	11.98	4.51	11.96
3	1.17	19.48	2.54	16.94	4.91	14.57	4.93	14.55
Aug.1	1.23	16.66	2.68	13.98	5.17	11.49	5.19	11.47
2	0.93	9.21	2.02	7.19	3.91	5.30	3.93	5.28
3	1.53	12.83	3.33	9.50	6.43	6.40	6.45	6.38
Sep.1	1.18	27.69	2.57	25.12	4.96	22.73	4.98	22.71
2	0.82	26.48	1.78	24.70	3.44	23.04	3.46	23.02
3	0.00	36.82	0.00	36.82	0.00	36.82	0.02	36.80
Oct.1	1.22	27.23	2.65	24.58	5.12	22.11	5.14	22.09
2	0.99	31.21	2.15	29.06	4.16	27.05	4.18	27.03
3	0.57	30.57	1.24	29.33	2.39	28.18	2.41	28.16
Nov.1	0.73	32.37	1.59	30.78	3.07	29.30	3.09	29.28
2	0.89	24.25	1.94	22.31	3.74	20.51	3.76	20.49
3	0.27	36.08	0.59	35.49	1.13	34.95	1.15	34.93
Dec.1	0.08	30.81	0.17	30.64	0.34	30.47	0.36	30.45
2	0.20	37.75	0.44	37.32	0.84	36.91	0.86	36.89
3	0.30	39.83	0.65	39.18	1.26	38.57	1.28	38.55

Note; For the Case-3, domestic water of 0.02 m3/sec are included.

Table V-40 MAXIMUM IRRIGABLE AREA (CASE STUDY)

Case2-2: Wet paddy, Jan.1
Dry paddy, Jul.16

Period	Unit Water Req.	River Dis.	Domestic Water	Maximum irrigable Area	Remarks
	1/s/ha	m3/s	m3/s	ha	
Jan.1	0.55	51.67	0.02	93909	
2	0.65	34.31	0.02	52754	
3	0.70	34.59	0.02	49386	
Feb.1	0.48	35.31	0.02	73521	
2	1.10	22.44	0.02	20382	
3	1.36	25.62	0.02	18824	
Mar.1	0.73	39.68	0.02	54329	
2	0.65	44.21	0.02	67985	
3	0.91	40.79	0.02	44802	
Apr.1	0.93	28.06	0.02	30151	
2	0.70	23.48	0.02	33514	
3	0.79	37.14	0.02	46987	
May 1	0.54	25.53	0.02	47241	
2	0.26	21.56	0.02	82846	
3	0.47	18.85	0.02	40064	
Jun.1	0.52	15.50	0.02	29769	
2	0.26	17.24	0.02	66231	
3	0.00	11.45	0.02		
Jul.1	0.00	16.61	0.02		
2	1.07	16.47	0.02	15374	
3	1.17	19.48	0.02	16632	
Aug.1	1.23	16.66	0.02	13528	
2	0.93	9.21	0.02	9882	
3	1.53	12.83	0.02	8373	< Max.
Sep.1	1.18	27.69	0.02	23449	Dry paddy
2	0.82	26.48	0.02	32268	
3	0.00	36.82	0.02		
Oct.1	1.22	27.23	0.02	22303	
2	0.99	31.21	0.02	31505	
3	0.57	30.57	0.02	53596	
Nov.1	0.73	32.37	0.02	44315	
2	0.89	24.25	0.02	27225	
3	0.27	36.08	0.02	133556	
Dec.1	0.08	30.81	0.02	384875	
2	0.20	37.75	0.02	188650	
3	0.30	39.83	0.02	132700	

CHAPTER 4 IRRIGATION AND DRAINAGE PLAN

4.1 INTAKE FACILITIES

4.1.1 General

The objective area for the study on the Air Selagan Irrigation Project is estimated at 14,800 ha on both the sides of the Selagan river. The weir is proposed as the intake facility for the irrigation to the objective area.

4.1.2 Study on the location

In viewing the ground elevation in the objective area and the intake water level, the site of weir is proposed at a certain place of the Selagan river within about 4 km from the upstream part of the river near Kp. Lubuk Sahung to the downstream part near Kp. Surian Bungkal.

As a result of the study by the available topographical maps and the field reconnaissance, two (2) weir sites are compared taking the following points into consideration.

a) Factors to select the site

- Line of existing river,
- River bed elevation, and shape and elevation of both the sides of the river,
- Location and condition of tributaries,
- Geological condition,
- Back water level at the time of flood which might influence to the villages in the upstream, and
- Construction method by temporary diversion channel or Coupure method.

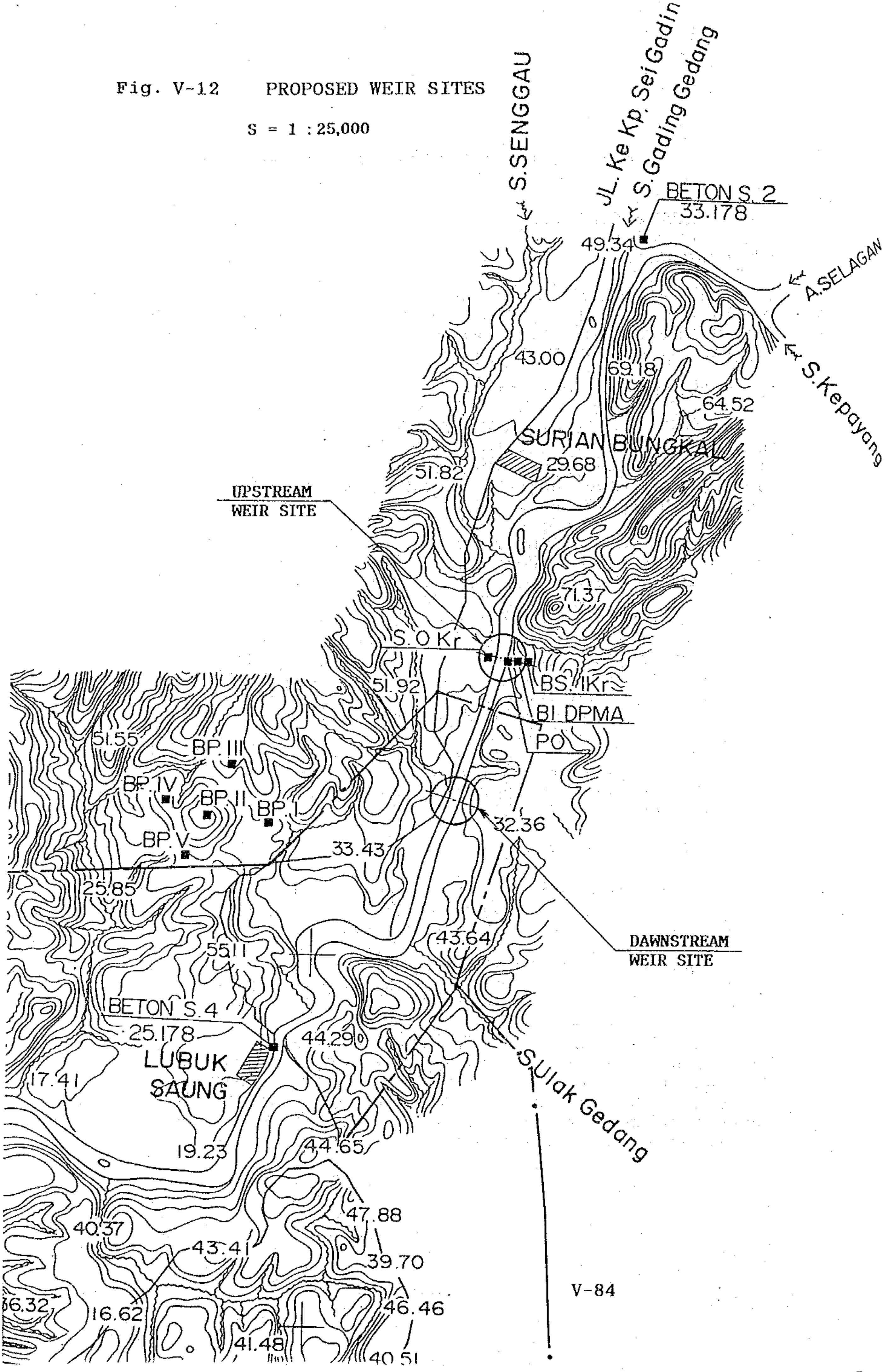
b) Comparative sites

- Downstream site : at the place about 2.3 km in the upstream from the Kp. Lubuk Sahung bridge (Plan of DPU, Province)
- Upstream site : at the place about 0.8 km in the upstream from the downstream site (Plan of D.P.M.A)

The location of the weir and the geological investigation has been carried out by the Directorate of Irrigation II, Provincial Public Work Office and the Hydraulic Institute in Bandung. During the Feasibility Study, additional geological investigation was carried out for the confirmation of bed condition and soil materials.

Fig. V-12 PROPOSED WEIR SITES

S = 1 : 25,000



c) Outline of downstream site and upstream site for weir

Table V-41 COMPARISON OF PROPOSED SITES FOR WEIR

Item	Downstream Site	Upstream Site
Location from estuary	48.7 km	49.5 km
Existing river width	64 m	50 m
Existing river bed elevation	22.20 m	21.00 m
Existing river slope	1 : 500	1 : 500
Catchment area	375 km^2	374 km^2
Planning flood discharge	1,000 m^3/s	997 m^3/s
Planning width of weir	74 m	73 m
Planning elevation of weir crest	26.00 m	26.35 m
Planning height of weir	3.80 m	5.50 m
Planning elevation of river bank	31.55 m	31.90 m
Planning flood elevation	30.05 m	30.40 m
Construction method	Temporary diversion	Temporary diversion
Influence of back water to Kp. Surian Bungkal at the time of flood	None	None
Ratio of economical comparison	1.00	1.23

The conditions of the above comparative study are as follows:

- The planning flood discharge is estimated by the hydrological data at downstream site.

  Q in one of 100 year probability : 1,000 m^3/s
  Q in one of 1,000 year probability : 1,300 m^3/s

- The planning width (B) of weir is based on the IDS-HEADWORKS:

  i. Existing river width x 1.2 and
  ii. Maximum flood discharge flow per meter, 14 m^3/s

In case of downstream site

i. 64 x 1.2 = 77 m > B
ii. 1,000 ÷ + width of pier = 74 m < B
B = 74 m is adopted

In case of upstream site

i. 50 x 1.2 = 60 m > B
ii. 997 ÷ 14 + width of pier = 73 m < B
B = 73 m is adopted

- The planning elevation of weir crest:

In case of downstream site

* Right side

Water level at BB.4	=	WL. 14.00 m
Head loss of canal, 22 km x 1/2,600	=	8.50 m
Head loss at the intake	=	0.40 m
Allowance at the crest	=	0.10 m
Total		EL. 23.00 m

* Light side

Ground elevation at SP4	=	GL. 19.00 m
Head loss of main canal 17 km x 1/2,600	=	6.50 m
Head loss at the intake	=	0.40 m
Allowance at the crest	=	0.10 m
Total		EL. 26.00 m

In case of upstream site

Crest elevation of downstream site	=	EL. 26.00 m
Head loss of canal 800 x 1/2,600	=	0.35 m
Total		EL. 26.35 m

In the above, the head loss of canal is obtained by the average gradient of canal including the head loss due to the attached structures.

- The planning height of weir:
Height of weir = Planning elevation of crest - existing river bed elevation

Downstream plan : EL. 26.00 - GL. 22.20 = 3.80 m
Upstream plan : EL. 26.35 - GL. 21.00 = 5.35 m

- The planning flood water level (HWL):
  HWL = Planning crest elevation + Critical water depth
  Downstream plan = EL. 26.00 + 4.05 = HWL 30.05 m
  Upstream plan = EL. 26.35 + 4.05 = HWL 30.40 m

- The planning elevation of river bank:
  River bank elevation = HWL (in 100 year probability) + Fb (1.50 m)
  Downstream plan = HWL 30.05 + 1.50 = EL. 31.55
  Upstream plan = HWL 30.40 + 1.50 = EL. 31.90

- Construction method:
  The temporary diversion is proposed for the construction method because the river line is straight and the land space assured in the right side.

Table V-42 COMPARISON OF WEIR SITE PLAN

Item	Unit	Downstream Plan	Upstream Plan
Width of weir	m	74.00	73.00
Height of weir body	m	26.00-22.20=3.80	26.35-21.00=5.35
Concrete Volume	m^3	15,700	16,400
Canal length	m	-	800
	Unit Price		
Weir	RP/m^3 253,000	10^3Rp 3,972,100	10^3Rp 4,149,200
Main canal	RP/m 900,000	-	720,000
Total		3,972,100	4,869,200
Ratio		1.00	1.23

- The back water at the time of flood:
  The influence of the back water at the time of flood to Kp. Surian Bungkal in the upstream is studied by the hydraulic calculation taking the planning crest elevation as the control point. The result is shown in Fig. V-13 and Table V-43.

- The flood discharge per unit width of weir and the maximum overflow depth:

According to the Irrigation Design Standards (Headworks), the maximum overflow depth (H max) is 3.5 m to 4.5 m in the case that the flood discharge per unit width of weir (q) is 12 $m^3/s/m$ to 14 $m^3/s/m$.

In case of the downstream plan,
$q = 1,000\ m^3/s/(74.0\ m - 2.20\ m) = 13.9\ m^3/s/m$
$H\ max = 0.467 \times 13.9^{2/3} \times 1.5 = 4.05\ m$

In case of the upstream plan,
$q = 997\ m^3/s/(73.0\ m - 2.20\ m) = 14.1\ m^3/s/m$
$H\ max = 0.467 \times 14.1^{2/3} \times 1.5 = 4.09\ m$

Fig. V-13 PROFILE OF FLOOD WATER SURFACE

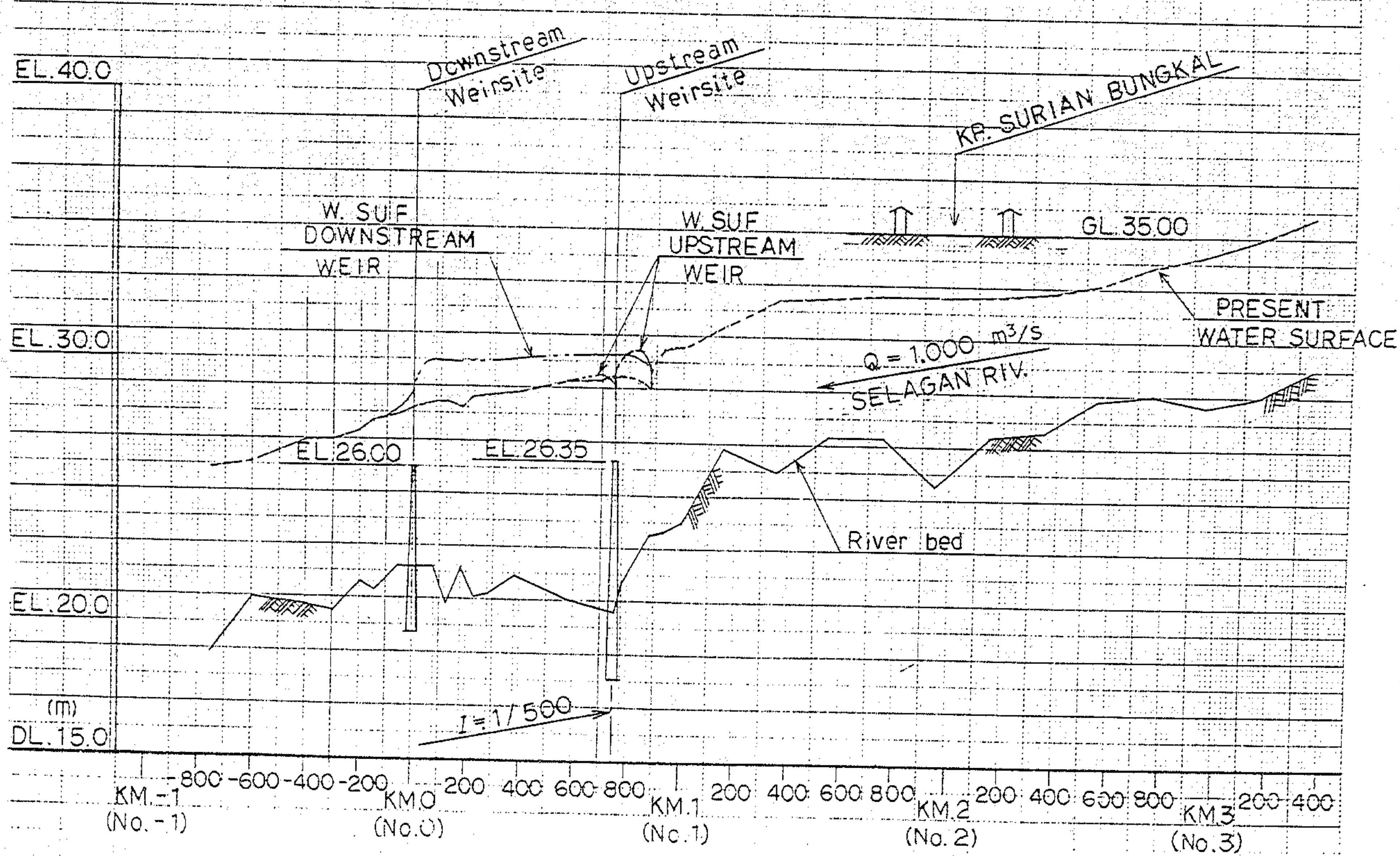


Table V-43.1 HYDRAULIC CALCULATION OF SELAGAN RIVER DURING FLOOD(1/3)
PRESENT CONDITION

Station	Distance (m)	Discharge (m3/s)	Water Depth (m)	Water Level (m)	Area of Flow (m2)	Water Surface Width (m)	Hydraulic Mean Depth (m)	Coefficient of Roughness	Velocity (m/s)	Velocity Head (m)	Hydraulic Gradient	Friction Loss (m)	Energy Height (m)
No. 0- 745.000	0.000	1000.00	7.033	25.893	359.564	87.027	3.923	0.040	2.781	0.395	0.002000	0.000	26.287
No. 0- 595.000	150.000	1000.00	5.210	26.160	297.314	91.314	3.152	0.040	3.363	0.577	0.003917	0.444	26.738
No. 0- 395.000	200.000	1000.00	6.194	26.944	386.312	90.434	4.124	0.040	2.589	0.342	0.001621	0.554	27.286
No. 0- 295.000	100.000	1000.00	6.499	27.009	306.777	94.806	3.064	0.040	3.260	0.542	0.003820	0.272	27.551
No. 0- 200.000	95.000	1000.00	5.686	27.296	286.189	81.240	3.422	0.040	3.494	0.623	0.003788	0.361	27.919
No. 0- 150.000	50.000	1000.00	6.387	27.697	376.864	100.299	3.656	0.040	2.653	0.359	0.002000	0.145	28.056
No. 0- 100.000	50.000	1000.00	6.070	27.830	385.401	123.212	3.032	0.040	2.595	0.343	0.002455	0.111	28.173
No. 0- 50.000	50.000	1000.00	5.652	27.932	365.629	119.802	2.964	0.040	2.735	0.382	0.002811	0.132	28.313
No. 0+ 0.000	50.000	1000.00	5.993	28.193	449.577	199.237	2.236	0.040	2.224	0.252	0.002708	0.138	28.445
No. 0+ 75.000	75.000	1000.00	6.252	28.422	554.240	188.717	2.913	0.040	1.804	0.166	0.001252	0.148	28.588
No. 0+ 125.000	50.000	1000.00	7.645	28.455	532.958	142.317	3.667	0.040	1.876	0.180	0.000996	0.056	28.634
No. 0+ 175.000	50.000	1000.00	5.977	28.197	295.782	100.769	2.837	0.040	3.381	0.583	0.004552	0.139	28.781
No. 0+ 225.000	50.000	1000.00	7.565	28.645	409.047	149.950	2.697	0.040	2.445	0.305	0.002547	0.177	28.950
No. 0+ 275.000	50.000	1000.00	7.526	28.696	376.112	76.958	4.673	0.040	2.659	0.361	0.001448	0.100	29.057
No. 0+ 375.000	100.000	1000.00	6.863	28.823	349.668	85.004	3.934	0.040	2.860	0.417	0.002107	0.178	29.240
No. 0+ 572.000	197.000	1000.00	8.177	29.237	394.999	78.349	4.817	0.040	2.532	0.327	0.001260	0.332	29.564
No. 0+ 703.000	131.000	1000.00	8.565	29.285	335.526	57.053	5.421	0.040	2.980	0.453	0.001492	0.180	29.738
No. 0+ 755.000	52.000	1000.00	8.798	29.418	360.011	72.853	4.616	0.040	2.778	0.394	0.001606	0.081	29.812
No. 0+ 800.000	45.000	1000.00	7.738	29.438	335.721	64.216	5.000	0.040	2.979	0.453	0.001660	0.073	29.891
No. 0+ 875.000	75.000	1000.00	5.481	29.031	175.771	53.217	3.223	0.040	5.689	1.651	0.010879	0.470	30.582 *
No. 0+ 930.000	55.000	1000.00	6.827	30.457	286.933	81.397	3.474	0.040	3.485	0.620	0.003693	0.401	31.077
No. 0+ 950.000	20.000	1000.00	6.555	30.555	290.307	83.329	3.434	0.040	3.445	0.605	0.003664	0.074	31.160
No. 1+ 150.000	200.000	1000.00	4.592	31.392	293.095	100.567	2.884	0.040	3.412	0.594	0.004537	0.820	31.986
No. 1+ 350.000	200.000	1000.00	6.400	32.400	820.024	230.003	3.553	0.040	1.219	0.076	0.000439	0.498	32.476
No. 1+ 550.000	200.000	1000.00	5.144	32.504	958.096	282.569	3.384	0.040	1.044	0.056	0.000343	0.078	32.559
No. 1+ 750.000	200.000	1000.00	5.272	32.572	1288.670	366.589	3.508	0.040	0.776	0.031	0.000181	0.052	32.603
No. 1+ 950.000	200.000	1000.00	7.103	32.603	1237.970	381.442	3.231	0.040	0.808	0.033	0.000219	0.040	32.636
No. 2+ 150.000	200.000	1000.00	5.253	32.653	1018.820	346.933	2.933	0.040	0.982	0.049	0.000367	0.059	32.702
No. 2+ 350.000	200.000	1000.00	5.187	32.787	468.391	224.442	2.084	0.040	2.135	0.233	0.002740	0.311	33.020
No. 2+ 550.000	200.000	1000.00	4.397	33.147	313.582	101.472	3.061	0.040	3.189	0.519	0.003661	0.640	33.666
No. 2+ 750.000	200.000	1000.00	4.865	33.865	329.701	101.380	3.216	0.040	3.033	0.469	0.003101	0.676	34.334
No. 2+ 950.000	200.000	1000.00	5.712	34.312	287.802	67.249	4.149	0.040	3.475	0.616	0.002897	0.600	34.928
No. 3+ 150.000	200.000	1000.00	5.962	34.962	301.949	79.656	3.678	0.040	3.312	0.560	0.003091	0.599	35.521
No. 3+ 350.000	200.000	1000.00	5.765	35.765	430.362	145.308	2.950	0.040	2.324	0.275	0.002042	0.513	36.041

* Critical Depth

Table V-43.2 HYDRAULIC CALCULATION OF SELAGAN RIVER DURING FLOOD(2/3)
DOWNSTREAM PLAN

Station	Distance (m)	Discharge (m3/s)	Water Depth (m)	Water Level (m)	Area of Flow (m2)	Water Surface Width (m)	Hydraulic Mean Depth (m)	Coefficient of Roughness	Velocity (m/s)	Velocity Head (m)	Hydraulic Gradient	Friction Loss (m)	Energy Height (m)
No. 0- 150.000	0.000	1000.00	6.387	27.697	376.887	100.300	3.656	0.040	2.653	0.359	0.002000	0.000	28.056
No. 0- 100.000	50.000	1000.00	5.781	27.781	358.516	116.386	2.972	0.037	2.789	0.397	0.002544	0.114	28.178
No. 0- 50.000	50.000	1000.00	5.969	28.069	501.887	184.805	2.604	0.038	1.992	0.203	0.001582	0.103	28.272
No. 0+ 0.000	50.000	1000.00	2.700	28.700	194.406	72.000	2.204	0.015	5.144	1.350	0.002075	0.091	30.050 *
No. 0+ 50.000	50.000	1000.00	7.656	29.956	601.832	83.223	6.551	0.023	1.662	0.141	0.000122	0.055	30.097
No. 0+ 100.000	50.000	1000.00	7.634	30.034	781.468	206.425	3.711	0.039	1.280	0.084	0.000423	0.014	30.118
No. 0+ 125.000	25.000	1000.00	9.223	30.033	768.419	156.197	4.811	0.040	1.301	0.086	0.000334	0.009	30.119
No. 0+ 175.000	50.000	1000.00	7.744	29.964	503.623	134.549	3.643	0.040	1.986	0.201	0.001125	0.036	30.166
No. 0+ 225.000	50.000	1000.00	9.004	30.084	651.384	186.076	3.466	0.040	1.535	0.120	0.000719	0.046	30.204
No. 0+ 275.000	50.000	1000.00	8.866	30.036	491.595	95.372	4.960	0.040	2.034	0.211	0.000783	0.038	30.247
No. 0+ 375.000	100.000	1000.00	8.166	30.107	465.595	95.342	4.676	0.040	2.148	0.235	0.000944	0.086	30.342
No. 0+ 572.000	197.000	1000.00	9.225	30.286	482.881	89.223	5.186	0.040	2.071	0.219	0.000764	0.168	30.504
No. 0+ 703.000	131.000	1000.00	9.562	30.282	394.697	61.671	5.894	0.040	2.534	0.328	0.000965	0.113	30.610
No. 0+ 755.000	52.000	1000.00	9.756	30.376	431.279	75.936	5.255	0.040	2.319	0.274	0.000942	0.050	30.651
No. 0+ 800.000	45.000	1000.00	8.682	30.383	398.777	70.060	5.436	0.040	2.508	0.321	0.001052	0.045	30.703
No. 0+ 875.000	75.000	1000.00	6.426	29.976	229.163	59.829	3.731	0.040	4.364	0.972	0.005265	0.237	30.947
No. 0+ 930.000	55.000	1000.00	6.977	30.607	299.281	83.494	3.533	0.040	3.341	0.570	0.003320	0.236	31.176
No. 0+ 950.000	20.000	1000.00	6.591	30.691	301.707	84.146	3.532	0.040	3.314	0.560	0.003268	0.066	31.251
No. 1+ 150.000	200.000	1000.00	4.640	31.440	297.983	100.761	2.926	0.040	3.356	0.575	0.004306	0.757	32.015
No. 1+ 350.000	200.000	1000.00	6.406	32.407	821.486	230.161	3.557	0.040	1.217	0.076	0.000437	0.474	32.482
No. 1+ 550.000	200.000	1000.00	5.149	32.510	959.757	282.680	3.389	0.040	1.042	0.055	0.000341	0.078	32.565
No. 1+ 750.000	200.000	1000.00	5.278	32.578	1290.700	366.711	3.513	0.040	0.775	0.031	0.000180	0.052	32.608
No. 1+ 950.000	200.000	1000.00	7.108	32.608	1240.010	381.517	3.236	0.040	0.806	0.033	0.000217	0.040	32.642
No. 2+ 150.000	200.000	1000.00	5.258	32.658	1020.590	347.081	2.937	0.040	0.980	0.049	0.000365	0.058	32.707
No. 2+ 350.000	200.000	1000.00	5.191	32.792	469.363	224.697	2.086	0.040	2.131	0.232	0.002725	0.309	33.023
No. 2+ 550.000	200.000	1000.00	4.399	33.149	313.775	101.491	3.062	0.040	3.187	0.518	0.003655	0.638	33.667
No. 2+ 750.000	200.000	1000.00	4.866	33.866	329.766	101.387	3.216	0.040	3.032	0.469	0.003099	0.675	34.335
No. 2+ 950.000	200.000	1000.00	5.713	34.313	287.823	67.250	4.149	0.040	3.474	0.616	0.002897	0.600	34.928
No. 3+ 150.000	200.000	1000.00	5.962	34.962	301.963	79.658	3.678	0.040	3.312	0.560	0.003090	0.599	35.522
No. 3+ 350.000	200.000	1000.00	5.766	35.766	430.375	145.310	2.950	0.040	2.324	0.275	0.002042	0.513	36.041

* Critical Depth

Table V-43.3 HYDRAULIC CALCULATION OF SELAGAN RIVER DURING FLOOD(3/3)
UPSTREAM PLAN

Station	Distance (m)	Discharge (m3/s)	Water Depth (m)	Water Level (m)	Area of Flow (m2)	Water Surface Width (m)	Hydraulic Mean Depth (m)	Coefficient of Roughness	Velocity (m/s)	Velocity Head (m)	Hydraulic Gradient	Friction Loss (m)	Energy Height (m)
No. 0+ 572.000	0.000	1000.00	8.177	29.237	395.032	78.353	4.818	0.040	2.531	0.327	0.001260	0.000	29.564
No. 0+ 655.000	83.000	1000.00	8.658	29.458	552.672	91.810	5.707	0.037	1.809	0.167	0.000450	0.071	29.625
No. 0+ 705.000	50.000	1000.00	8.619	29.519	629.209	73.000	6.973	0.035	1.589	0.129	0.000232	0.017	29.648
No. 0+ 730.000	25.000	1000.00	8.529	29.529	622.623	73.000	6.914	0.035	1.606	0.132	0.000240	0.006	29.661
No. 0+ 755.000	25.000	1000.00	2.725	29.075	193.499	71.000	2.215	0.015	5.168	1.363	0.002081	0.029	30.438 *
No. 0+ 780.000	25.000	1000.00	9.296	30.346	678.619	73.000	7.409	0.035	1.474	0.111	0.000184	0.028	30.457
No. 0+ 805.000	25.000	1000.00	9.279	30.379	763.491	91.559	7.693	0.035	1.310	0.088	0.000138	0.004	30.467
No. 0+ 855.000	50.000	1000.00	9.123	30.323	559.604	81.216	6.411	0.037	1.787	0.163	0.000371	0.013	30.486
No. 0+ 875.000	20.000	1000.00	5.481	29.031	175.771	53.217	3.223	0.040	5.689	1.651	0.010879	0.112	30.682 *
No. 0+ 930.000	55.000	1000.00	6.827	30.457	286.933	81.397	3.474	0.040	3.485	0.620	0.003693	0.401	31.077
No. 0+ 950.000	20.000	1000.00	6.555	30.555	290.307	83.329	3.434	0.040	3.445	0.605	0.003664	0.074	31.160
No. 1+ 150.000	200.000	1000.00	4.592	31.392	293.095	100.567	2.884	0.040	3.412	0.594	0.004537	0.820	31.986
No. 1+ 350.000	200.000	1000.00	6.400	32.400	820.024	230.003	3.553	0.040	1.219	0.076	0.000439	0.498	32.476
No. 1+ 550.000	200.000	1000.00	5.144	32.504	958.096	282.569	3.384	0.040	1.044	0.056	0.000343	0.078	32.559
No. 1+ 750.000	200.000	1000.00	5.272	32.572	1288.670	366.589	3.508	0.040	0.776	0.031	0.000181	0.052	32.603
No. 1+ 950.000	200.000	1000.00	7.103	32.603	1237.970	381.442	3.231	0.040	0.808	0.033	0.000219	0.040	32.636
No. 2+ 150.000	200.000	1000.00	5.253	32.653	1018.820	346.933	2.933	0.040	0.982	0.049	0.000367	0.059	32.702
No. 2+ 350.000	200.000	1000.00	5.187	32.787	468.391	224.442	2.084	0.040	2.135	0.233	0.002740	0.311	33.020
No. 2+ 550.000	200.000	1000.00	4.397	33.147	313.582	101.472	3.061	0.040	3.189	0.519	0.003661	0.640	33.666
No. 2+ 750.000	200.000	1000.00	4.865	33.865	329.701	101.380	3.216	0.040	3.033	0.469	0.003101	0.676	34.334
No. 2+ 950.000	200.000	1000.00	5.712	34.312	287.802	67.249	4.149	0.040	3.475	0.616	0.002897	0.600	34.928
No. 3+ 150.000	200.000	1000.00	5.962	34.962	301.949	79.656	3.678	0.040	3.312	0.560	0.003091	0.599	35.521
No. 3+ 350.000	200.000	1000.00	5.765	35.765	430.362	145.308	2.950	0.040	2.324	0.275	0.002042	0.513	36.041

* Critical Depth

d) Composite Comparison

The following matters could be mentioned by the above approximate comparative study.

- Both the sites have almost the same catchment area and flood discharge.
- Both the sites have straight and stable river line.
- River bed elevation of downstream plan is higher than upstream site because of present river condition.
- The size of the weir in the upstream site is smaller by 1.0 m of the weir width and higher 1.55 m of the weir height, also the upstream site needs the additional canal of 0.80 km in length with deep excavation. Therefore, the upstream plan is not economical.
- As there is no influence of the back water to the upstream villages, the type of the weir is proposed more economical fixed weir type for both the plans than movable weir type.
- The upstream site has more difficult hydraulic conditions on the flood discharge because the expansion rate of natural river width is more than 1.2 times against the basis of the Irrigation Design Standard.
- Both the sites have almost the same geological condition and almost no problem as the foundation of the weir.
- The temporary diversion channel is proposed for the construction method of both the plans.

Since, the site of the weir for the Project is more favorable on the downstream plan than the upstream one by the all-round study.

4.1.3 Design & hydraulic calculation on size of facility

(1) Dimension of the structure

a) Dimension of the structure

Water source	: Selagan river
Location of intake facility	: about 2.3 Km upstream from Kp. Lubuk Sahung
Catchment area	: 375 Km^2
Elevation of river bed	: EL 22.20 m
Elevation of crest	: EL 26.00 m
Height of weir	: 3.80 m
Height of weirbody	: 6.30 m
Width of weir	: 74.00 m
Intake water level	: NWL. 25.90 m
Flood discharge	: HWL. 30.05 m

	(1 in 100 year probability)
Flood discharge	: HWL. 30.85 m (1 in 1000 year probability)
Elevation of river bank	: EL. 31.55 m
Freeboard	: 1.50 m (1/100 y. prob.)
Freeboard	: 0.70 m (1/1000 y. prob.)
Type of weir	: Fixed type
Flood way	: Fixed weir (Length of span 68.0 m)
Scouring sluice	: Under sluice (2 m x 2 gates x 2 stairs)
Intake	: Sluice type gate (2.90 x 2.05 m x 3 gates)
Design intake discharge	: 6.45 m^3/s
Scale of fishway	: Step type, width 2.00 m Length 21.24 m
Small-scale Hydro-power Generation	: 290 Kw, available head 3.50 m
Construction method	: Temporary diversion

b) Hydrologic condition (From the hydrologic data)

1 in 5 year flood discharge probability Q1/15 = 660 m^3/s

1 in 25 year flood discharge probability Q1/25 = 840 m^3/s

1 in 50 year flood discharge probability Q1/50 = 910 m^3/s

1 in 100 year flood discharge probability Q1/100 = 1,000 m^3/s

1 in 1000 year flood discharge probability Q1/1000 = 1,300 m^3/s

(2) Study of weir width

Total width of weir is decided by unit flood quantity which is the standard, q = 12.0 ~ 14.0 $m^3/s/m$

B' = Q1/100/q = 1,000 m^3/s/14.0 $m^3/s/m$ = 71.43 m

Design width of weir crest B;

B = 71.43 + (width of pier of scouring sluice 1.00 m x 2 piers)
= 71.43 + (1.00 x 2) = 73.43 = 74.00 m

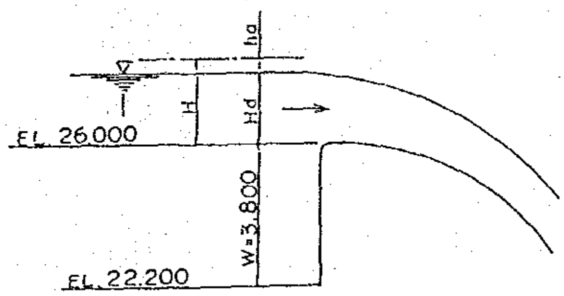
Width of flood way crest = 74.00 - 6.00 = 68.00 m

(3) Hydraulic calculation at the time of flood

a) Calculation of overflow depth

$Q = Cd \times Be \times H^{2/3}$

Here Q : Quantity of overflow m^3/s
Be : Width of crest m
H : Overflow head
(Overflow depth, Hd + Volocity head, ha)
Cd : Coefficient of discharge
$= 2.200 - 0.0416\ (H/W)^{0.990}$
W : Height of weir = 3.80 m



o Calculation of effective width of overflow (Be)

$Be = B - 2\ (n \cdot Kp + Ka) \times H$
$= 74.00 - 2 \times (2 \times 0.01 + 0.0) \times H$
$= 74.00 - 0.04\ H$

Here Bn: Total width of overflow (m)
Kp: Coefficient by pier (Circle = 0.01)
n : Number of piers (3 Nos.)
Ka: Coefficient by side wall (0.00)
H : Overflow head (m)

Table V-44 OVERFLOW DEPTH AND DISCHARGE FOR WEIR

H (m)	WL (m)	Be (m)	Cd	Q (m^3/s)	
0.200	26.200	73.99	2.198	0.7	
0.400	26.400	73.98	2.196	5.2	
0.600	26.600	73.98	2.193	17.5	
0.800	26.800	73.97	2.191	41.5	
1.000	27.000	73.96	2.189	80.9	
1.200	27.200	73.95	2.187	139.7	
1.400	27.400	73.94	2.185	221.6	
1.600	27.600	73.94	2.182	330.5	
1.800	27.800	73.93	2.180	470.0	
2.000	28.000	73.92	2.178	644.0	
2.200	28.200	73.91	2.176	856.2	
2.300	28.300	73.91	2.175	977.8	
2.350	28.350	73.91	2.174	1042.7	*
2.400	28.400	73.90	2.174	1110.3	
2.500	28.500	73.90	2.173	1254.3	
2.550	28.550	73.90	2.172	1330.7	**
2.600	28.600	73.90	2.171	1410.1	
2.800	28.800	73.89	2.169	1759.3	
3.200	29.200	73.87	2.165	2620.2	
3.400	29.400	73.86	2.163	3139.4	
3.600	29.600	73.86	2.161	3722.5	
3.800	29.800	73.85	2.158	4373.1	
3.850	29.850	73.85	2.158	4546.8	
4.000	30.000	73.84	2.156	5094.9	

Note: * is 1/100 flood case.
** is 1/1,000 flood case.

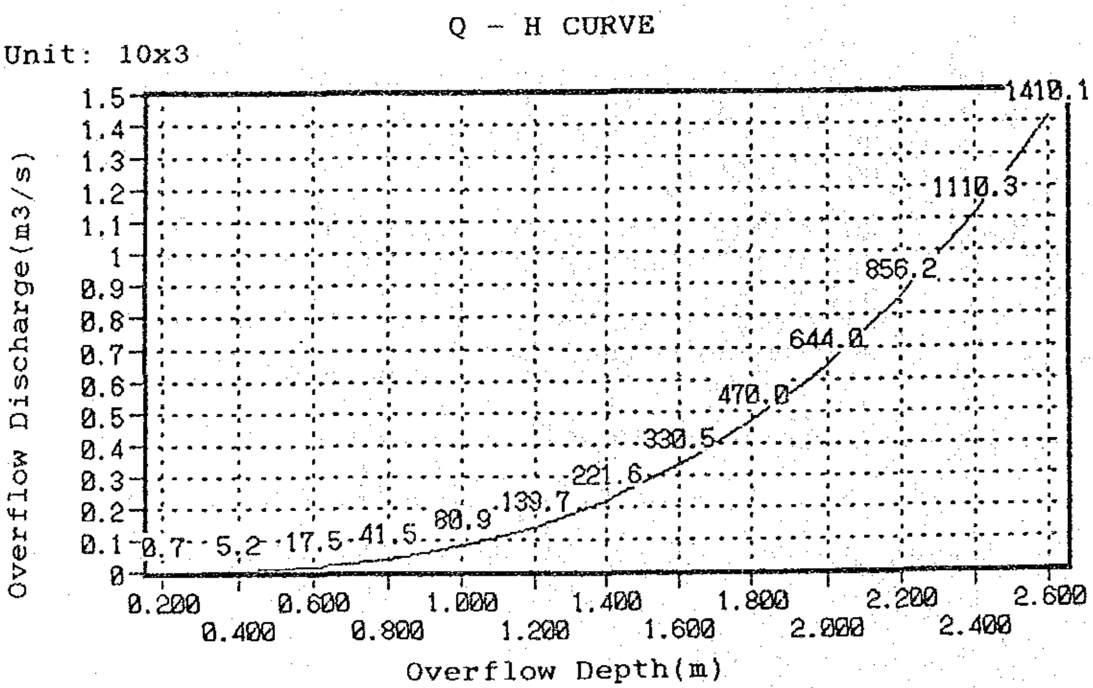
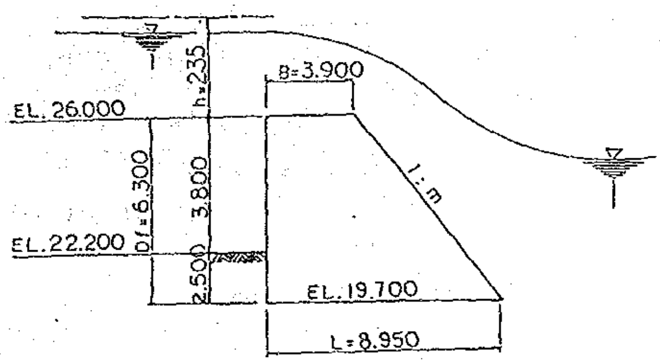


Fig. V-14 Q-H CURVE AT WEIR SITE

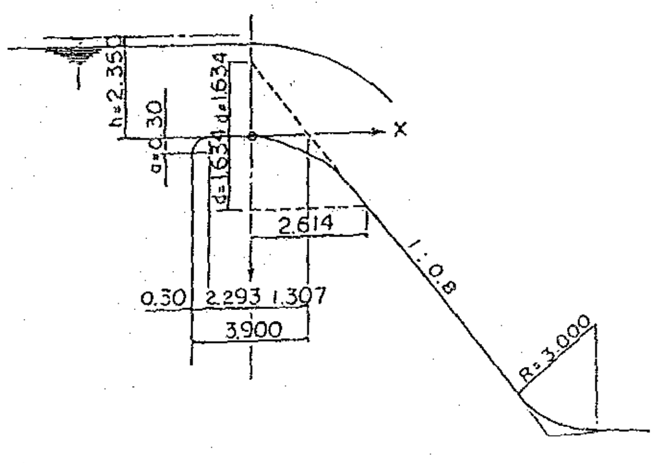
b) Basic cross section of weir

o Assumption of the cross section

When m = 0.8 is applied safe and economic section modulus (α) is considered as α = 0.62
B = α x Df = 0.62 x 6.30 = 3.900 m
L = (α + m) Df = (0.62 + 0.8) x 6.30 = 8.95m



o Modification of the trapezoid section.



There are several modified sections for the modification of trapizoid section which is the basic section of weir. However it is always required to apply a curve formula considered that the vein of overflow must fit to the body, satisfy the hydraulic conditions, and be easy for the construction works.

$$X2 = 4 \cdot m^2 \cdot d \cdot Y \qquad d \geqq 1.78h/4m^2$$

$$d = 1.78 \times 2.35/4 \times 0.80^2$$

$$= 1.634$$

$$= 4 \times m^2 \times d \times Y = 4 \times 0.80^2 \times 1.634 \times Y$$

$$= 4.183 \cdot Y$$

Y	0.0	0.05	0.10	0.20	0.40	0.80	1.60	2.614
X	0.0	0.457	0.647	0.915	1.294	1.829	2.587	2.614

o Upstream top side of weir is a quarter circle
$a = 0.125h = 0.12 \times 2.35 = 0.30m$

o Bucket curve is set at the water cushion to change the direction of the falling vein into the horizontal one.
$R = Df \times (1/2 \sim 1/3)$
$= 6.30 \times (1/2 \sim 1.3) = 3.15 \sim 2.10 = 3.00m$

$$\tan\alpha = 1/0.8 = 1.250$$

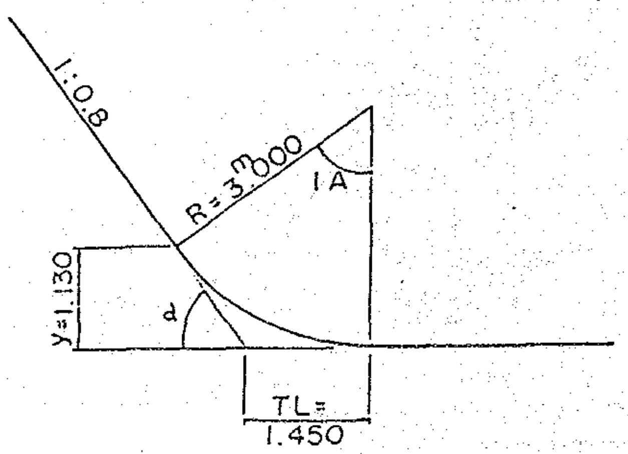
$$= IA = 51^\circ 20' 25''$$

$$TL = R \tan IA/2$$

$$= 1.442 = 1.450m$$

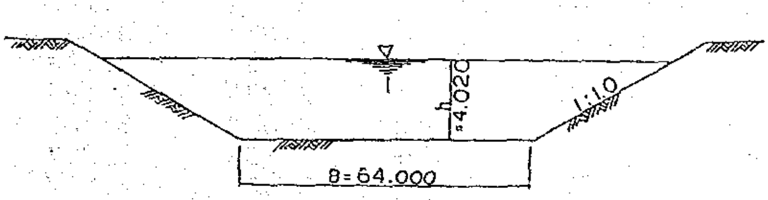
$$y = \sin\alpha \cdot TL$$

$$= 1.126 = 1.130m$$



c) Calculation of canal sections by Coupure method

Quantity	$Q = 1{,}000 m^3/s$
Longitudinal slope of canal	$I = 1/500$
Slope	$Z = 1:1.0$
Width of canal	$B = 64.0$
Coefficient of roughness	$n = 1/30 = 0.033$



$A = Bh + Zh2$ $\qquad$ $P = B + 2h \quad 1+Z2$

$R = A/P$ $\qquad$ $V = 1/n \cdot R2/3 \cdot \quad 1$

$Q = A \times V \quad (m^3/s)$ $\qquad$ $= 30 \times R2/3 \times \; 0.0020$

$= 1.3416 \cdot R2/3$

Table V-45 HYDRAULIC CALCULATION OF DOWNSTREAM SECTION

h (m)	A (m2)	P (m)	R (m)	R 2/3	V (m/s)	Q (m3/s)	
0.50	37.250	75.414	0.494	0.625	0.838	31.2	
1.00	75.000	76.828	0.976	0.984	1.310	99.0	
1.50	113.250	78.243	1.447	1.280	1.716	194.3	
2.00	152.000	79.657	1.908	1.538	2.063	313.6	
2.50	191.250	81.071	2.359	1.772	2.376	454.5	
3.00	231.000	82.485	2.801	1.987	2.664	615.5	
3.50	271.250	83.899	3.233	2.186	2.932	795.3	
4.00	312.000	85.314	3.657	2.374	3.183	993.1	
4.01	312.820	85.342	3.665	2.377	3.188	997.3	
4.02	313.640	85.370	3.674	2.381	3.193	1001.4	*
4.03	314.461	85.398	3.682	2.385	3.198	1005.6	
4.04	315.282	85.427	3.691	2.388	3.203	1009.7	
4.05	316.103	85.455	3.699	2.392	3.207	1013.9	
4.09	319.388	85.568	3.733	2.406	3.227	1030.6	
4.10	320.210	85.596	3.741	2.410	3.232	1034.8	
4.50	353.250	86.728	4.073	2.550	3.420	2308.2	
5.00	395.000	88.142	4.481	2.718	3.645	1439.8	

Note: * means the case of 1/100 flood discharge.

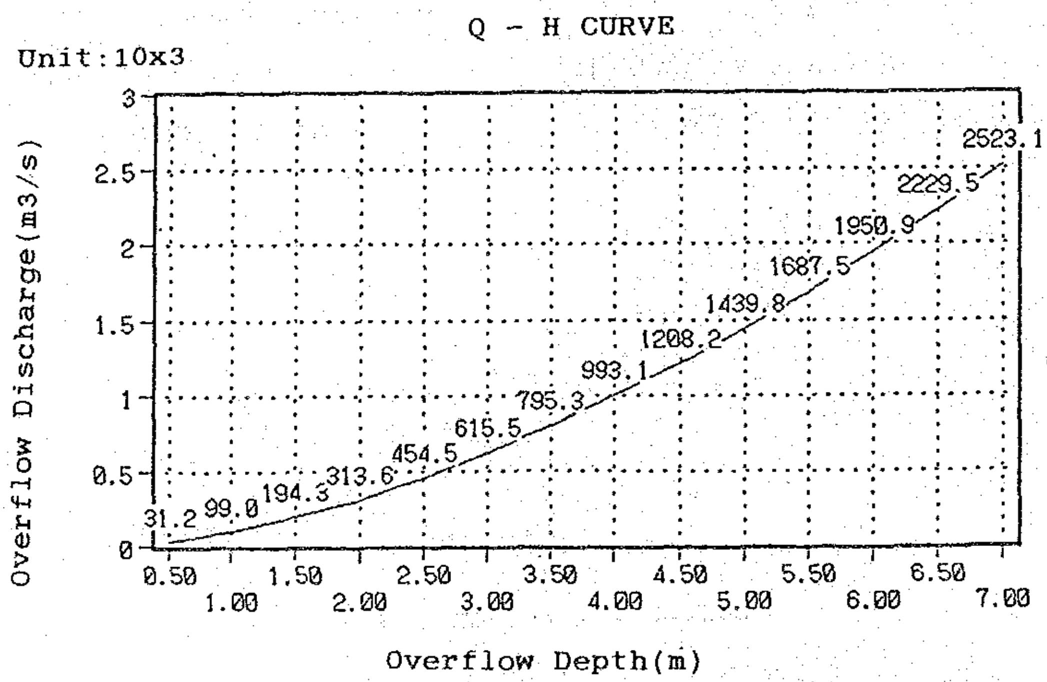
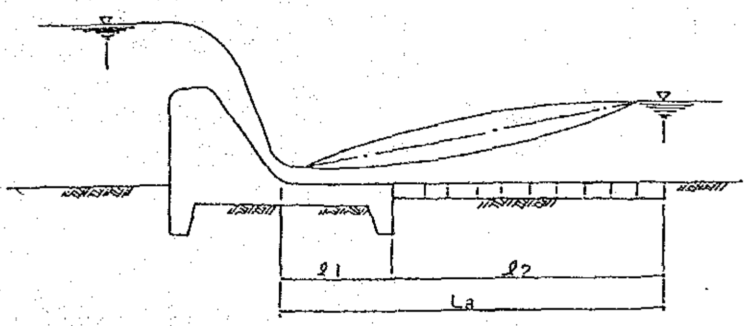


Fig. V-15 Q-H CURVE AT DOWNSTREAM SECTION

(4) Study of energy dissipator

a) Study of fore apron and the protection works of river bed

The lengths of fore apron and downstream protection works of river bed are calculated against scouring the downstream bed by overflow water.



o Length of fore apron

$L1 = 0.6 \cdot C\sqrt{D1}$

Where C: Bligh's coefficient (Coarse sand 12)

D1: Height between the crest and apron (3.80m)

$= 0.6 \times 13 \sqrt{3.80} = 14.04 = 14.50m$

o Length of the protection works of river bed

$LB = 0.67 \cdot C\sqrt{Hd \cdot q}$

Here C: Bligh's coefficient (Coarse sand 12)

Hd: Difference of water elevation between flood stage & draughty water level (D_1 = Hd = 3.80m)

q: Unit quantity of flood discharge $1{,}000m^3/s/74m = 13{,}514m^3/s/m$

$= 0.67 \times 13 \sqrt{3.80 \times 12.8}$

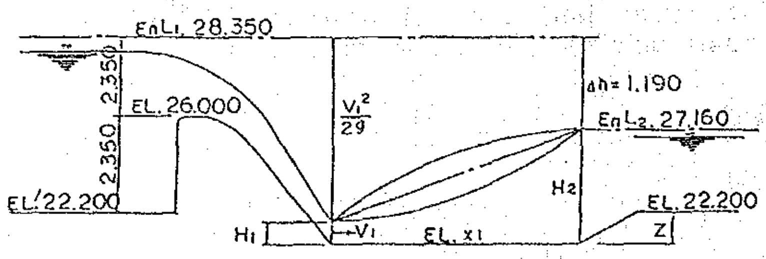
$= 57.62 = 58.00m$

$L2 = LB - L1 = 58.00 - 14.50 = 43.50m$

As a result of the above calculation, the structure of downstream side of weir is decided as the type of energy dissipator.

- The river bed protection works is uneconomic by increasing the length of the protection works because the back water height is high.
- Most of the results of Indonesian construction are also in the type of energy dissipator.

b) Hydraulic calculation of energy dissipator



Height of hydraulic jump $H_2/H_1 = 1/2(\sqrt{1+8F2} - 1)$

$V = q/H1$

$H_2 = H_1(\sqrt{1+8F2} - 1) \times 1/2$

$q = 13.514\ m^3/s$

Froude number $F1 = V1/\sqrt{g \cdot H1}$

$EL \times 1 = 28.35 - (H1 + V1^2/2g)$

$EL \times 2 = 26.74 - H2$

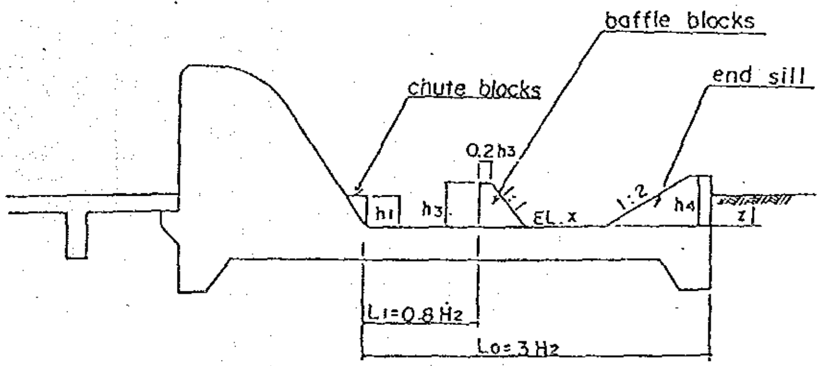
Table V-46 HYDRAULIC CALCULATION OF ENERGY DISSIPATOR

H1 (m)	V1 (m/s)	V1^2/2g (m)	ELX1 (m)	F	H2 (m)	ELX2 (m)	ELX1 -ELX2
2.50	5.41	1.49	24.36	1.09	2.81	23.93	0.43
2.00	6.76	2.33	24.02	1.53	3.43	23.21	0.71
1.50	9.01	4.14	22.71	2.35	4.29	22.45	0.26
1.49	9.07	4.20	22.66	2.37	4.31	22.43	0.23
1.48	9.13	4.25	22.62	2.40	4.33	22.41	0.21
1.47	9.19	4.31	22.57	2.42	4.35	22.39	0.18
1.46	9.26	4.37	22.52	2.45	4.37	22.37	0.15
1.45	9.32	4.43	22.47	2.47	4.40	22.34	0.12
1.44	9.38	4.49	22.42	2.50	4.42	22.32	0.09
1.43	9.45	4.56	22.36	2.52	4.44	22.30	0.06
1.42	9.52	4.62	22.31	2.55	4.46	22.28	0.03
1.41	9.58	4.69	22.25	2.58	4.48	22.26	0.00
1.40	9.65	4.75	22.20	2.61	4.51	22.23	-0.04
1.30	10.40	5.51	21.54	2.91	4.74	22.00	-0.46
1.20	11.26	6.47	20.68	3.28	5.01	21.73	-1.06

c) Type of energy dissipator

As a result of the above hydraugh calculation, height of the jump (H_2) is H_2 = 4.48mt with a condition of the vein of inflow H_1 = 1.410 m, F = 2.58, V_1 = 9.580 m^3/s and it can be connected smoothly with the downstream water surface.

As a type of energy dissipator, the forced jump USBR type III can be applied based on the condition of unit quantity of flow (less than 18.5 $m^3/s/m$), Velocity of inflow (less than 18.0 m/s), Froude number of inflow vein (Less than 4.5).



Length of energy dissipator

LO = 3 · H2 = 3 x 4.48 = 13.44 = 13.50m

Location of baffle pier

L1 = 0.8 · H2 = 0.8 x 4.48 = 3.584 = 3.60m

Height of chute block

	Height	h 1=H 1=1.41 = 1.40m
	Width	W1 = H1=1.40m
	Distance	S1 = H1=1.40m (edge 0.6m)
Baffle pier	Height	h3/H1 = 2, H3 = 2 x 1.41 m = 2.80 m
	Width	W3 = 0.75 · H3 = 0.75 x 2.80 = 2.10 m
	Distance	S3 = 0.75 · h3 = 0.75 x 2.80 = 2.10 m
	Crest width of weir	= 0.20 · h3 = 0.20 x 2.80 = 0.55m
End sill	h4/H1 = 1.5	h4 = 1.5 x 1.41 = 2.10m

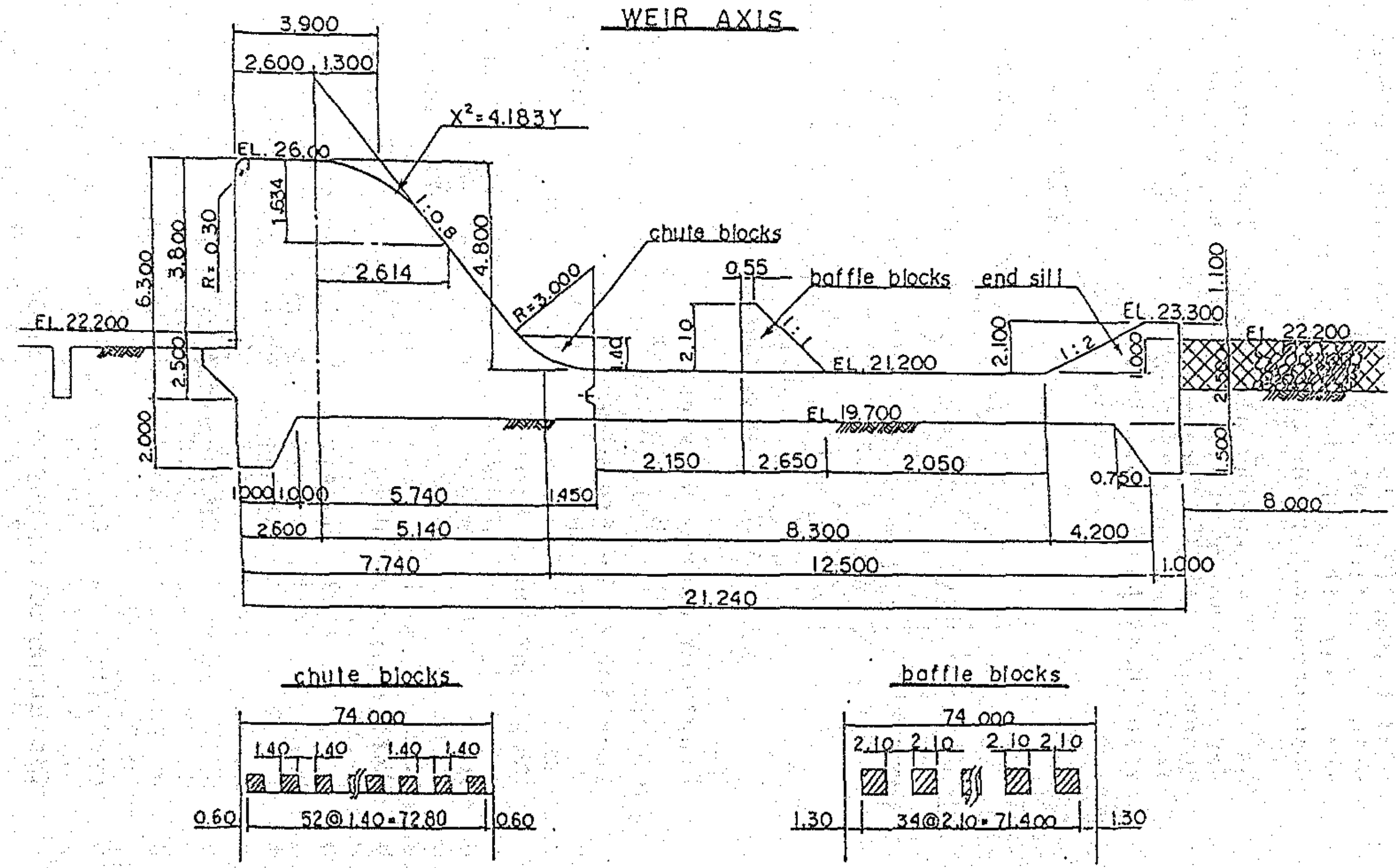
Elevation of energy dissipator

Z = EL22.20-ELX1 22.25 = -0.05

Giving a surplus : Z = 1.0m

ELx = EL22.20 - 1.00 = EL21.200m

Fig. V-16 PROFILE OF WEIR BODY



(5) Study of creep length

o Bligh's method

$L \geqq V \cdot \Delta h$

Here C: Bligh's coefficient (Coarse sand 12)

h: Maximum head between the upstream and the downstream (3.80m)

$\Delta h \cdot C = 12 \times 3.80 = 45.60m$

Actual length of weir body (See the above figure)

$L = 4.5 + 21.24 + 4.0 + (2.0 + 1.5) \times 1.118$
$= 33.653m$

* 45.60m ≧ 33.65m ..... No (Short length = 11.95m)

o Lane's method

$L' \geqq C' \cdot \Delta h$

Here C': Lene's creep ratio (Coarse sand 6)

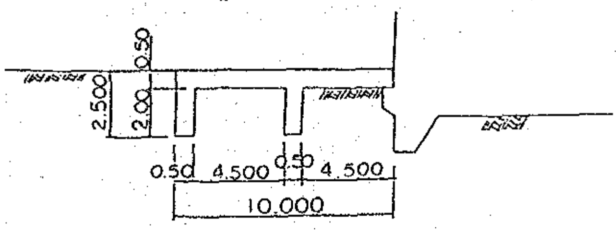
$C' \Delta h = 6 \times 3.80 = 22.80m$

Actual length of weir body (See the above figure)

$L' = (4.5+2.0+1.5+4.0) + (21.24 \times 1/3) = 19.08m$

22.80m ≧ 19.08m ..... No (Short length = 3.72m)

According to the above calculation, it is found that the creep length is not enough against the length of weir body. Generally, it is secured by water stop board, fore apron etc. but geologically it is very hard to apply water stop board because of construction difficulty. Thus, rear apron is provided to prevent piping by securing creep length as there were many construction examples in Indonesia, too.



When rear apron is provided like the above figure, creep length can be as follows.

o Bligh's method

$\Sigma L = 33.65 + (10.00+2.00 \times 3+2.50) = 52.15m$

$* \Sigma L \geqq C \cdot \Delta h = 52.15 \geqq 45.60m$ ..... OK

o Lane's method

$\Sigma L' = 19.08 + (2.50+2.00 \times 3+10.00 \times 1/3) = 30.91m$

$* \Sigma L \geqq C \cdot \Delta h = 30.01 \geqq 22.80m$ ..... OK

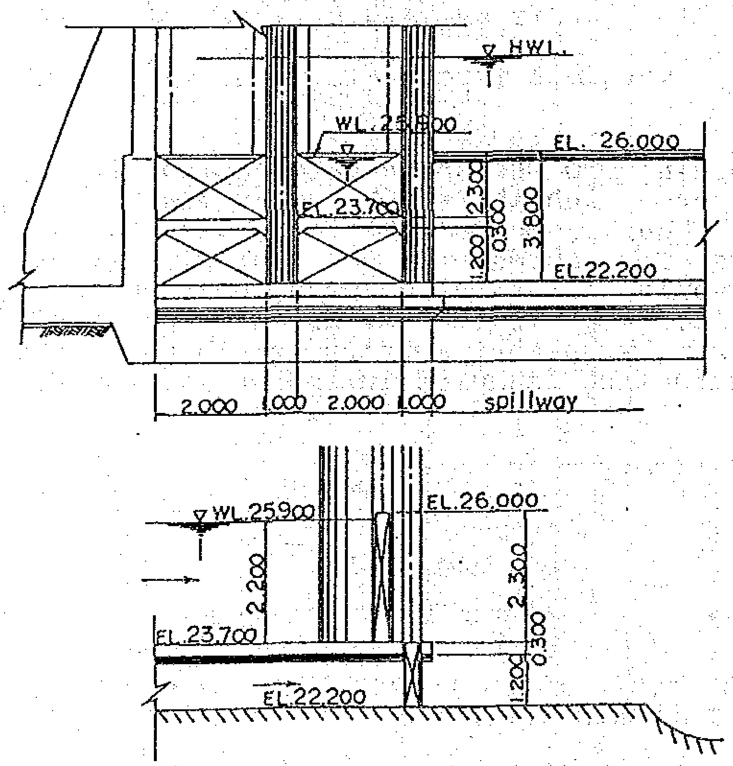
Thus, creep length can be secured by rear apron.

(6) Scouring sluice

Under sluice type is selected because it has many construction example of the same type for scouring sluice in Indonesia Numbers and each length of the spans are decided referring to similar scale of Indonesian ones.

Number of span N = 2 gates
B = 2.00m (Scale of gate should be possible to be controlled by hand.)

Width of scouring sluice
(Width of the inflow mouth x about 0.6)
= 6.60 x 0.6 = 3.96
= 4.00m



(7) Study of intake

o Maximum regulated intake quantity

$Q = 6.61\ m^3/s$

o Design velocity of standard intake flow

$V = 10 \cdot d^{0.5}$

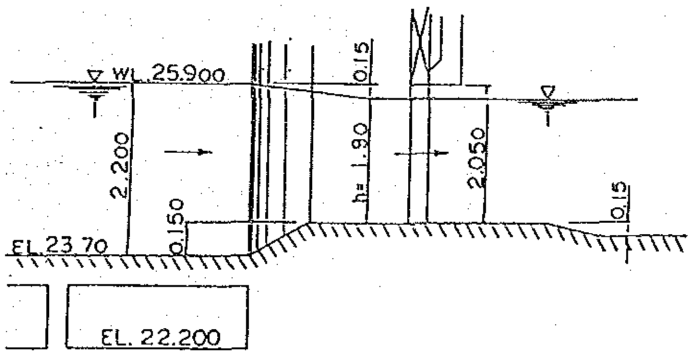
Where d = Grain size of river bed material according to the study of the grain size of river bed material at the proposed point of weir in the present condition, average grain size is around 3.5mm by the sieving study of 50% grain size.
In this Design, intake velocity is applied to stop grain size of 3.5mm.

$* \ V = 10 \times 0.0035^{0.5}$
$= 0.592 = 0.60\ m/s$

o Design intake depth

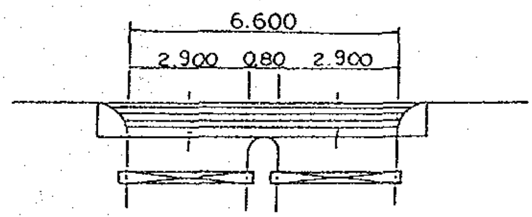
When the intake loss head is 0.15m and intake sill is about 0.15m, water depth of immediate downstream of intake is h = 1.900m

$h = 2.200 - (0.15 + 0.15) = 1.900\ m$



o Design width of inflow

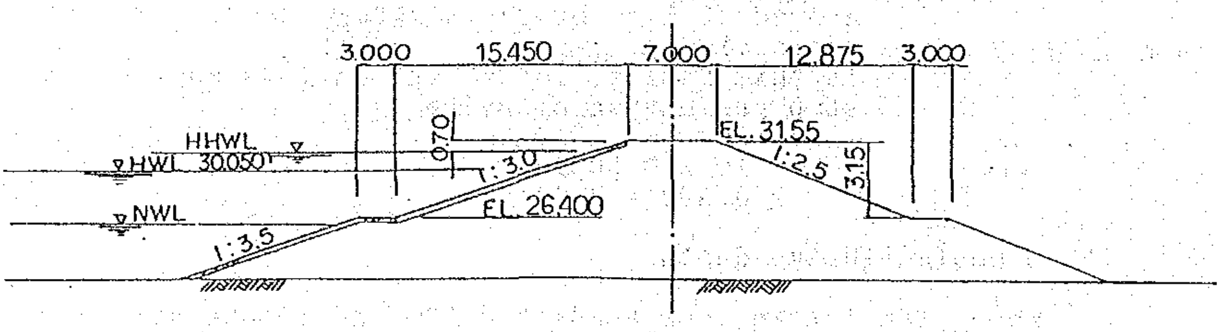
Design width of inflow = $6.61 m^3/s/1.90m \times 0.60m/s$
= 5.798 = 5.80m
Width of each gate = 5.80m/2 gates = 2.900m



(8) Study of sub-dike

Section of sub-weir is assumed as follows and upstream slope is protected by stone.

Fig. V-17 PROFILE OF SUB-DIKE



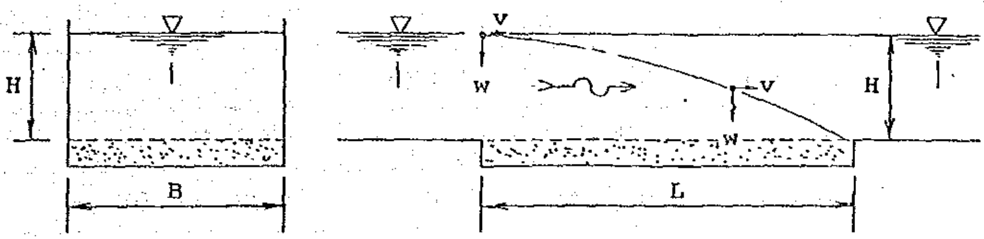
(9) Study of Sand Trap

a) Relation between Velocity and Grain Diameter

$$Vd=10xd^{0.5}$$

where Vd : Average Velocity (m/s)
d : Grain Diameter (m)

b) Dimension of Sand Trap (Length and Width)



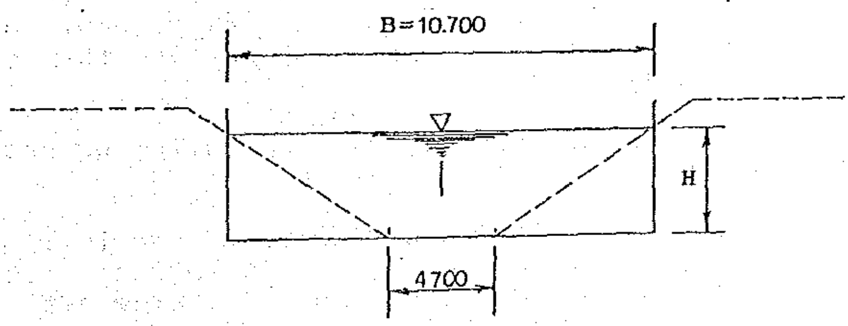
Therefore : $H/W = L/V$ with $V = Q/H \cdot B$

$$L = (H \cdot V/W)\ F$$

where H : Depth of Canal Flow (m)
W : Falling Velocity of Sediment Particle (m/s)
L : Length of Sediment Trap (m)
V : Flowing Velocity of Water (m/s)
Q : Canal Discharge (m/s)
B : Width of Sediment Trap (m)
F : Safety Rate (1.5 - 2.0)

$Q_{max}=6.61\ m^3/s$ B=10.70m
$V=Q/H \cdot B=0.281$ m/s

H=2.20 (Considering flush canal)



c) Relation between Length of Sediment Trap and Grain Diameter under the condition of Maximum Canal Discharge

d (m)	Vd(m/s)	W(m/s)	H (m)	L (m)	
0.0035	0.600	0.280	2.20	5.00	(*1)
0.0009	0.300	0.095	2.20	14.00	(*2)
0.0003	0.173	0.030	2.20	42.00	(*3)
0.00007	0.084	0.004	2.20	309.00	(*4)

From the above table, the following matters can be pointed out:

1) The maximum grain size flowing from the Intake is 3.5mm.(*1)

2) The maximum grain size of bed load following under the condition of minimum water velocity is 0.9mm.(*2)

3) In the case of the application of grain material (0.3mm) produced in Japan, the required length of Sediment Trap gets 42.0m.(*3)

4) In the case of the application of grain material (0.07mm) produced in Indonesia, the required length of Sediment Trap gets 309.0m.(*4)

5) The grain sizes less than 0.065mm of bed load in the vicinity of the Head Works were resulted within the range between 0.47 percent and 0.81 percent of bed load as shown in the following table and chart.

Judging from the above study, the following points are concluded:

1) The actual length of Sediment Trap is limited by the conditions such as geological condition, necessity of drainage canal for blow off.

2) The function as a sand trap shall be employed instead of the one as a sediment trap.

3) The available grain size to be applied shall be within the range of 0.3mm to 3.5mm.

4) Taking into account the above point, the length of Sand Trap shall be 42.0 ÷ 45.00m.

Source : Final Report/Penelitian Kualitas Air dan Sediment Transport Air Dikit, Air Selagan dan Air Manjuto, April 1984/DPMA.

Table V-47 GRAIN SIZE ANALYSIS OF SELAGAN RIVER BED LOAD

River home : Air Selagan
Site : Desa Tras Terunjam
Date : Nov. 9, 1983

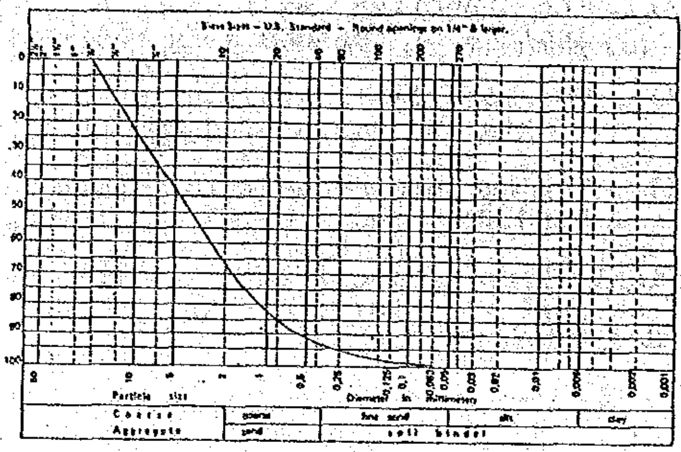
No. of Sample	Unit	Percentage of Grain Size								
	mm	20.00	10.00	5.00	2.00	1.00	0.50	0.25	0.125	0.063
1	%	100	68.33	53.29	33.94	26.20	21.20	12.41	2.63	0.81
2	%	100	86.74	72.08	45.62	31.30	22.84	12.94	3.35	0.65
3	%	100	93.89	73.50	32.32	21.56	13.58	5.84	1.49	0.47

Source : Final Report/Penelitian Kualitas Air dan Sediment Transport Air Dikit, Air Selagan dan Air Manjuto, April 1984/DPMA

DIREKTORAT PENYELIDIKAN MASALAH AIR
LABORATORIUM UNIT PENELITIAN SEDIMEN
Jalan Ir. H. Juanda 193 – Tlp. 84553 . 84554 . 81807

River : Air Selagan
Station Lokasion : Tras Trunjam
Date : 9-11-1983
Sample NO : 1.

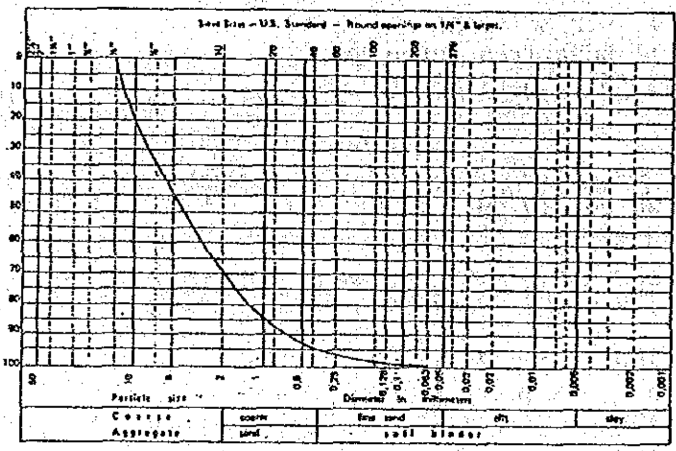
MECHANICAL ANALYSIS
GRAIN SIZE ACCUMULATION CURVE



DIREKTORAT PENYELIDIKAN MASALAH AIR
LABORATORIUM UNIT PENELITIAN SEDIMEN
Jalan Ir. H. Juanda 193 – Tlp. 84553 . 84554 . 81807

River : Air Selagan
Station Lokasion : Tras Trunjam
Date : 9-11-1983
Sample NO : 2.

MECHANICAL ANALYSIS
GRAIN SIZE ACCUMULATION CURVE



DIREKTORAT PENYELIDIKAN MASALAH AIR
LABORATORIUM UNIT PENELITIAN SEDIMEN
Jalan Ir. H. Juanda 193 – Tlp. 84553 . 84554 . 81807

River : Air Selagan
Station Lokasion : Tras Terunjam
Date : 9-11-1983
Sample NO : 3.

MECHANICAL ANALYSIS
GRAIN SIZE ACCUMULATION CURVE

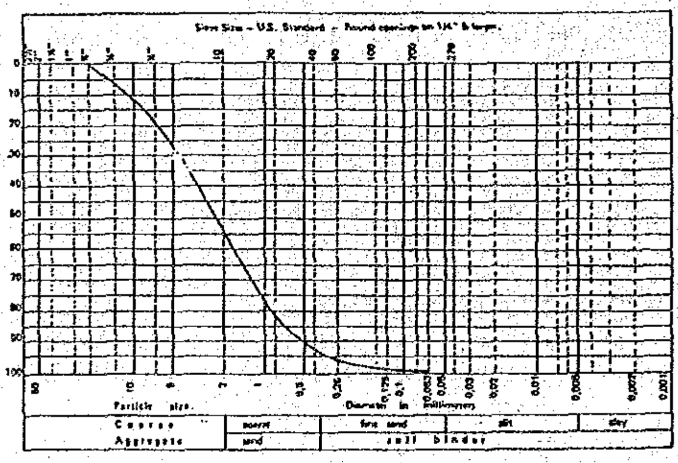


Fig. V-18 GRAIN SIZE ANALYSIS AT TERAS TERUNJAM

4.2 Irrigation System

(1) Water Source

Irrigation water is required for the study area all the year round and is supplied from the weir on the Air Selagan where location was decided during the study.

According to low water discharge analysis, mean annual discharge is 39.6 m^3/sec, and minimum monthly discharge with 5 year probability of non-exceedance is 12.9 m^3/s. Annual discharge of the Air Selagan is 875x10^6 with 5 year probability of non-exceedance. Irrigation water to be supplied is 44.3x$10^6 m^3$ during rainy season, being 51.5x$10^6 m^3$ during dry season respectively. As to the domestic water, annual water supply is planned to be 0.6x$10^6 m^3$ in maximum. Namely, 11.0% of annual discharge is utilized for irrigation and domestic supply.

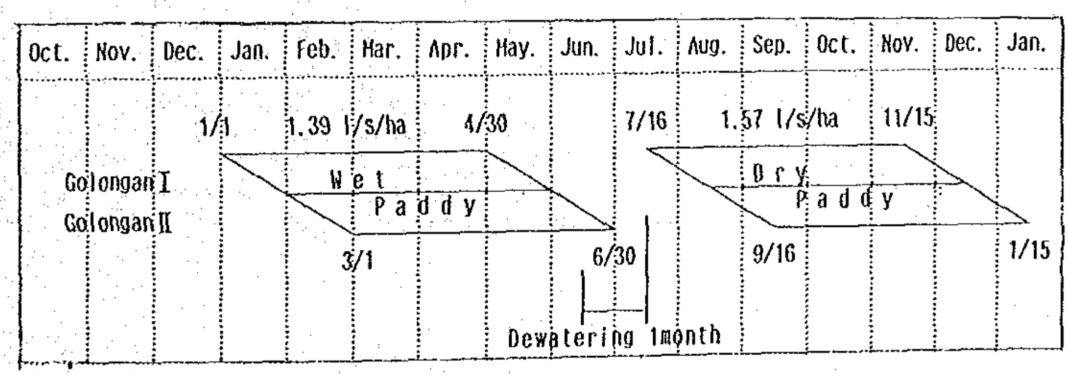
The maximum and minimum intake discharge are as follows:

	Maximum	Minimum
Wet season paddy	5.84 m^3/s	0.88 m^3/s
Dry season paddy	6.59	0.29
Domestic water supply	0.02	0.02

(2) Distribution Method of Irrigation Water

Golongan system and plot to plot irrigation will be adopted for the project area.

As to the wet paddy also dry paddy, the whole area of 4,200 ha will be divided into two Golongan blocks. The area of one Golongan block will become about 2,100 ha. For the sake of canal capacity, however, the Golongan system will be adopted about each secondary canal during wet and dry seasons paddy cultivation. Conception of Golongan system is shown as below.



Plot to plot irrigation method will be taken at steep slope fields at every several plots. In case of flat area, separated canals for irrigation and drainage will be equipped in order to make a plain farming practice.

(3) Cropping Period and Irrigation Area

The dry season paddy cultivation is proposed to start two and half months after the harvest of the wet season paddy and the period to release water from canal for operation and maintenance is also proposed one month during dewatering period of the dry season paddy cultivation.

The following table shows the most applicable case on the basis of the study.

Season's crop	Commencement date of puddling	Irrigation area	Max.Diversion requirement
Wet paddy	Jan. 1	4,200 ha	1.36/s/ha
Dry paddy	Jul. 16	4,200	1.53

Taking into consideration resorting a weir without storage effect, fluctuation of average ten days discharge, the planning total household of transmigrants, distribution area for paddy cultivation per household, surplus water to downstream etc., the most appropriate cropping areas in the both seasons are obtained as the above table.

(4) Ten Day Intake Discharge

The ten days intake discharge for paddy cultivation of 4,200 ha in wet and dry seasons are estimated as below.

Table V-48 TEN DAY INTAKE DISCHARGE

Wet paddy, Jan. 1 A=4,200 ha
Dry paddy, Jul.16 A=4,200 ha

Period	Unit Water Req.	River Dis.	Irrigation Water	Domestic Water Supply	Intake Discharge	Surplus Discharge
	l/s/ha	m3/s	m3/s	m3/s	m3/s	m3/s
Jan.1	0.55	51.67	2.31	0.02	2.33	49.34
2	0.65	34.31	2.73	0.02	2.75	31.56
3	0.70	34.59	2.94	0.02	2.96	31.63
Feb.1	0.48	35.31	2.02	0.02	2.04	33.27
2	1.10	22.44	4.62	0.02	4.64	17.80
3	1.36	25.62	5.71	0.02	5.73	19.89
Mar.1	0.73	39.68	3.07	0.02	3.09	36.59
2	0.65	44.21	2.73	0.02	2.75	41.46
3	0.91	40.79	3.82	0.02	3.84	36.95
Apr.1	0.93	28.06	3.91	0.02	3.93	24.13
2	0.70	23.48	2.94	0.02	2.96	20.52
3	0.79	37.14	3.32	0.02	3.34	33.80
May 1	0.54	25.53	2.27	0.02	2.29	23.24
2	0.26	21.56	1.09	0.02	1.11	20.45
3	0.47	18.85	1.97	0.02	1.99	16.86
Jun.1	0.52	15.50	2.18	0.02	2.20	13.30
2	0.26	17.24	1.09	0.02	1.11	16.13
3	0.00	11.45	0.00	0.02	0.02	11.43
Jul.1	0.00	16.61	0.00	0.02	0.02	16.59
2	1.07	16.47	4.49	0.02	4.51	11.96
3	1.17	19.48	4.91	0.02	4.93	14.55
Aug.1	1.23	16.66	5.17	0.02	5.19	11.47
2	0.93	9.21	3.91	0.02	3.93	5.28
3	1.53	12.83	6.43	0.02	6.45	6.38
Sep.1	1.18	27.69	4.96	0.02	4.98	22.71
2	0.82	26.48	3.44	0.02	3.46	23.02
3	0.00	36.82	0.00	0.02	0.02	36.80
Oct.1	1.22	27.23	5.12	0.02	5.14	22.09
2	0.99	31.21	4.16	0.02	4.18	27.03
3	0.57	30.57	2.39	0.02	2.41	28.16
Nov.1	0.73	32.37	3.07	0.02	3.09	29.28
2	0.89	24.25	3.74	0.02	3.76	20.49
3	0.27	36.08	1.13	0.02	1.15	34.93
Dec.1	0.08	30.81	0.34	0.02	0.36	30.45
2	0.20	37.75	0.84	0.02	0.86	36.89
3	0.30	39.83	1.26	0.02	1.28	38.55

Note : The river dischage of 1/5 years probability is used.

(5) Diversion Requirement of Development Stage

During the development stage, the irrigation, efficiency will be planned as 0.50 because new reclaimed paddy fields will need more irrigation water.

Therefore the diversion discharge will increase during development stage for paddy fields. These increasement of diversion discharge will be conveyed using canal free board as much as possible.

The relation of the diversion discharge and canal capacity will be studied in next stage.

4.3 Basic Plan of Irrigation System

(1) Alignment and system of main canal

The objective area for the study lies on both the sides of the Selagan river, then both-sides intake method can be considered. Actually, however, the upstream part from Kp. Pondok Kopi has mountainous topography with steep land slope and under these topographical conditions, it is difficult to assure the economical cost, smooth construction, efficient and effective operation and maintenance, and so on for the canal system.

From the above view point of topographical condition, one-side intake method on the right of the river is accepted for the weir and the main canal is divided into two (2) about 6.0 km in the downstream of the intake, and then the main canal to the left side crosses the Selagan river by a syphon and conveys water to the left side area.

All the canals pass through the governmental land such as forests, transmigration area, etc., but there is a concession area for P.T. Tolan Tiga on the left side in the downstream part of the weir and it will be necessary to keep the land of about 300m width from the river bank as the land for the inspection road to the weir.

A part of the existing canal system of the Muko-Muko Irrigation Project, which is located in the objective area, is included in the canal system for the Air Selagan Project.

As a result of the leveling survey between the bench mark (SCN.7) for the Air Selagan Project and BB6 of the Muko-Muko irrigation canal, the difference of 0.50m between two (2) systems is found. It simply means that the elevation of the Muko-Muko irrigation system is higher by 0.50m on the map and the attention on this matter should be paid to the study on the canal system.

(2) Study on intake method

The following points could be mentioned on the comparison between the both-sides intake method and the one-side intake method.

a) The both-sides intake method can be adopted because the the river line at the proposed weir site is straight.

b) The alignment of the main canal in the upstream part, if planned, is obliged to run through the mountainous land with steep slope on both the sides of the river.

c) As to the canal length between the weir site and the proposed place of a syphon, the alignment on the right side is estimated at 4.6km, and that on the left side at 5.4km and is longer by about 800m.

d) The places for drainage culvert works crossing the main canal are counted at 25 places on the right side alignment and those at 17 places on the left side alignment.

e) The most upstream part of the irrigable area is located at the right side of the river.

f) The left side canal route has two (2) parts of the land with steep slope near the river bank and the construction and the operation and maintenance of the canal will be more difficult than those of the right side canal route.

g) The length of the syphon crossing the Selagan river is estimated at about 460m.

h) The place proposed for the syphon has a sand bank in the center of the river and the syphon can be constructed by the method of half closure of the river.

i) The construction costs for each intake method are roughly estimated as follows:

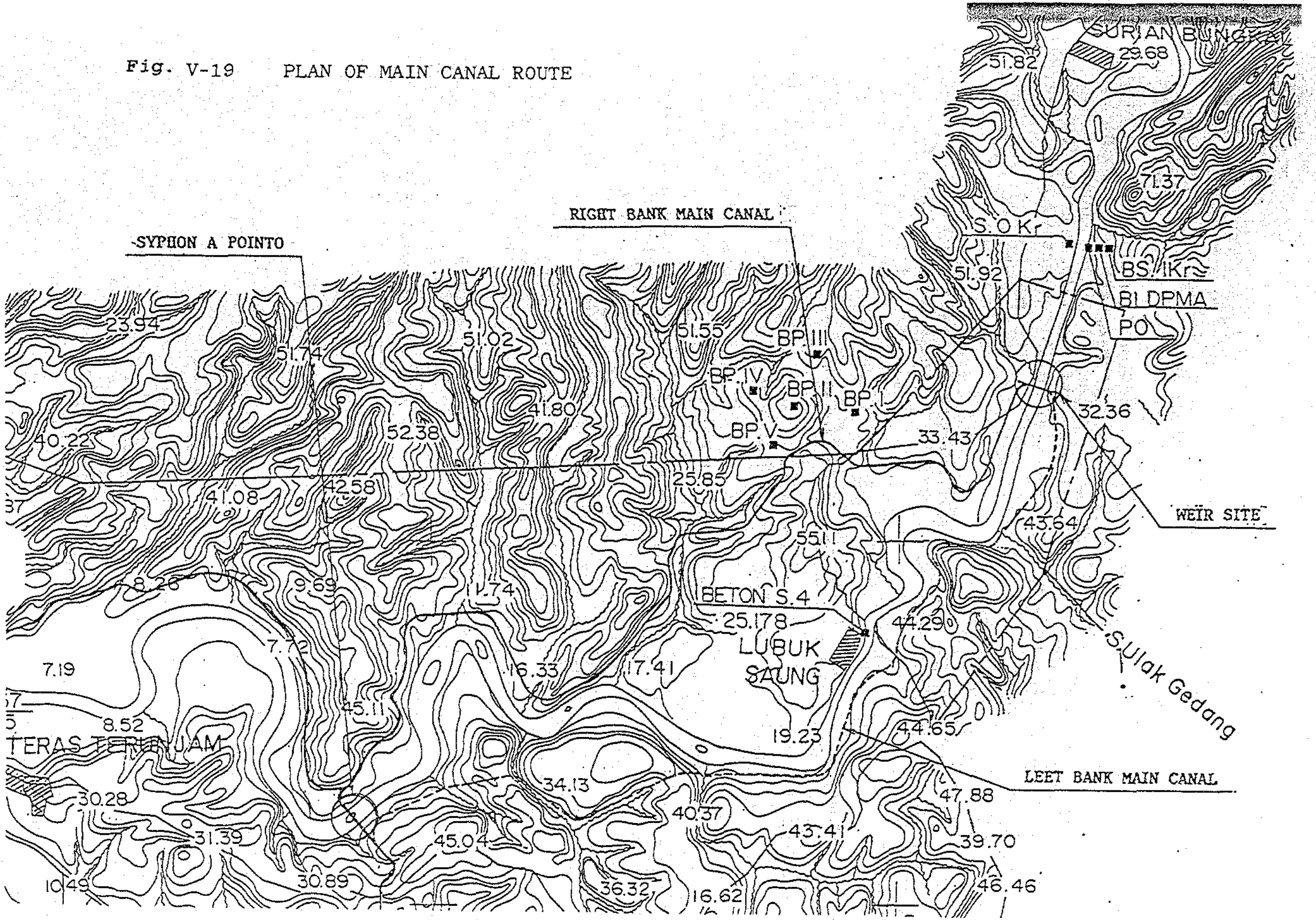
Unit : Million Rp.

Item	Unit	Both sides intakes			One side intake					
					Left			Right		
		Q'ty	Unit Price	Amount	Q'ty	Unit Price	Amount	Q'ty	Unit Price	Amount
Right main canal	km	4.6	300	1,380	-	-	-	4.6	325	1,495
Left main canal	km	5.4	300	1,620	5.4	350	1,890	-	-	-
Syphon	m	100	2.05	205	-	-	-	100	2.84	2.84
Syphon	m	-	-	-	460	2.52	1,159	460	2.52	1,159
Right side drainage culvert	nos	25	9.2	230	-	-	-	25	9.2	230
Left side drainage culvert	nos	17	9.2	156	17	9.2	156	-	-	-
Total				3,591			3,205			3,168
Ratio				113			101			100

The one-side intake method is more economical than the both-sides intake method and the right side canal route is the most appropriate.

From comprehensive view points, the one-side intake method and the right side main canal route are adopted taking into considerations the planning irrigation area, the topographic difficulty near the river bank on the left side, etc.

Fig. V-19 PLAN OF MAIN CANAL ROUTE



(3) Type of Canal

The earth canal with trapezoid section is generally adopted for the type of the irrigation canal from a economical point of view and with reference to the existing irrigation canal of the Muko-Muko Project and the soil condition mainly consisting of tuffaceous clay and volcanic ash clay in the objective area.

(4) Water Depth of Canal

As to the planning water depth, the following modified formula of Haring Huizen of PROSIDA in Indonesia is adopted.

$$h = 0.887 \times Q^{0.277}$$

where, h is water depth (m)
Q is planning discharge (m^3/sec)

The above coefficients were decided upon the Table A.2.1 to A.2.3 in the Irrigation Design Standard, KP-03.

(5) Maximum and Minimum Velocities

From the consistency test results and soil classification, the maximum velocity is taken as 0.7 m/sec as follows:

$$V\,max = Vb \times A \times B \times C$$
$$= 0.8 \times 1.1 \times 0.8 \times 1.0 = 0.70 \text{ m/s}$$

where V max is maximum allowable velocity in m/s
V b is basic velocity in m/s
A is correction factor for void ratio of canal surface
B is correction factor for water depth
C is correction factor for curvature

As to the minimum velocity, it is taken as 0.30 m/s.

(6) Side Slope

Minimum side slopes for various soils

Soil Material	Group Symbol	Side Slope Range 1 : m
Rock		< 0.25
Stiff peat	PT	1 - 2
Stiff clay, loam, loesses	CL,CH,MH	1 - 2
Sandy clay, cohesive	SC,SM	1.5 - 2.5
Sandy soil		
Silty sand	SM	2 - 3
Soft peat	PT	3 - 4

Minimum side slopes for canals in well compacted fill

Water depth + freeboard D (m)	Minimum side slope
$D \leq 1.0$	1 : 1
$1.0 < D < 2.0$	1 : 1.5
$D \leq 2.0$	1 : 2

(7) Free board

Minimum freeboard for unlined canals

Q in m^3/s	Freeboard in mm
< 0.5	0.40
0.5 - 1.5	0.50
1.5 - 5.0	0.60
5.0 - 10.0	0.75
10.0 - 15.0	0.85
> 15.0	1.00

(8) Coefficient of roughness

Design discharge in m^3/s	k
$Q > 10$	45
$5 < Q < 10$	42.5
$1 < Q < 5$	40
$Q \leq 1$ and tertiary service canal	35

(9) Ratio of width and water depth (b/h)

Discharge in m^3/s	Side slope 1:m	Ratio b/h n	Roughness factor k
0.15 - 0.30	1.0	1.0	35
0.30 - 0.50	1.0	1.2 - 1.2	35
0.50 - 0.75	1.0	1.2 - 1.3	35
0.75 - 1.00	1.0	1.3 - 1.5	35
1.00 - 1.50	1.0	1.5 - 1.8	40
1.50 - 3.00	1.5	1.8 - 2.3	40
3.00 - 4.50	1.5	2.3 - 2.7	40
4.5 - 5.00	1.5	2.7 - 2.9	40
5.00 - 6.00	1.5	2.9 - 3.1	42.5
6.00 - 7.50	1.5	3.1 - 3.5	42.5
7.50 - 9.00	1.5	3.5 - 3.7	42.5
9.00 - 10.00	1.5	3.7 - 3.9	42.5
10.00 - 11.00	2.0	3.9 - 4.2	45
11.00 - 15.00	2.0	4.2 - 4.9	45
15.00 - 25.00	2.0	4.9 - 6.5	45
25.00 - 40.00	2.0	6.5 - 9.0	45

(10) Width of inspection road (B) and opposite embankment (B')

Minimum embankment width

	B'	B
Design discharge in m^3/s	Without Inspection road in m	With Inspection road in m
$Q \leq 1$	2.00	3.50
$1 < Q \leq 5$	2.00	5.00
$5 < Q$	3.00	5.00

Table V-49.1 CANAL NAME , COVERING AREA & DESIGN CAPACITY

RIGHT BANK	CANAL NAME	DIVERSION STRUCTURE	COVERING AREA GROSS	NET	DIVERSION REQUIREMENT	DOMESTIC WATER	DESIGN CAPACITY
MAIN	LINK CANAL		ha	ha	m3/s	m3/s	m3/s
		BS0-BS1	4,700	4,200	6.43	0.02	6.45
		BS1-BS2	4,690	4,191	6.41	0.02	6.43
		BS2-BS3	4,633	4,140	6.33	0.02	6.35
		BS3-BS4	4,623	4,131	6.32	0.02	6.34
MAIN	RIGHT BANK MAIN CANAL						
		BS4-BR1	1,913	1,722	3.15	0.01	3.16
		BR1-BR2	1,874	1,687	3.09	0.01	3.10
		BR2-BR3	1,806	1,626	2.98	0.01	2.99
		BR3-BR4	1,764	1,588	2.91	0.01	2.92
		BR4-BR5	1,747	1,573	2.88	0.01	2.89
		BR5-BR6	1,712	1,541	2.82	0.01	2.83
		BR6-BR7	1,706	1,536	2.81	0.01	2.82
		BR7-BR8	1,680	1,513	2.77	0.01	2.78
		BR8-BR9	1,657	1,492	2.73	0.01	2.74
		BR9-BR10	1,634	1,471	2.69	0.01	2.70
		BR10-BR11	1,619	1,457	2.67	0.01	2.68
SEC.	S.S.PONDOK BATU						
		BR11-BP1	576	518	0.95	0.01	0.96
		BP1-BP2	531	477	0.87	0.01	0.88
		BP2-BP3	480	431	0.79	0.01	0.80
		BP3-BP4	470	424	0.78	0.01	0.79
		BP4-BP5	417	376	0.69	0.01	0.70
		BP5-BP6	353	318	0.58	0.01	0.59
		BP6-BP7	328	295	0.54	0.01	0.55
		BP7-BP8	273	245	0.45		0.45
		BP8-BP9	235	211	0.39		0.39
		BP9-BP10	179	160	0.29		0.29
		BP10-BP11	84	75	0.14		0.14
SEC.	S.S.HITAM						
		BR11-BH1	1,043	939	1.72	0.01	1.73
		BH1-BH2	169	152	0.28		0.28
		BH2-BH3	103	92	0.17		0.17
		BH3-BH4	50	44	0.08		0.08
SEC.	TRANSFER CANAL						
		BH1-BTR1	842	758	1.39		1.39
		BTR1-BTR2	837	753	1.38		1.38
		BTR2-BTR3	597	537	0.98		0.98
		BTR3-BTR4	525	473	0.86		0.86
		BTR4-BB4	456	410	0.75		0.75
SEC.	TRANSFER CANAL 2						
		BTR2-BB6	203	183	0.33		0.33

Table V-49.2 CANAL NAME , COVERING AREA & DESIGN CAPACITY

LEFT BANK	CANAL NAME	DIVERSION STRUCTURE	COVERING AREA GROSS	NET	DIVERSION REQUIREMENT	DOMESTIC WATER	DESIGN CAPACITY
			ha	ha	m3/s	m3/s	m3/s
MAIN	LEFT BANK MAIN CANAL						
		BS4-BL1	2,710	2,409	4.41	0.01	4.42
		BL1-BL2	2,700	2,400	4.39	0.01	4.40
		BL2-BL3	2,624	2,333	4.27	0.01	4.28
		BL3-BL4	2,568	2,287	4.19	0.01	4.20
		BL4-BL5	2,552	2,273	4.16	0.01	4.17
		BL5-BL6	2,540	2,262	4.14	0.01	4.15
		BL6-BL7	2,488	2,217	4.06	0.01	4.07
		BL7-BL8	2,482	2,212	4.05	0.01	4.06
		BL8-BL9	2,480	2,210	4.04	0.01	4.05
		BL9-BL10	1,796	1,601	2.93	0.01	2.94
		BL10-BL11	1,786	1,596	2.92	0.01	2.93
		BL11-BL12	1757	1,572	2.88	0.01	2.89
		BL12-BL13	1645	1,472	2.69	0.01	2.70
		BL13-BL14	1590	1,422	2.60	0.01	2.61
		BL14-BL15	1036	928	1.70	0.01	1.71
		BL15-BL16	991	887	1.62	0.01	1.63
SEC.	S.S.BL9Ka						
		BL9-BK1	684	609	1.11	0.01	1.12
		BK1-BK2	676	604	1.11	0.01	1.12
		BK2-BK3	521	465	0.85		0.85
		BK3-BK4	415	370	0.68		0.68
		BK4-BK5	287	255	0.47		0.47
		BK5-BK6	149	133	0.24		0.24
SEC.	S.S.BK2Ka						
		BK2-BBK1	155	139	0.25		0.25
		BBK1-BBK2	152	136	0.25		0.25
SEC.	S.S.BL14Ka						
		BL14-BLK1	494	443	0.81		0.81
		BLK1-BLK2	444	398	0.73		0.73
		BLK2-BLK3	274	245	0.45		0.45
		BLK3-BLK4	214	191	0.35		0.35
SEC.	S.S.MUKOMUKO						
		BL16-BM1	916	820	1.50		1.50
		BM1-BM2	847	758	1.39		1.39
		BM2-BM3	806	721	1.32		1.32
		BM3-BM4	527	472	0.86		0.86
		BM4-BM5	284	256	0.47		0.47
		BM5-BM6	243	219	0.40		0.40
		BM6-BM7	92	83	0.15		0.15
SEC.	S.S.TANAHREKA						
		BM4-BT1	243	216	0.40		0.40
		BT1-BT2	73	65	0.12		0.12