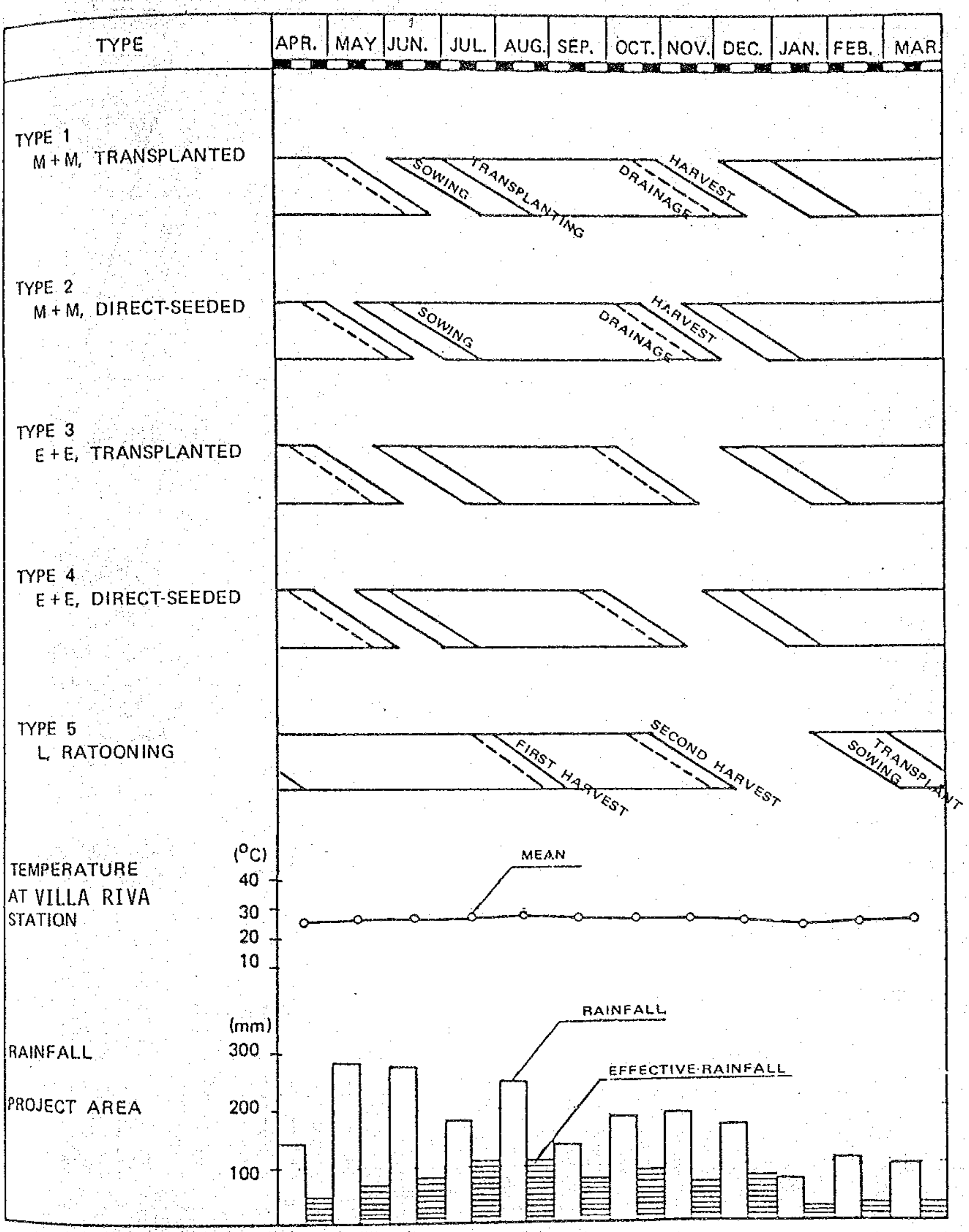
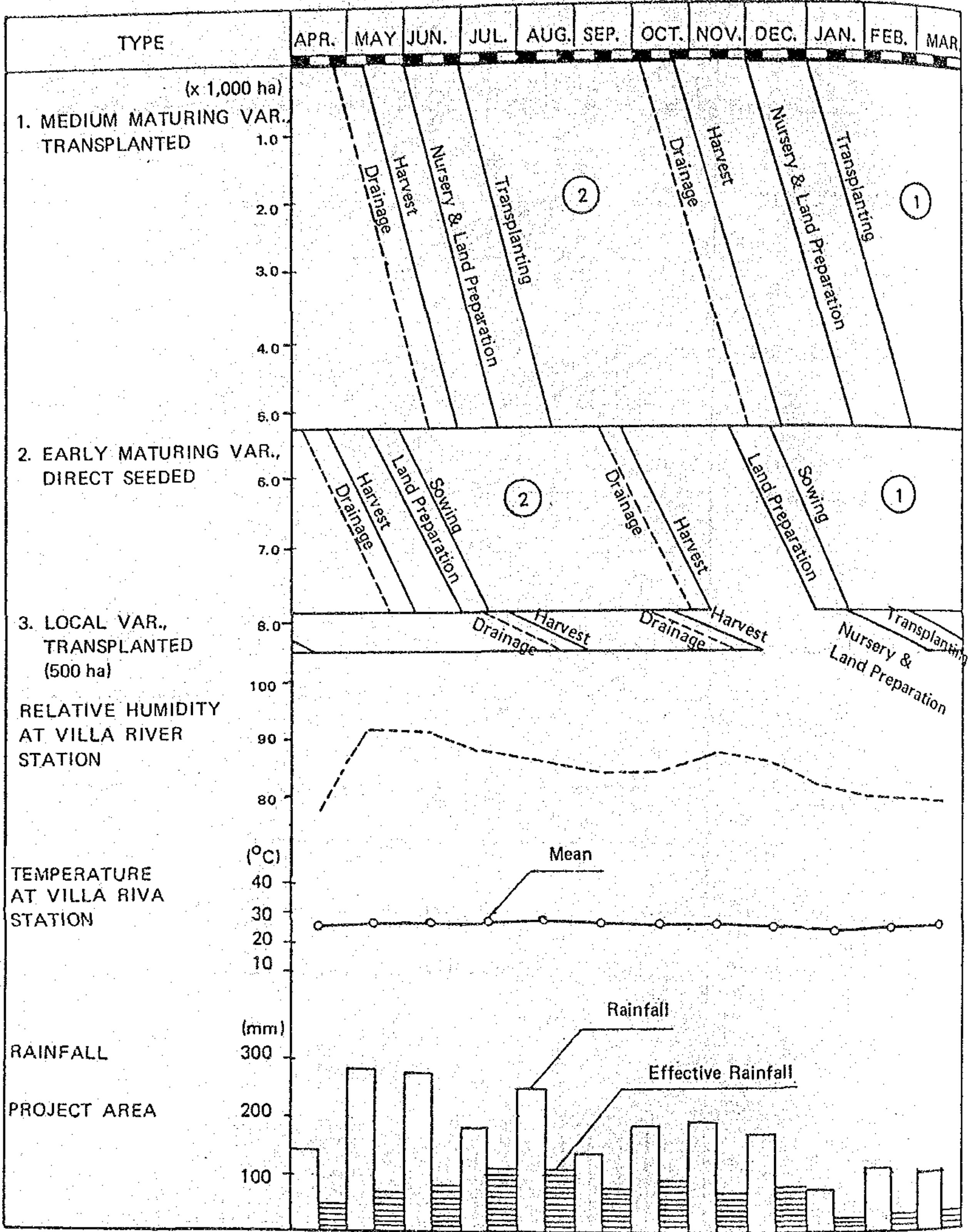
FIG. I.2.1 PROPOSED CROPPING CALENDAR



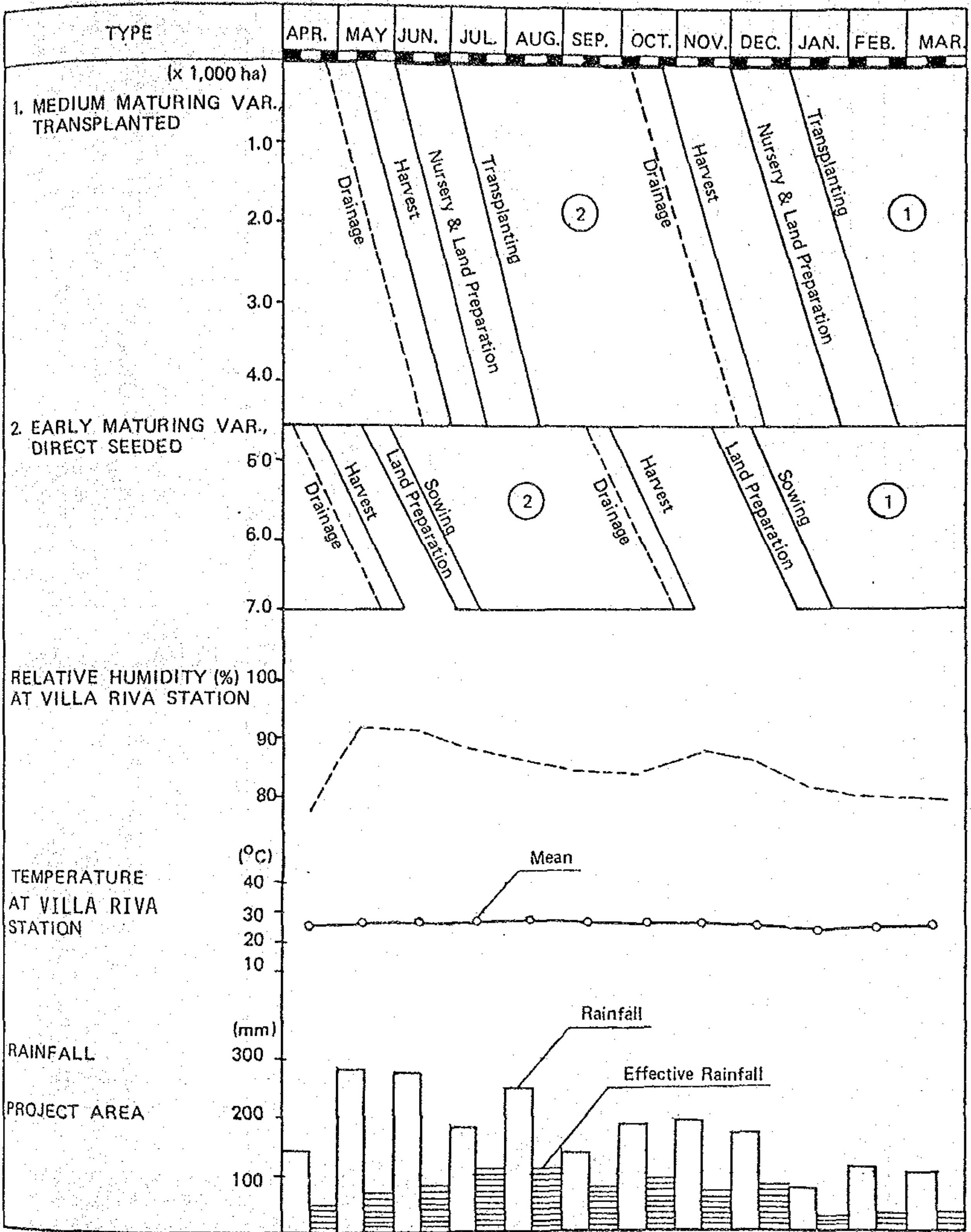
Note; M ...... MEDIUM MATURING VARIETIES (JUMA, 57, 58, 51)
E .... EARLY MATURING VARIETIES (ISA 40, TANIOKA 5)
L .... LOCAL VARIETIES (MINGORO, etc.)

FIG. I.2.2 CROPPING PATTERN (ALTERNATIVE A)



Note: M ...... MEDIUM MATURING VARIETIES (JUMA, 57, 58, 51)
E ...... EARLY MATURING VARIETIES (ISA 40, TANIOKA 5)
L ...... LOCAL VARIETIES (MINGORO, etc.)

FIG. I.2.3 CROPPING PATTERN (ALTERNATIVE B)



Note; M ...... MEDIUM MATURING VARIETIES (JUMA, 57, 58, 51)
E .... EARLY MATURING VARIETIES (ISA 40, TANIOKA 5)
L .... LOCAL VARIETIES (MINGORO, etc.)

FIG. I.3.1 PROPOSED FARMING PRACTICE, TRANSPLANTING RICE, IMPROVED VARIETIES

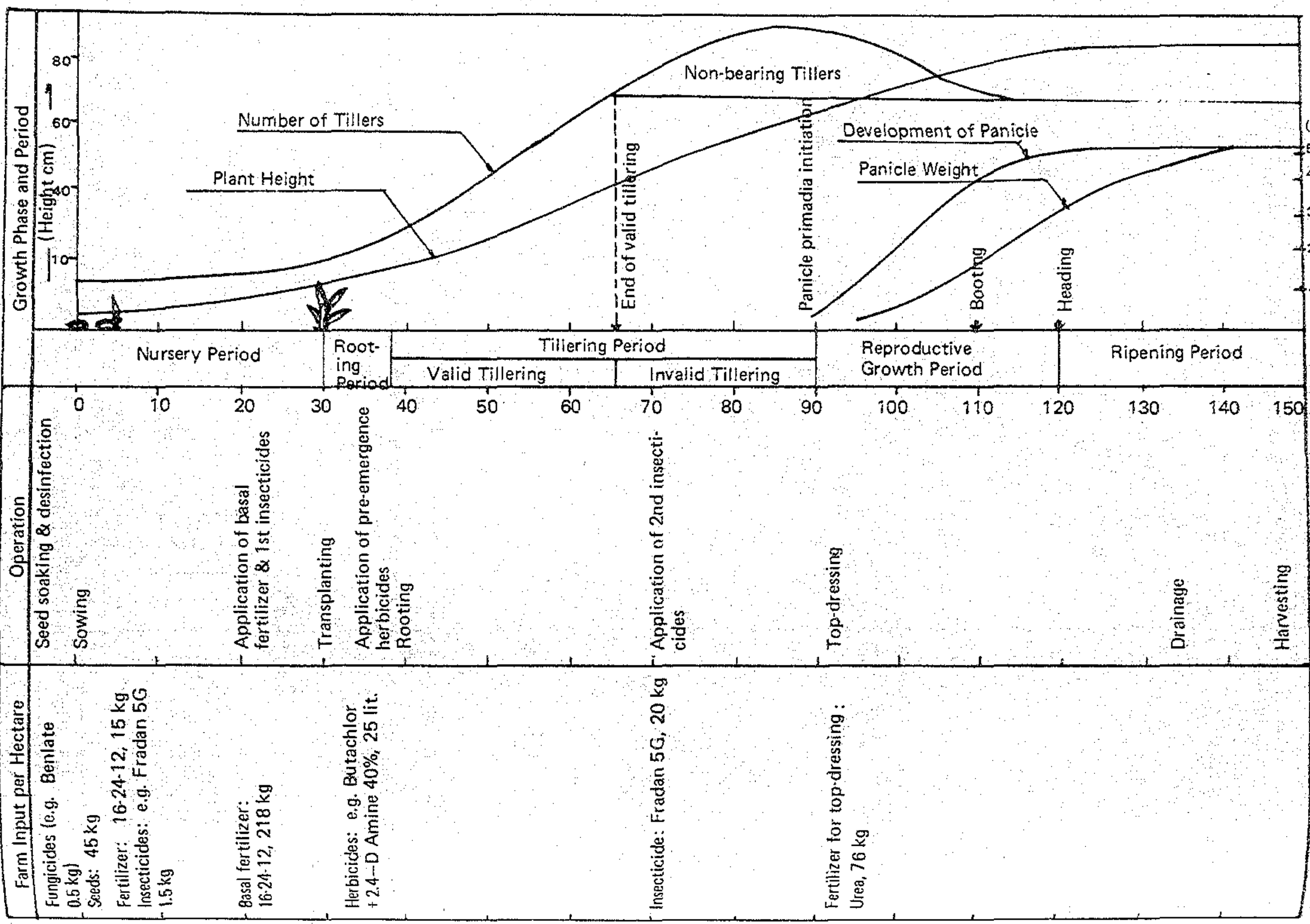


FIG. I.3.2 PROPOSED FARMING PRACTICE, DIRECT-SEEDING RICE, IMPROVED VARIETIES

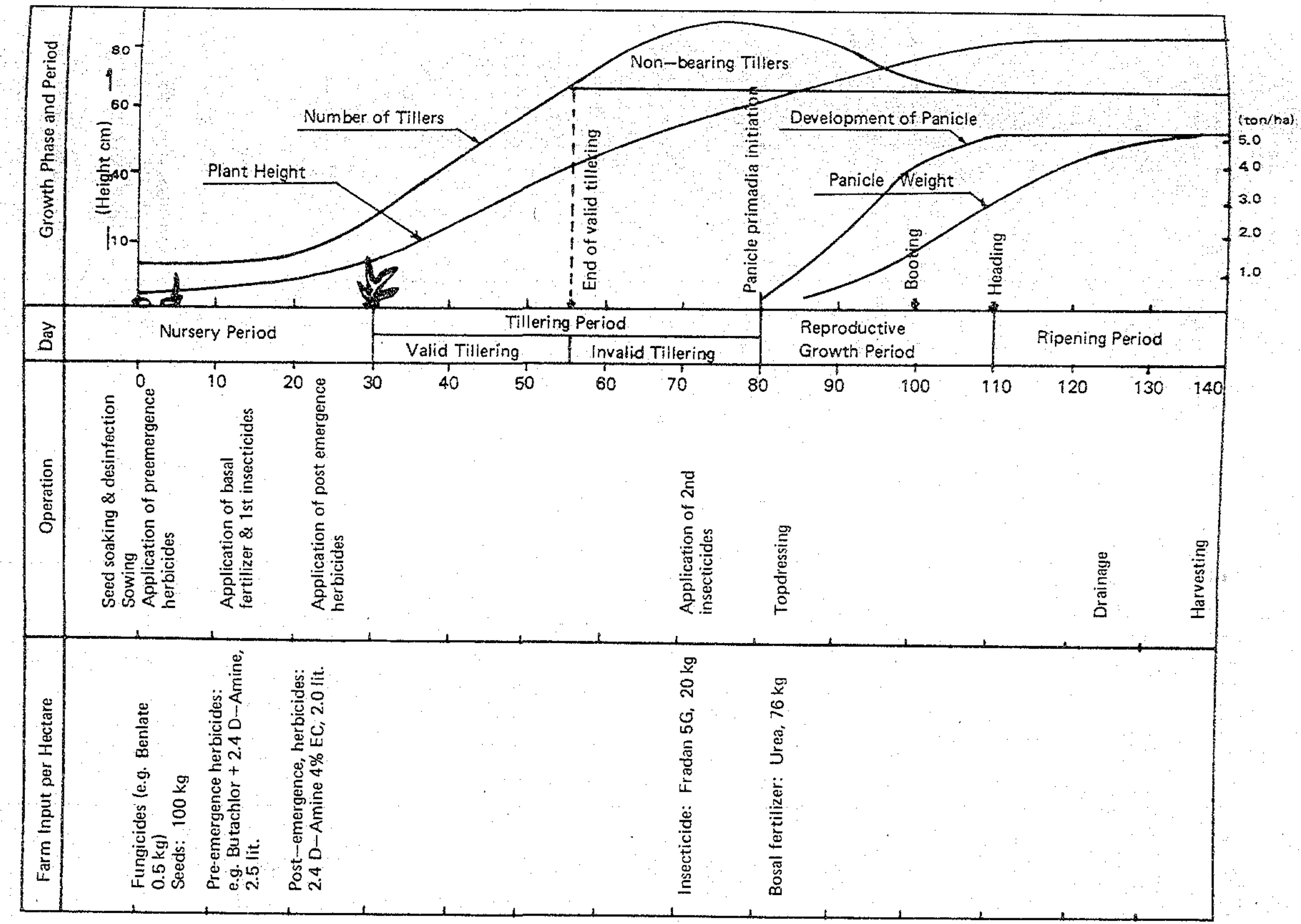


FIG. I.3.3 PROPOSED FARMING PRACTICE, TRANSPLANTING RICE, LOCAL VARIETIES

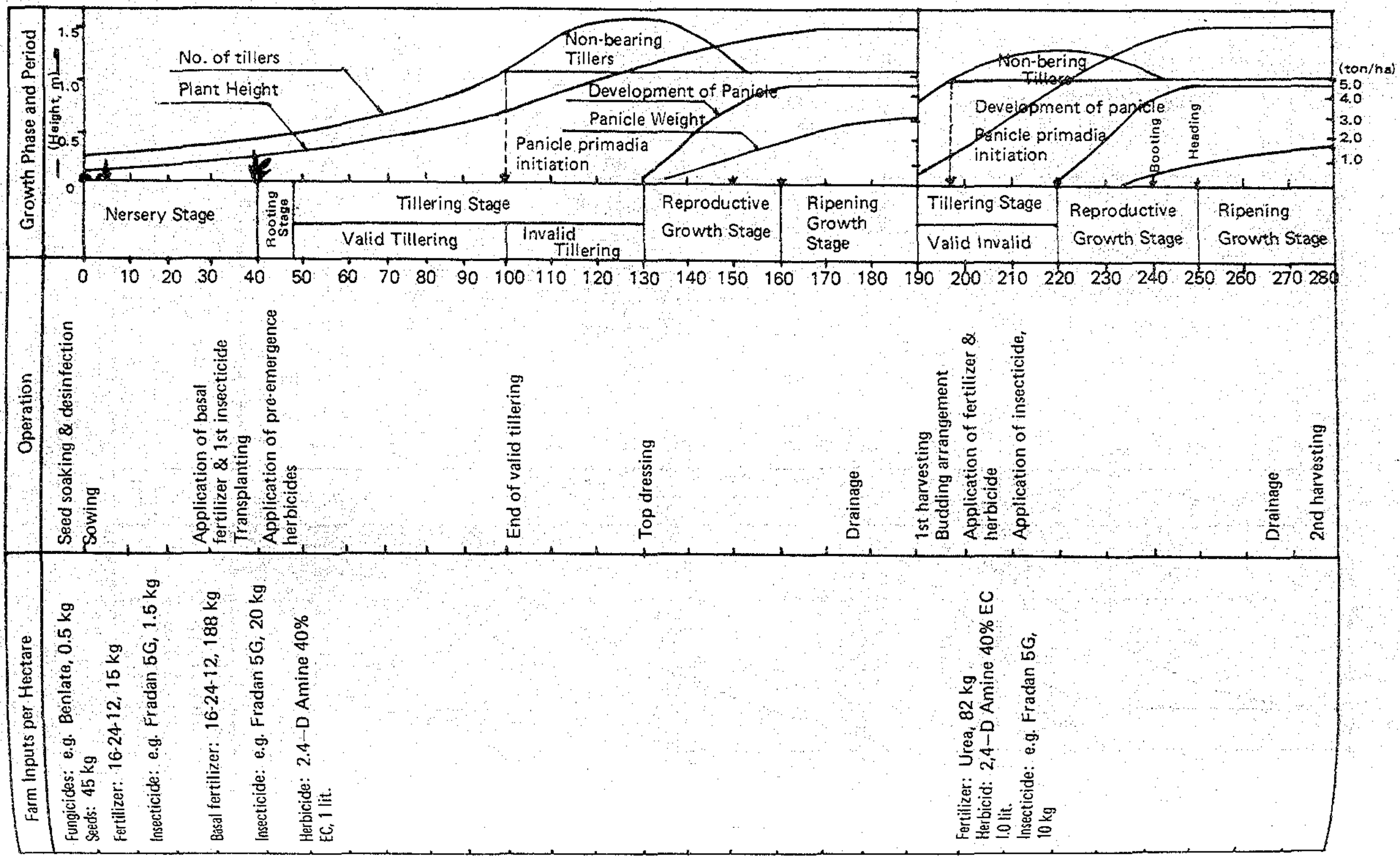
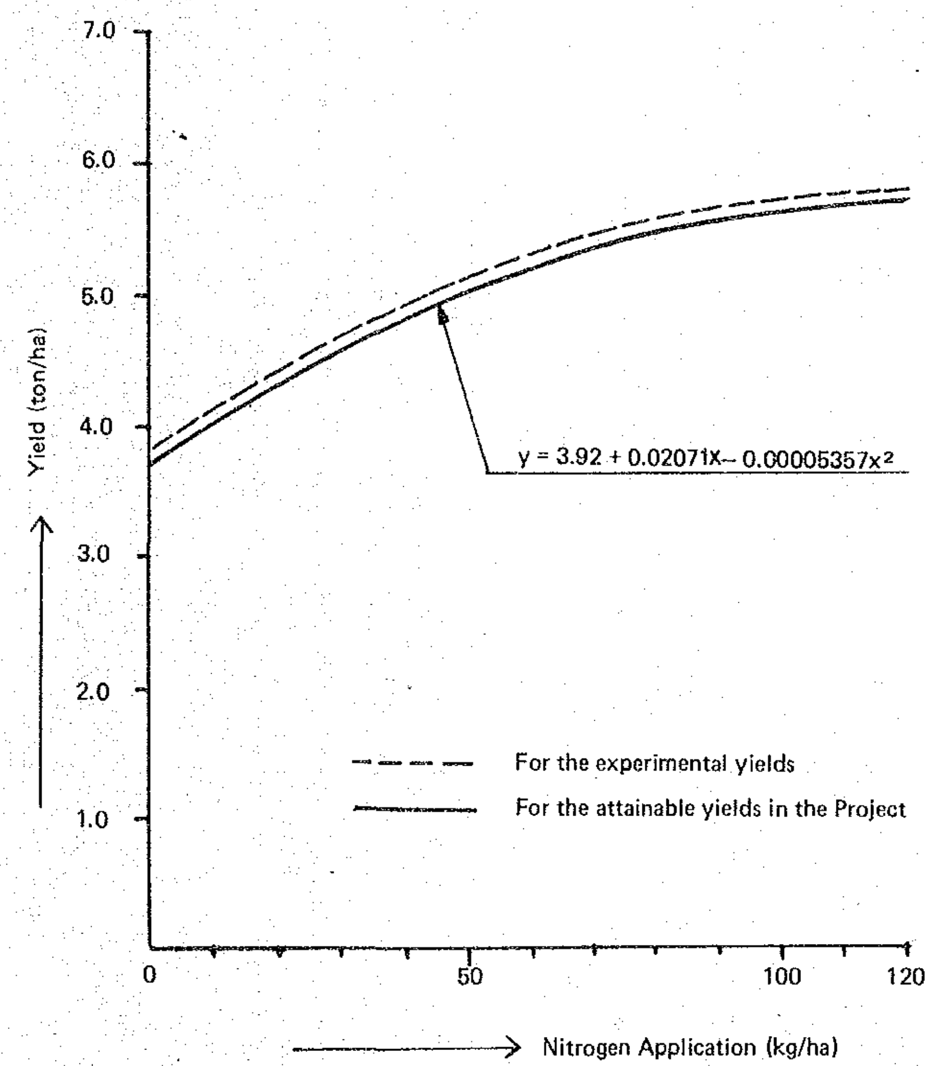


FIG. I.4.1 REGRESSION BETWEEN RICE YIELD AND APPLIED NITROGEN AMOUNT



Note: The experimental data of yields and the applied nitrogen are indicated at each dots for the mean yields between Juma 57 and Fuma 58 (Bonao Experimental Station, CEDIA, Planted in January 1973)

FIG. I.5.1 FARM OPERATION METHOD OF RICE CULTIVATION

	Plowing & Soil Breaking	Harrowing & Leveling	Trans-planting	Spraying	Weeding	Harvesting	Threshing	Drying	Hauling
1. 4-wheel Tractor + Draft Animal	4-wheel Tractor (Disc Harrow)	Draft Animal (Harrow & Leveler)	Manpower	Manpower (Hand Sprayer)	Manpower (Rotary Weeder)	Manpower (Sickle)	Power Thresher	Sunshine & Dryer	Draft Animal (Cart)
2. Power Tiller	Power Tiller (Rotary)	Power Tiller (Harrow & Leveler)	Manpower	Manpower (Hand Sprayer)	Manpower (Rotary Weeder)	Manpower (Sickle)	Power Thresher	Sunshine & Dryer	Power Tiller (Trailer)

Note: (1) The farm operation method of the direct seeded rice is the same as the above method except for the operation of seeding
(2) No. of passing of each operation are indicated in Table I.5.1

ANNEX J: IRRIGATION PLAN

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LIST OF FIGURES

ANNEX J: IRRIGATION PLAN

1. Source for Irrigation Water

1.1 General

Water for irrigation within the study area and into circumference may be obtained from one or more of the following sources:

- The Yuna River
- Tributaries within the study area: Caño Gran Estero, Caño Ponton, Caño Moreno, the Guayabo River
- Groundwater in marshes

The selection among these sources of the most appropriate one for the development area depends on their reserves, water quality, topographic and geological conditions, etc. After having studied these factors, the Yuna River has been selected as the principal source to supply the development area with irrigation water; others may be relied upon as supplementary sources of irrigation water to areas where the conduction of water from the Yuna River is technically unfavorable from topographical viewpoint. Technical factors are as mentioned below.

(1) The Yuna River presents no constrains on volume and quality of water and diversion of irrigation water by gravity is technically feasible by constructing headworks with less operation cost.

(2) Tributaries within the study area face with the deficiency of water during droughty season due to the limited extension of the catchment area and lower preservation capacity of their basin extended over marshes.

(3) Groundwater existing marshes has such constraint as to be reduced water collecting capacity and to be acidificated by the resolution of peat soils with the progress of development.

(4) In case of all sources except the Yuna River, pumping will be the only proposal to obtain irrigation water; furthermore, the topographic condition forces to construct pumping station on poor foundation well as to conduct irrigation water from lower to higher lands.

1.2 Available Irrigation Water from the Yuna River

1.2.1 Minimum Discharge at Intake Site

According to the water balance study carried out for the detailed design of the El Pozo Project (see ANNEX B.5.3), the minimum discharges of the Yuna River at Arenoso for the return period of 5 years are as summarized in the table below.

MINIMUM DISCHARGE AT ARENOSO

Case	Discharge at the Hatillo Dam	Minimum Discharge for the Return Period of 1/5 (m^3/s)
1	Regulated dischrge at 29.3 m^3/s will be secured for the purpose of power generation	22.7
2	Regulated discharge at 20.0 m^3/s will be secured for the purpose of power generation	17.1
3	Discharge will be made to satisfy the irrigation requirements of the lower catchment area	25.3
4	Discharge will be made to satisfy the irrigation requirements of the Yuna Canal	14.0

As mentioned above the minimum discharge at Arenoso varies according with the discharge at the Hatillo Dam. Though the discharge program at the Hatillo Dam has not been established, the Study Team consulted with the INDRHI and an agreement was made between the parties that the discharge of 29.0 m^3/s at the Hatillo Dam would be applied for this purpose. Accordingly, the irrigation scheme will be established on the assumption that the minimum discharge at Arenoso is to be secured at 22.7 m^3/s for the return period of 5 years.

1.2.2 Discharge to Lower Stream

Water discharge from the study area to the lower catchment area has not been regulated. In the cases of the North Yaque Irrigation Project and the AGLIPO Agricultural Development Project, this volume had been determined equivalent to the minimum discharge in the past; for the AGLIPO (El Pozo) Project it was established to be 5.5 m^3/s. This discharge may be determined considering the following factors.

a. Preservation of the irrigation water for the lower catchment area
b. Environmental conservation of resources in the river-mouth area
c. Maintenance of the river operation

Among three factors cited above, b. and c. can be disregarded their influence being negligible. On the contrary, preservation of the river flow to the paddy field which is located on the right bank should be taken into account.

In order to supply the total paddy field ranging 2,000 - 3,000 ha with irrigation, the irrigation water requirement is estimated to be at between 2 m^3/s and 3 m^3/s.

The discharge of 5.5 m^3/s which was established in the El Pozo Project will be also applied in planning the irrigation scheme of the study.

1.2.3 Available Irrigation Water

The available irrigation water from the Yuna River to the development area is calculated by substracting the water requirement for the El Pozo Project and the preservation dischrge to the lower stream from the minimum discharge at Arenoso.

Minimum Discharge at Arenoso		22.7 m^3/s
Water Requirement for the El Pozo Project	(-)	5.5 m^3/s
Preservation Discharge	(-)	5.5 m^3/s
Available Irrigation Water		11.7 m^3/s

2. Net Irrigation Requirements

2.1 Reference Crop Evapotranspiration, ETo

Reference crop evapotranspiration has been computed by applying the Penman Method. Climate data for this purpose are collected at the Barraquito Station which is located adjacent to the study area and provides more reliable data.

The form of the equation used in this method is:

$$ETo = c.[\underbrace{W.Rn}_{\text{radiation term}} + \underbrace{(1-W).f(u).(ea-ed)}_{\text{aerodynamic term}}]$$

where, ETo : reference crop evapotranspiration in mm/day

W : temperature-related weighting factor

Rn : net radiation in equivalent evaporation in mm/day

f(u) : wind-related function

(ea-ed): difference between the saturation vapour pressure at mean air temperature and the mean actual vapour pressure of the air, both in mbar

c : adjustment factor to compensate for the effect of day and night weather conditions

ETo computation is summarized in Table J.2.1.

TABLE J.2.1 CALCULATION OF REFERENCE CROP EVAPOTRANSPIRATION

	JUN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	Reference
(1) T mean °c	23.5	23.8	24.6	25.6	26.5	27.4	27.2	27.2	27.1	26.7	25.5	23.7	Appendix J.2.1
(2) RH mean %	85.1	83.8	81.7	79.3	81.9	83.1	83.6	84.2	84.3	84.6	86.0	86.3	Appendix J.2.2
(3) ea mbar	29.0	29.5	30.9	32.8	34.7	36.5	36.1	36.1	35.9	35.1	32.7	29.3	Appendix J.2.5
(4) ed=(2)x(3) mbar	24.7	24.7	25.2	26.0	28.4	30.3	30.2	30.4	30.3	29.7	28.1	25.3	
(5) ea-ed mbar	4.3	4.8	5.7	6.8	6.3	6.2	5.9	5.7	5.6	5.4	4.6	4.0	
(6) Wind U km/day	130	138	147	164	147	138	130	104	112	104	104	104	Appendix J.2.3
(7) $f(u)=0.27(1+\frac{U}{100})$	0.62	0.64	0.67	0.71	0.67	0.64	0.62	0.55	0.57	0.55	0.55	0.55	
(8) Weighting factor (1-W)	0.27	0.27	0.26	0.25	0.24	0.24	0.24	0.24	0.24	0.24	0.25	0.27	Appendix J.2.7
(9) Weighting factor (W)	0.73	0.73	0.74	0.75	0.76	0.76	0.76	0.76	0.76	0.76	0.75	0.73	Appendix J.2.8
(10) Ra mm/day	11.2	12.7	14.4	15.6	16.3	16.4	16.3	15.9	14.8	13.3	11.6	10.7	Appendix J.2.9
(11) n/N Ratio	0.60	0.64	0.64	0.62	0.57	0.58	0.58	0.58	0.59	0.59	0.59	0.59	Appendix J.2.4
(12) (0.25+0.5n/N)	0.55	0.57	0.57	0.56	0.54	0.54	0.54	0.54	0.55	0.55	0.55	0.55	
(13) Rs=(12)x(10) mm/day	6.2	7.2	8.2	8.7	8.8	8.9	8.8	8.6	8.1	7.3	6.4	5.9	
(14) Rns=(1-α)x(13) mm/day	4.7	5.4	6.2	6.5	6.6	6.7	6.6	6.5	6.1	5.5	4.8	4.4	α=0.25
(15) f(T)	15.3	15.4	15.6	15.8	16.0	16.2	16.1	16.1	16.1	16.0	15.8	15.3	Appendix J.2.11
(16) f(ed)	0.12	0.12	0.12	0.12	0.12	0.11	0.11	0.11	0.11	0.12	0.12	0.12	Appendix J.2.12
(17) f(n/N)	0.64	0.68	0.68	0.66	0.62	0.62	0.62	0.62	0.63	0.63	0.63	0.63	Appendix J.2.13
(18) Rnl=(15)x(16)x(17)	1.2	1.3	1.3	1.3	1.2	1.1	1.1	1.1	1.1	1.2	1.2	1.2	
(19) Rn=(14)-(18)	3.5	4.1	4.9	5.2	5.4	5.6	5.5	5.4	5.0	4.3	3.6	3.2	
(20) Adjustment factor (C)	1.05	1.07	1.08	1.13	1.14	1.14	1.14	1.12	1.08	1.07	1.05	1.05	Appendix J.2.14
(21) (9)x(19)	2.56	2.99	3.63	3.90	4.10	4.26	4.18	4.10	3.80	3.27	2.70	2.34	
(22) (8)x(7)x(5)	0.72	0.83	0.99	1.21	1.01	0.95	0.88	0.75	0.77	0.71	0.63	0.59	
(23) (21)+(22)	3.28	3.82	4.62	5.11	5.11	5.21	5.06	4.85	4.57	3.98	3.33	2.93	
(24) ETo=(20)x(23) mm/day	3.4	4.1	5.0	5.8	5.8	5.9	5.8	5.4	4.9	4.3	3.5	3.1	
(25) For Project F	1.10	1.12	1.13	1.15	1.15	1.15	1.15	1.15	1.13	1.12	1.11	1.10	
(26) ETo mm/day	3.7	4.6	5.7	6.7	6.7	6.8	6.7	6.2	5.5	4.8	3.9	3.4	
(27) ETo mm/month	114.7	128.8	176.7	201.0	207.7	204.0	207.7	192.0	165.0	148.8	117.0	105.4	
* ETo of Elpozo mm/month	112	129	177	192	198	192	198	192	171	146	117	109	

2.2 Crop Water Requirements, ETcrop

Crop water requirements have been estimated by the equation as follows:

$$ETcrop = Kc \times ETo$$

where, Kc : crop coefficient
ETo : reference crop evapotranspiration

and the results are shown in Tables J.2.2 - J.2.4.

The crop coefficients, which have been obtained according with FAO's guideline, are as follows:

Period of Growth	Crop Coefficient
1st month	1.1
2nd month	1.1
Mid-season	1.05
Last 4 weeks	0.95

2.3 Field Water Requirements

Field water requirements may be obtained by adding deep percolation and water requirement for land preparation and nursery to crop water requirement to respond the proposed cropping patterns. The calculation is summarized in Tables J.2.2 - J.2.4.

(1) Proposed Cropping Patterns

In compliance with the prevailing farm conditions, the following three proposals on cropping patterns have been presented:

Cropping Pattern A : Medium maturing improved varieties will be sowed twice a year by means of transplanting

Cropping Pattern B : Medium maturing improved varieties will be sowed twice a year by means of direct seeding

Cropping Pattern C : Traditional varieties will be sowed by transplanting for the first stage and by ratooning for the second stage

(2) Water Requirements for Land Preparation and Nursery

Water requirements for land preparation and nursery are calculated to be 100 mm for both first and second harvests.

2.4 Irrigation Water Requirements

Irrigation water requirements are calculated by means of the formula:

$$I.W.R. = \frac{F.W.R. - \text{Effective Rainfall}}{\text{Crop Coefficient}}$$

where, I.W.R. = Irrigation Water Requirements
F.W.R. = Field Water Requirements

(1) Effective Rainfall

Effective rainfall for the return period of five years has been estimated employing 15 years (1969-1983) rainfall data with the computation of their probability.

In estimating the effective rainfall, the following equation has been applied to meet the height of levee.

$$Re = R \times 0.8 - (ETcrop + P) \leq 100 \text{ mm}$$

where, Re : Effective rainfall
R : Rainfall
ETcrop: Crop water requirement
P : Deep percolation 0.5 mm/day

The results of this estimation is presented in Appendix J.2.16.

(2) Irrigation Efficiency

Irrigation efficiency has been calculated as follows:

E = Ec.Eb.Ea = 0.58

where, E : Irrigation efficiency

Ec : Conveyance efficiency = 0.9

Eb : Field canal efficiency = 0.8

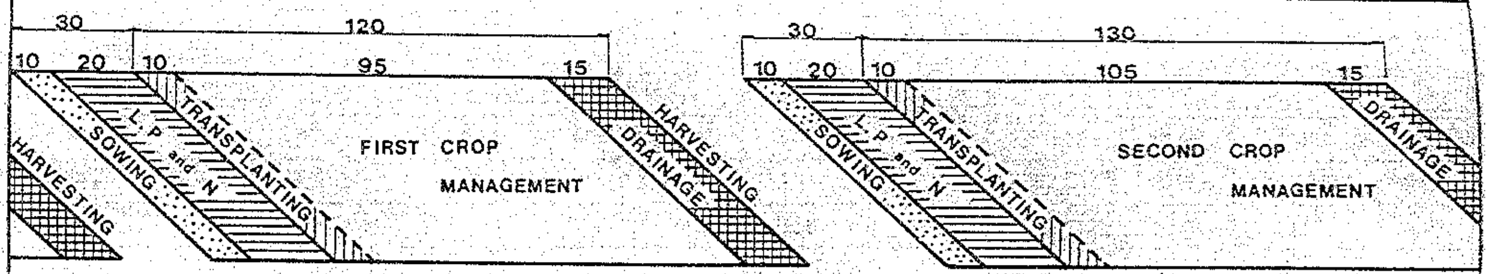
Ea : Field application efficiency = 0.8

(3) Irrigation Water Requirements

Irrigation water requirements for each month have been calculated after obtaining field water requirements, effective rainfall and irrigation efficiency (see Table J.2.5).

(1) Cropping Pattern A

Note ; C.W.R : Crop Water Requirement
L.P : Land Preparation
N : Nursery
F.C : Flooding for Cultivation Water



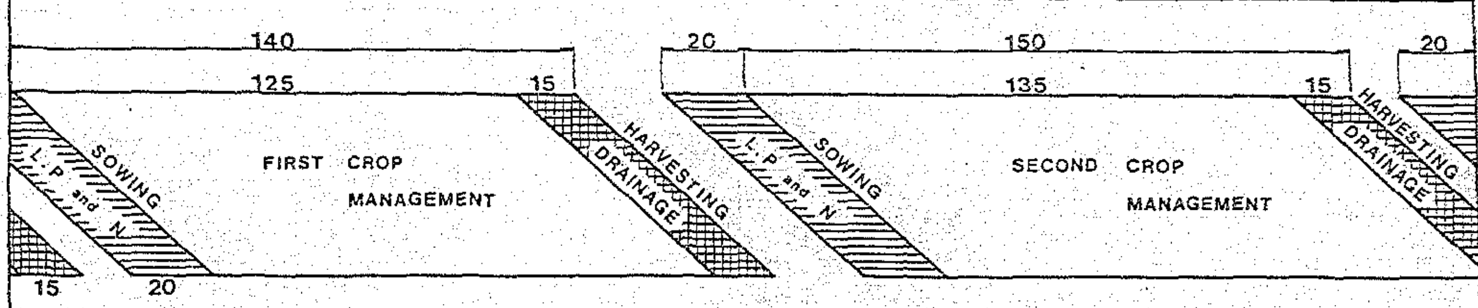
		JUN	JUL	AUG	SEP	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY
(2) ET₀	(mm/month)	204.0	207.7	192.2	165.0	148.8	117.0	105.4	114.7	128.8	176.7	201.0	207.7
(3) Crop Coefficiency		1.00	1.10	1.10	1.10	1.05	0.95	1.10	1.10	1.10	1.10	1.05	0.95
(4) C.W.R	(mm/month)	204.0	228.5	211.4	181.5	156.2	111.2	115.9	126.2	141.7	194.4	212.0	197.3
(5) Area Factor of C.W.R		0.074	0.318	0.884	1.000	0.895	0.340	0.009	0.318	0.871	1.000	0.992	0.590
(6) Weighted C.W.R	(mm/month)	15.1	72.7	186.9	181.5	139.8	37.8	1.0	40.1	123.4	194.4	210.3	116.4
(7) Area Factor of L.P and N		0.200	0.620	0.180	-	-	-	0.200	0.620	0.180	-	-	-
(8) Water Requirement L.P and N	(mm/month)	20.0	62.0	18.0	-	-	-	20.0	62.0	18.0	-	-	-
(9) Field Water Requirement	(mm/month)	35.1	134.7	204.9	181.5	139.8	37.8	21.0	102.1	141.4	194.4	210.3	116.4
(10) Effective Rainfall	(mm/month)	74.3	113.0	114.8	85.0	102.1	67.7	39.0	32.4	37.6	37.3	57.0	77.0
(11) Area Factor of Effective Rainfall		0.207	0.718	1.000	1.000	0.895	0.340	0.138	0.718	1.000	1.000	0.992	0.590
(12) Weighted Effective Rainfall	(mm/month)	15.4	81.1	114.8	85.0	91.4	23.0	5.4	23.3	37.6	37.3	56.5	45.4
(13) Net Irrigation Requirement	(mm/month)	19.7	53.6	90.1	96.5	48.4	14.8	15.6	78.8	103.8	157.1	153.8	71.0
(14) Irrigation Efficiency		0.58	0.58	0.58	0.58	0.58	0.58	0.58	0.58	0.58	0.58	0.58	0.58
(15) Irrigation Requirement	(mm/month)	34.0	92.4	155.3	166.4	83.4	25.5	26.9	135.9	179.0	270.9	265.2	122.4
(16) "	(mm/day)	5.469	4.152	5.011	5.546	3.006	2.502	6.287	6.104	6.392	8.737	8.883	6.693
(17) "	(l/s/ha)	0.633	0.481	0.580	0.642	0.348	0.290	0.728	0.706	0.740	1.011	1.028	0.775

Note :
(4) = (2) x (3)
(6) = (4) x (5)
(8) = 100.0 x (7)
(9) = (6) + (8)
(12) = (10) x (11)
(13) = (9) - (12)
(15) = (13)/(14)
(16) = (15)/(Days of Month x (11))
(17) = (16)/8.64

TABLE J.2.3 CALCULATION OF UNIT WATER REQUIREMENT (B)

(1) Cropping Pattern B

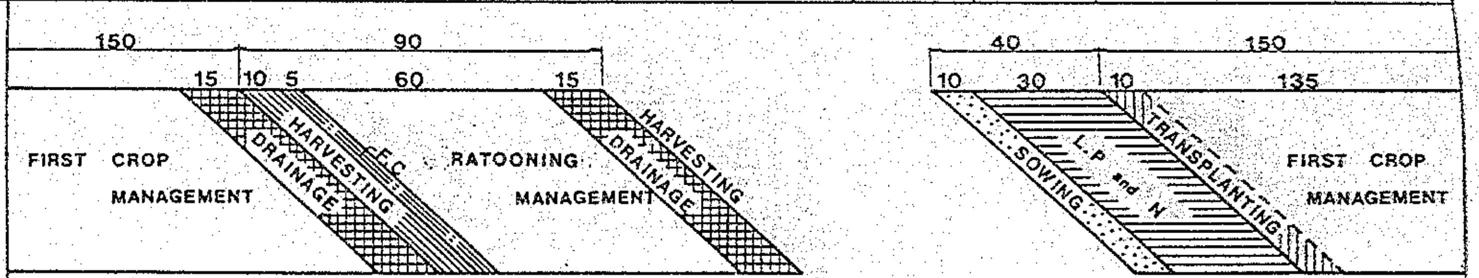
Note ; C.W.R : Crop Water Requirement
L.P : Land Preparation
N : Nursery
F.C : Flooding for Cultivation Water



	JUN	JUL	AUG	SEP	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY
(2) ET_o (mm/month)	204.0	207.7	192.2	165.0	148.8	117.0	105.4	114.7	128.8	176.7	201.0	207.7
(3) Crop Coefficiency	1.10	1.10	1.10	1.05	1.00	0.95	1.10	1.10	1.10	1.05	1.00	0.95
(4) Crop Water Requirement (mm/month)	224.4	228.5	211.4	173.3	148.8	111.2	115.9	126.2	141.7	185.5	201.0	197.3
(5) Area Factor of C.W.R	0.303	0.871	1.000	1.000	0.747	0.161	0.310	0.884	1.000	1.000	0.915	0.370
(6) Weighted C.W.R (mm/month)	68.0	199.0	211.4	173.3	111.2	17.9	35.9	111.6	141.7	185.5	183.9	73.0
(7) Area Factor of L.P and N	0.600	0.200	-	-	-	0.200	0.620	0.180	-	-	-	0.200
(8) Water Requirement L.P and N (mm/month)	60.0	20.0	-	-	-	20.0	62.0	18.0	-	-	-	20.0
(9) Field Water Requirement (mm/month)	128.0	219.0	211.4	173.3	111.2	37.9	97.9	129.6	141.7	185.5	183.9	93.0
(10) Effective Rainfall (mm/month)	74.3	113.0	114.8	85.0	102.1	67.7	39.0	32.4	37.6	37.3	57.0	77.0
(11) Area Factor of Effective Rainfall	0.712	1.000	1.000	1.000	0.747	0.295	0.710	1.000	1.000	1.000	0.915	0.499
(12) Weighted Effective Rainfall (mm/month)	52.9	113.0	114.8	85.0	76.3	20.0	27.7	32.4	37.6	37.3	52.2	38.4
(13) Net Irrigation Requirement (mm/month)	75.1	106.0	96.6	88.3	34.9	17.9	70.2	97.2	104.1	148.2	131.7	54.6
(14) Irrigation Efficiency	0.58	0.58	0.58	0.58	0.58	0.58	0.58	0.58	0.58	0.58	0.58	0.58
(15) Irrigation Requirement (mm/month)	129.5	182.8	166.6	152.2	60.2	30.9	121.0	167.6	179.5	255.5	227.1	94.1
(16) " (mm/day)	6.063	5.897	5.374	5.073	2.600	3.492	5.498	5.406	6.411	8.242	8.273	6.083
(17) " (l/s/ha)	0.702	0.682	0.622	0.587	0.301	0.404	0.636	0.626	0.742	0.954	0.958	0.704

Note : (4) = (2) × (3) (9) = (6) + (8) (15) = (13)/(14)
(6) = (4) × (5) (12) = (10) × (11) (16) = (15)/(Days of Month × (11))

	JUN	JUL	AUG	SEP	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY
(1) Cropping Pattern												



Note ; C.W.R : Crop Water Requirement
L.P : Land Preparation
N : Nursery
F.C : Flooding for Cultivation Water

	JUN	JUL	AUG	SEP	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY
(2) ET₀ (mm/month)	204.0	207.7	192.2	165.0	148.8	117.0	105.4	114.7	128.8	176.7	201.0	207.7
(3) Crop Coefficiency	1.05	1.00	1.00	1.10	1.05	0.95	0.95	1.10	1.10	1.10	1.10	1.10
(4) Crop Water Requirement (mm/month)	214.2	207.7	192.2	181.5	156.2	111.2	100.1	126.2	141.7	194.4	221.1	228.5
(5) Area Factor of C.W.R	1.000	0.895	0.445	0.661	0.884	0.320	0.000	0.002	0.010	0.315	0.880	1.000
(6) Weighted C.W.R (mm/month)	214.2	185.9	85.5	120.0	138.1	35.6	0.0	0.3	1.4	61.2	194.6	228.5
(7) Area Factor of L.P and N	-	-	-	-	-	-	-	0.001	0.299	0.580	0.120	-
(8) Water Requirement L.P and N (mm/month)	-	-	-	-	-	-	-	0.1	29.9	58.0	12.0	-
(9) Area Factor of F.C	-	-	0.410	0.582	0.008	-	-	-	-	-	-	-
(10) Flooding for Cultivation Water (mm/month)	-	-	41.0	58.2	0.8	-	-	-	-	-	-	-
(11) Field Water Requirement (mm/month)	214.2	185.9	126.5	178.2	138.9	35.6	0.0	0.4	30.4	119.2	206.6	228.5
(12) Effective Rainfall (mm/month)	74.3	113.0	114.8	85.0	102.1	67.7	39.0	32.4	37.6	37.3	57.0	77.0
(13) Area Factor of Effective Rainfall	1.000	0.895	0.511	0.758	0.885	0.320	0.000	0.003	0.330	0.876	1.000	1.000
(14) Weighted Effective Rainfall (mm/month)	74.3	101.1	58.7	64.4	90.4	21.7	0.0	0.1	12.4	32.7	57.0	77.0
(15) Net Irrigation Requirement (mm/month)	139.9	84.8	67.8	113.8	48.5	13.9	-	0.3	18.0	86.5	149.6	151.5
(16) Irrigation Efficiency	0.58	0.58	0.58	0.58	0.58	0.58	-	0.58	0.58	0.58	0.58	0.58
(17) Irrigation Requirement (mm/month)	241.2	146.2	116.9	196.2	83.6	24.0	-	0.5	31.0	149.1	257.9	261.2
(18) " (mm/day)	8.040	5.270	7.379	8.628	3.048	2.496	-	5.376	3.359	5.492	8.598	8.426
(19) " (1/s/ha)	0.931	0.610	0.854	0.999	0.353	0.289	0.0	0.622	0.389	0.636	0.995	0.975

Note : (4) = (2) x (3)
(6) = (4) x (5)
(8) = 100 x (7)
(10) = 100 x (9)
(11) = (6) + (8) + (10)
(14) = (12) x (13)
(15) = (11) - (14)
(17) = (15)/(16)
(18) = (17)/(Days of Month x (13))
(19) = (18)/8.64

TABLE J.2.5 UNIT IRRIGATION REQUIREMENT

Irrigation Efficiency	Cropping Pattern	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	Remarks
	A	0.706	0.740	1.011	1.028	0.775	0.633	0.481	0.580	0.642	0.348	0.290	0.728	Transplanting
E = 0.58	B	0.626	0.742	0.954	0.958	0.704	0.702	0.682	0.622	0.587	0.301	0.404	0.636	Direct Seeding
	C	0.622	0.389	0.636	0.995	0.975	0.931	0.610	0.854	0.999	0.353	0.289	0	Ratooning
	A	0.640	0.671	0.916	0.932	0.702	0.574	0.436	0.526	0.582	0.315	0.263	0.660	Transplanting
E = 0.64	B	0.567	0.672	0.865	0.868	0.638	0.636	0.618	0.564	0.532	0.273	0.366	0.576	Direct Seeding
	C	0.564	0.353	0.576	0.902	0.884	0.844	0.553	0.774	0.905	0.320	0.262	0	Ratooning

3. Return Flow

For the purpose of an efficient utilization of water resources, the use of return flow will be considered.

Given the field water requirements is 1.00 irrigation water requirement at intake point will be calculated as follows:

Field Water Requirement		:	1.00
Conveyance Loss	(+)	:	0.16
Field Canal Loss	(+)	:	0.28
Field Application Loss		:	0.28
Irrigation Water Requirements		:	1.72

Available water for return flow will be calculated adding field canal loss to field application loss, and if 60% of this volume is to be utilized repeatedly, the return flow will be calculated as follows:

Return Flow = (Field Canal Loss + Field Application Loss) x 0.6
= (0.28 + 0.28) x 0.6 = 0.34

If return flow should be used in the adjacent fields located in the lower catchment, the irrigation requirements to the adjacent field to the study area are calculated by subtracting conveyance loss from total irrigation requirement (I.R.):

I.R. = 1.72 - 0.16 = 1.56

Therefore, the project efficiency will become: 0.34 ÷ 1.56 = 0.22, which indicates that if water is taken to irrigate 100 ha of field, another 22 ha of land will be irrigated by using return flow.

4. Design Intake Rate

Taking return flow into account, the design intake rate is computed using the following equation:

$$Q = \frac{A.q}{1 + R} \times 10^{-3}$$

where, Q = intake rate (m/s)

A = irrigable area (ha)

q = net water requirement (1/s/ha)

R = project efficiency = 0.22

Then, the maximum intake rate with regard to each intake facility have been calculated as follows:

Heatworks (Yuna River) : $Q = 5.90\ m^3/s$

Pumping Station (Cruz de Rincon) : $Q = 0.46\ m^3/s$

Pumping Station (Rincon Molinillo): $Q = 0.21\ m^3/s$

Pumping Station (El Guayabo) : $Q = 0.41\ m^3/s$

Table J.4.1 shows the intake rate by month.

TABLE J.4.1 INTAKE RATE

Intake Facility	Location	Irrigable Area (ha)		Unit	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	[illegible]
Headworks	Yuna River	7,000	q	l/s/ha	0.706	0.740	1.011	1.028	0.775	0.633	0.481	0.580	0.642	0.348	0.290	[illegible]
			Q	m^3/s	4.06	4.25	5.81	5.90	4.45	3.64	2.76	3.33	3.69	2.00	1.67	[illegible]
Pumping Station	Cruce de Rincon	550	q	l/s/ha	0.706	0.740	1.011	1.028	0.775	0.633	0.481	0.580	0.642	0.348	0.290	[illegible]
			Q	m^3/s	0.32	0.33	0.46	0.46	0.35	0.29	0.22	0.26	0.29	0.16	0.13	[illegible]
	Rincon Molinillo	250	q	l/s/ha	0.706	0.740	1.011	1.028	0.775	0.633	0.481	0.580	0.642	0.348	0.290	[illegible]
			Q	m^3/s	0.14	0.15	0.21	0.21	0.16	0.13	0.10	0.12	0.13	0.07	0.06	[illegible]
	El Guayabo	500	q	l/s/ha	0.622	0.389	0.636	0.995	0.975	0.931	0.610	0.854	0.999	0.353	0.269	[illegible]
			Q	m^3/s	0.26	0.16	0.26	0.41	0.40	0.38	0.25	0.35	0.41	0.14	0.12	[illegible]

5. Division of Irrigable Area (Paddy Fields)

In accordance with topographic condition, networks of irrigation and drainage and in-farm road system, the proposed irrigable area has been divided into blocks as shown in Fig. J.5.1 and Table J.5.1.

TABLE J.5.1 DIVISION OF IRRIGABLE AREA

(Unit : ha)

Blocks	Irrigable Area	Blocks	Irrigable Area	Blocks	Irrigable Area
Block 1 -1	(114)	Block 9 -1	(301)	Block 17 -1	(227)
" 1 -2	(174)	" 9 -2	(117)	" 17 -2	(95)
" 1 -3	(83)	" 9 -3	(42)	" 17 -3	(28)
" 1 -4	(99)	Sub-total	460	Sub-total	350
Sub-total	470				
		Block 10	840	Total 1 - 17	(7,000)
Block 2 -1	(272)				
" 2 -2	(58)	Block 11	290	Block 18 -1	(152)
" 2 -3	(80)			" 18 -2	(162)
Sub-total	410	Block 12	190	" 18 -3	(180)
				" 18 -4	(56)
Block 3 -1	(90)	Block 13 -1	(330)	Sub-total	550
" 3 -2	(126)	" 13 -2	(110)		
" 3 -3	(74)	" 13 -3	(30)	Block 19 -1	(117)
Sub-total	290	Sub-total	470	" 19 -2	(78)
				" 19 -3	(550
Block 4	100	Block 14 -1	(353)	Sub-total	250
		" 14 -2	(186)		
Block 5 -1	(104)	" 14 -3	(139)	Block 20 -1	(365)
" 5 -2	(76)	" 14 -4	(95)	" 20 -2	(135)
Sub-total	180	" 14 -5	(37)	Sub-total	500
		Sub-total	810		
Block 6	490			Total 18 - 20	(1,300)
		Block 15	230		
Block 7	550			Total 1 - 20	8,300
		Block 16	430		
Block 8	440				

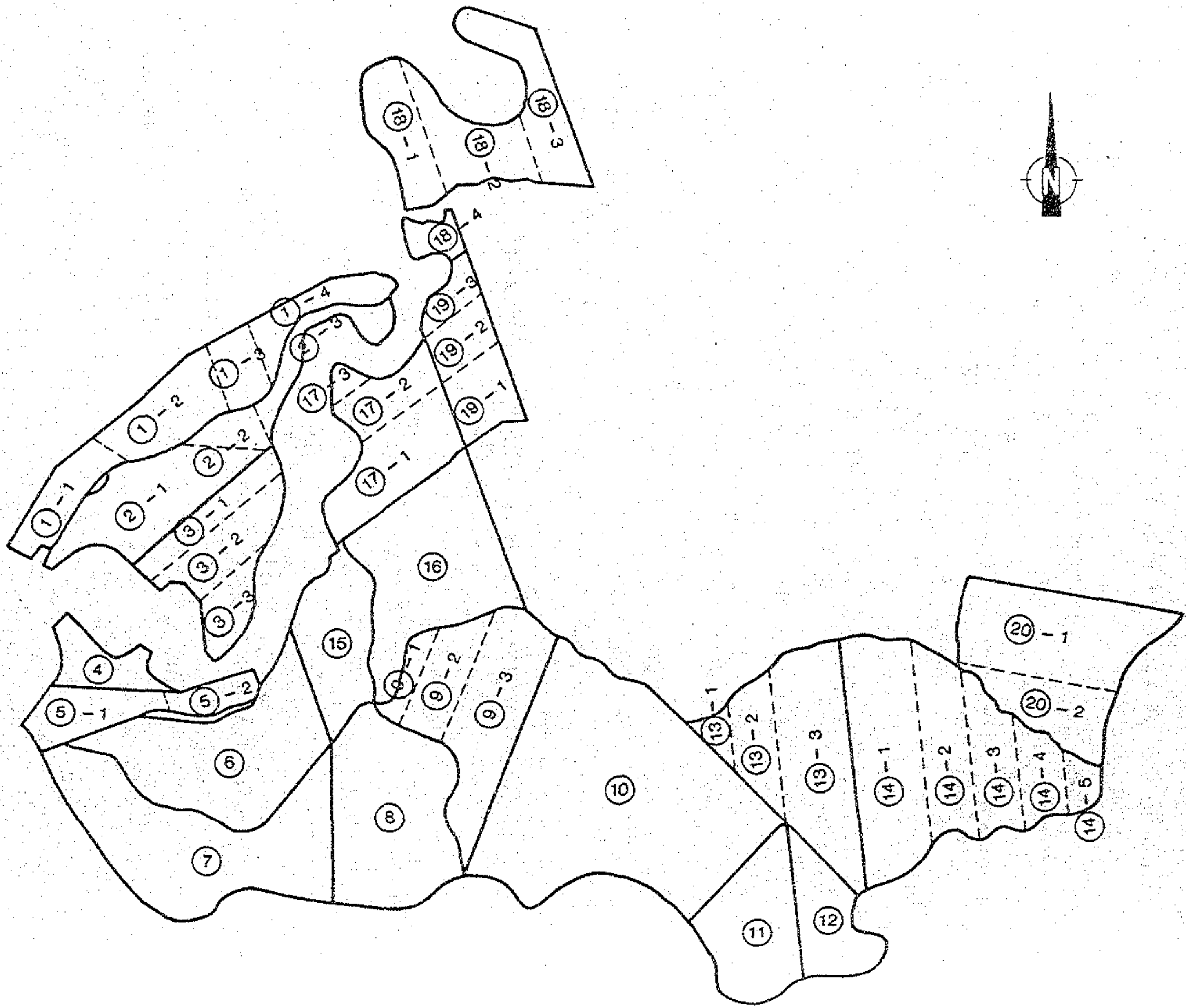


FIG. J.5.1 DIVISION OF THE PADDY FIELD BLOCK

6. Main Irrigation Canal and Irrigation Network

Based on the division of irrigation area together with topographic conditions, etc., the main irrigation canal has been delineated as shown in Fig. J.6.1. Furthermore, irrigation network proposal is illustrated in Fig. J.6.2.

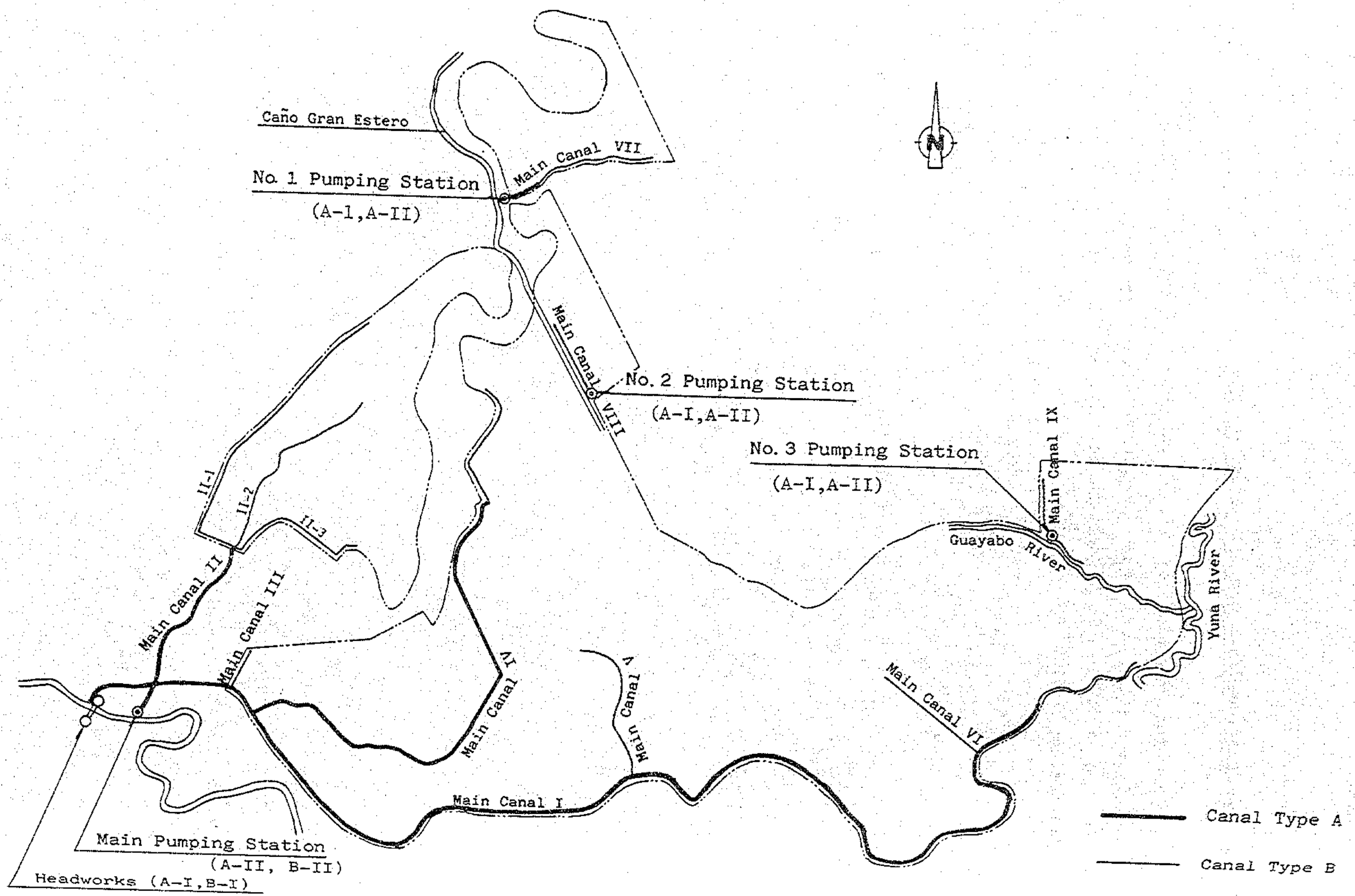


FIG. J.6.1 PLAN OF MAIN IRRIGATION CANAL

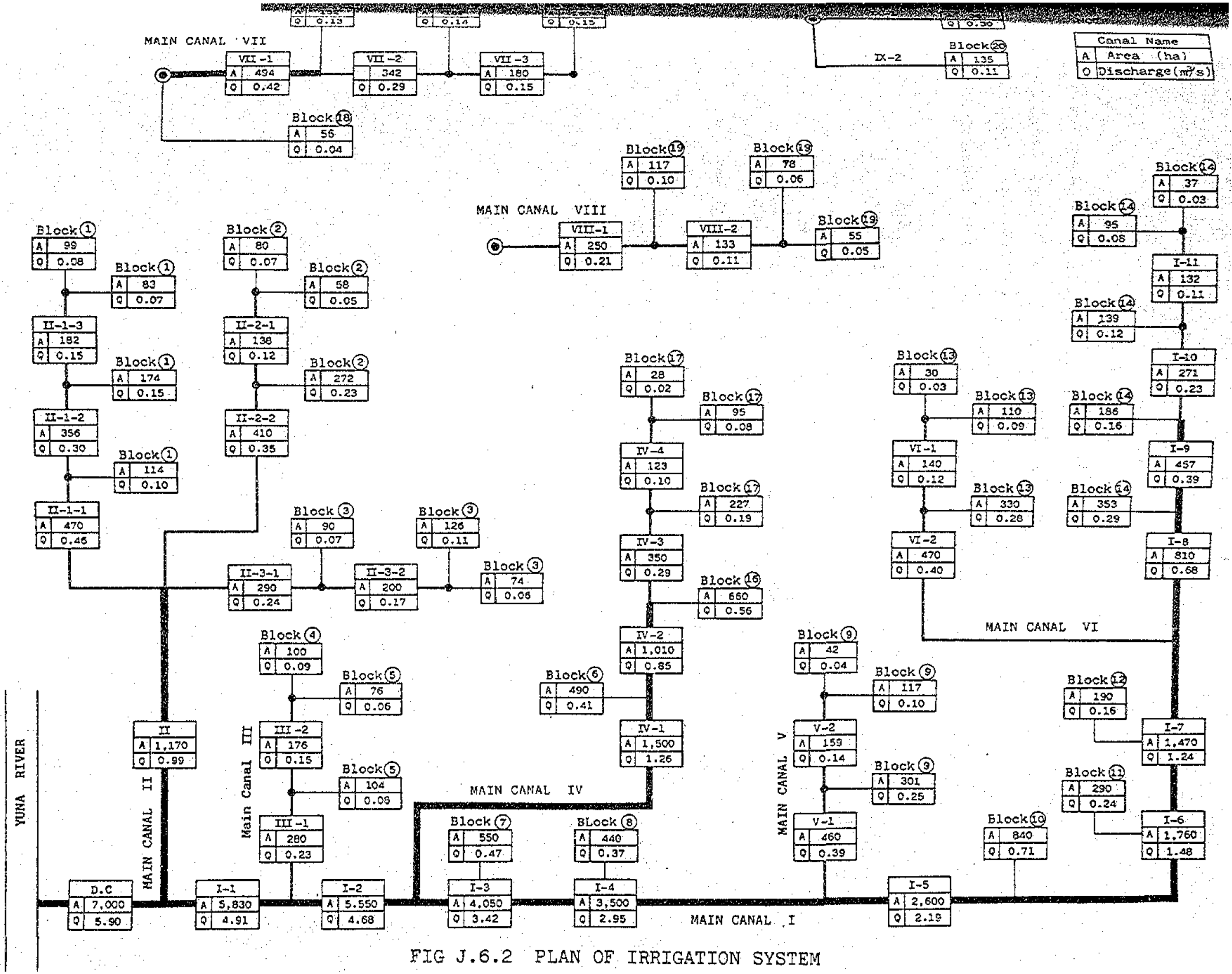


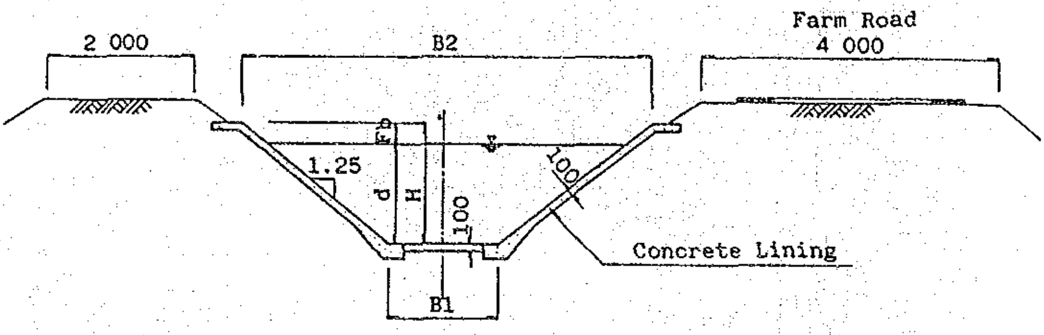
FIG J.6.2 PLAN OF IRRIGATION SYSTEM

7. Irrigation Canal

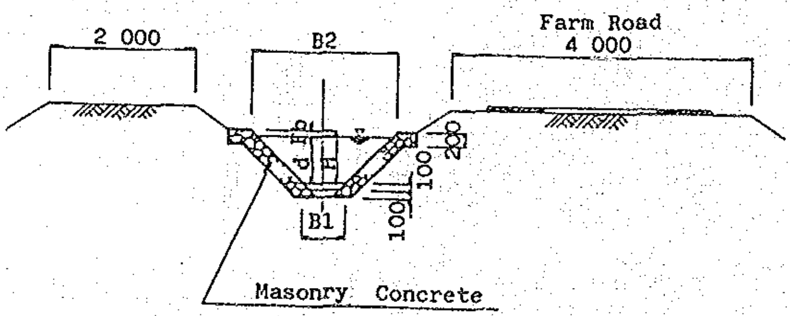
7.1 Main Irrigation Canal

7.1.1 Canal Section

In relation to the section of main irrigation canals, concrete lining structure (TYPE A) has been proposed for canals with design discharge more than 0.4 m^3/s and masonary lining structure (TYPE B) for less than 0.4 m^3/s. Length and dimension of each canal are presented in Table J.7.1.



TYPE - A



TYPE - B

TABLE J.7.1 DIMENSION OF IRRIGATION CANAL

Canal Type	B1 (m)	B2 (m)	H (m)	Identification of Canal	
A-1	2.00	7.00	2.00	Driving Canal	
A-2	1.80	6.55	1.90	Main Canal	I-1, I-2
A-3	1.60	5.85	1.70	"	I-3
A-4	1.50	5.50	1.60	"	I-4
A-5	1.40	5.15	1.50	"	I-5
A-6	1.00	4.25	1.30	"	I-6, I-7
A-7	1.00	3.75	1.10	"	II
A-8	0.80	3.55	1.10	"	IV-1
A-9	0.80	3.30	1.00	"	I-8, IV-2
A-10	0.80	3.05	0.90	"	VII-1
A-11	0.80	2.80	0.80	"	I-9
B-1	0.60	2.20	0.80	"	II-1-1, IV-3, VII-2, IX-1
B-2	0.60	2.00	0.70	"	I-10, II-2-1, II-3-1, V-1, VI-1, VIII-1
B-3	0.60	1.80	0.60	"	II-1-2, VII-3
B-4	0.50	1.70	0.60	"	I-ii, II-3-2, VIII-2, IX-2, III-2
B-5	0.50	1.50	0.50	"	II-1-3, III-1, IV-4, V-2, VI-2
B-6	0.40	1.40	0.50	"	II-2-2

7.1.2 Hydraulic Calculation

Manning formula is employed to calculate the canal velocity as follows:

$$V = \frac{1}{n} \cdot R^{2/3} \cdot I^{1/2} \quad (m/sec)$$

$$Q = A \cdot V \quad (m^3/sec)$$

where, V = mean velocity (m/sec)

n = coefficient of roughness
(Assumed to be 0.03 for earth canal)

R = hydraulic mean depth (m) = A/P

A = cross-sectional area of flow (m)

P = wetted perimeter (m)

I = hydraulic gradient (= bed slope)

Q = discharge (m^3/sec)

B = width of canal invert (m)

d = water depth (m)

hv = velocity head (m)

F.b = free board

Type A: Fb = 0.05d+hv+0.15 (m)

Type B: Fb = 0.05d+hv+0.05 (m)

H = Height of sidewall (m)

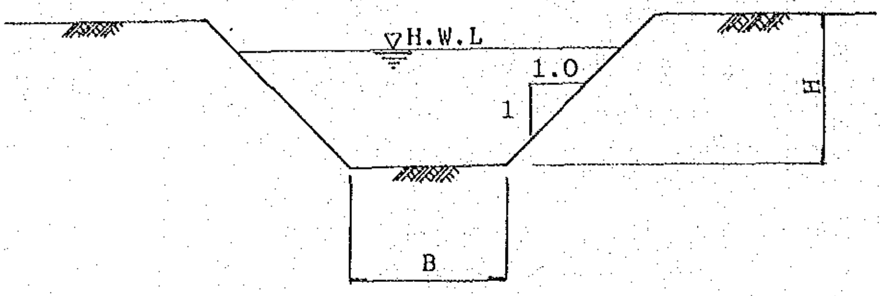
The result of this calculation is as presented in Table J.7.2.

TABLE J.7.2 RESULT OF HYDRAULIC CALCULATION OF MAIN CANAL

Canal Name	Canal Length (m)	Q (m3/s)	Canal Type	n	I	B1 (m)	d (m)	A (m2)	P (m)	R (m)	V (m/s)	hv (m)	Fb (m)	H (m)	B2 (m)	Remarks
Driving Canal	1,250 (700)	5.90	A	0.015	1/6,000	2.000	1.715	7.107	7.491	0.949	0.831	0.035	0.271	2.000	7.000	A-1
Main Canal I-1	1,250	4.91	"	"	"	1.800	1.619	6.191	6.983	0.887	0.794	0.032	0.263	1.900	6.550	A-2
" I-2	500	4.68	"	"	"	1.800	1.581	5.970	6.862	0.870	0.784	0.031	0.260	1.900	6.550	"
" I-3	5,950	3.42	"	"	"	1.600	1.406	4.721	6.101	0.774	0.725	0.027	0.247	1.700	5.850	A-3
" I-4	2,100	2.95	"	"	"	1.500	1.334	4.225	5.771	0.732	0.699	0.025	0.242	1.600	5.500	A-4
" I-5	4,100	2.19	"	"	"	1.400	1.178	3.384	5.171	0.654	0.649	0.021	0.230	1.500	5.150	A-5
" I-6	2,850	1.48	"	"	"	1.000	1.074	2.516	4.439	0.567	0.589	0.018	0.222	1.300	4.250	A-6
" I-7	2,650	1.24	"	"	"	1.000	0.986	2.201	4.157	0.530	0.563	0.016	0.215	1.300	4.250	"
" I-8	850	0.68	"	"	"	0.800	0.788	1.407	3.323	0.423	0.485	0.012	0.201	1.000	3.300	A-9
" I-9	1,000	0.39	"	"	"	0.800	0.600	0.930	2.721	0.342	0.421	0.009	0.189	0.800	2.800	A-11
" I-10	850	0.23	B	0.020	"	0.600	0.622	0.760	2.359	0.322	0.303	0.005	0.086	0.700	2.000	B-2
" I-11	800	0.11	"	"	"	0.500	0.458	0.439	1.795	0.244	0.252	0.003	0.076	0.600	1.700	B-4
II	2,900	0.99	A	0.015	1/6,000	1.000	0.884	1.861	3.830	0.486	0.532	0.014	0.208	1.100	3.750	A-7
II-1-1	2,000	0.46	B	0.020	1/1,500	0.600	0.622	0.760	2.359	0.322	0.607	0.019	0.101	0.800	2.200	B-1
II-1-2	1,100	0.30	"	"	"	0.600	0.502	0.553	2.020	0.274	0.544	0.015	0.090	0.600	1.800	B-3
II-1-3	2,150	0.15	"	"	"	0.500	0.376	0.329	1.563	0.211	0.457	0.011	0.080	0.500	1.500	B-5
II-2-1	2,150	0.35	"	"	1/1,200	0.600	0.512	0.569	2.048	0.278	0.615	0.019	0.095	0.700	2.000	B-2
II-2-2	1,350	0.12	"	"	"	0.400	0.344	0.256	1.373	0.186	0.471	0.011	0.078	0.500	1.400	B-6
II-3-1	1,550	0.24	"	"	1/3,000	0.600	0.534	0.606	2.110	0.287	0.397	0.008	0.085	0.700	2.000	B-2
II-3-2	1,000	0.17	"	"	"	0.500	0.478	0.467	1.852	0.252	0.365	0.007	0.081	0.600	1.700	B-4
III-1	750	0.23	"	"	1/600	0.500	0.370	0.322	1.547	0.208	0.717	0.026	0.095	0.500	1.500	B-5
III-2	1,700	0.15	"	"	1/3,000	0.500	0.448	0.425	1.767	0.241	0.353	0.006	0.078	0.600	1.700	B-4
IV-1	5,500	1.26	A	0.015	1/2,000	0.800	0.813	1.477	3.403	0.434	0.854	0.037	0.228	1.100	3.550	A-8
IV-2	3,300	0.85	"	"	1/3,000	0.800	0.742	1.282	3.176	0.404	0.665	0.023	0.210	1.000	3.300	A-9
IV-3	800	0.29	B	0.020	1/5,000	0.600	0.666	0.843	2.484	0.339	0.344	0.006	0.089	0.800	2.200	B-1
IV-4	1,500	0.10	"	"	"	0.500	0.416	0.381	1.677	0.227	0.263	0.004	0.075	0.500	1.500	B-5
V-1	1.900	0.39	"	"	1/1,500	0.600	0.573	0.672	2.221	0.303	0.582	0.017	0.096	0.700	2.000	B-2
V-2	750	0.14	"	"	"	0.500	0.363	0.313	1.527	0.205	0.449	0.010	0.078	0.500	1.500	B-5
VI-1	1,000	0.40	"	"	1/1,500	0.600	0.580	0.684	2.240	0.305	0.586	0.018	0.097	0.700	2.000	B-2
VI-2	1,000	0.12	"	"	"	0.500	0.334	0.279	1.445	0.193	0.431	0.009	0.076	0.500	1.500	B-5
VII-1	1,250	0.42	A	0.015	1/6,000	0.800	0.622	0.981	2.791	0.352	0.429	0.009	0.190	0.900	3.050	A-10
VII-2	800	0.29	B	0.020	"	0.600	0.697	0.904	2.571	0.352	0.322	0.005	0.089	0.800	2.200	B-1
VII-3	800	0.15	"	"	"	0.600	0.501	0.552	2.017	0.273	0.272	0.004	0.079	0.600	1.800	B-3
VIII-1	650	0.21	"	"	"	0.600	0.594	0.709	2.280	0.311	0.296	0.004	0.084	0.700	2.000	B-2
VIII-2	800	0.11	"	"	"	0.500	9.457	0.437	1.793	0.244	0.252	0.003	0.076	0.600	1.700	B-4
IX-1	1.100	0.30	"	"	"	0.600	0.708	0.926	2.603	0.356	0.324	0.005	0.090	0.800	2.200	B-1
IX-2	700	0.11	"	"	"	0.500	0.457	0.437	1.793	0.244	0.252	0.003	0.076	0.600	1.700	B-4

7.2 Secondary Canal

Secondary canals will be constructed in earth structure as illustrated in the figure below.



TYPICAL CROSS-SECTION

Given the hydraulic gradient to be at 1:5,000 and the coefficient of roughness at 0.03, the discharges for each type of canal are as summarized in table below:

TABLE J.7.3 HYDRAULIC CALCULATION OF SECONDARY CANAL

Type	Width of Opening Canal Bed (m)	Wdith of Opening Canal (m)	Depth of Water (m)	Height of Side Wall (m)	Discharge (m /s)	Cover Hectarage (ha)
A	0.60	1.80	0.50	0.60	0.109	129
B	0.50	1.50	0.40	0.50	0.062	74
C	0.40	1.20	0.30	0.40	0.030	36

And, the length of canal for each type is as calculated below:

(Unit: km)

	Alternative A	Alternative B
Type A	102.7	86.1
Type B	112.1	90.9
Type C	27.8	23.9
Total	242.6	200.9

8. Rural and In-farm Road Plan

Actually the following three rural roads are existing within the study area.

Route	Length (km)
Cruce de Rincon - Aguacate Pumping Station	11.8
Cruce de Ponton - Cruce Las Cabirmas	12.8
Arenoso - La Mata	30.3
Total	54.9

These roads (width : 8.0 m) are constructed with gravel finish and the maintenance is left in the charge of SEOPC/IAD. In the road network planning, the existing road will be left as they are except such improvement works as to cut-off of meandering part of the Cruce de Ponton - Cruce Las Cabrimas Route. A road network has been proposed connecting in-farm roads with rural roads (see Fig. J.8.1). The length of this proposed road network is summarized in Table J.8.1.

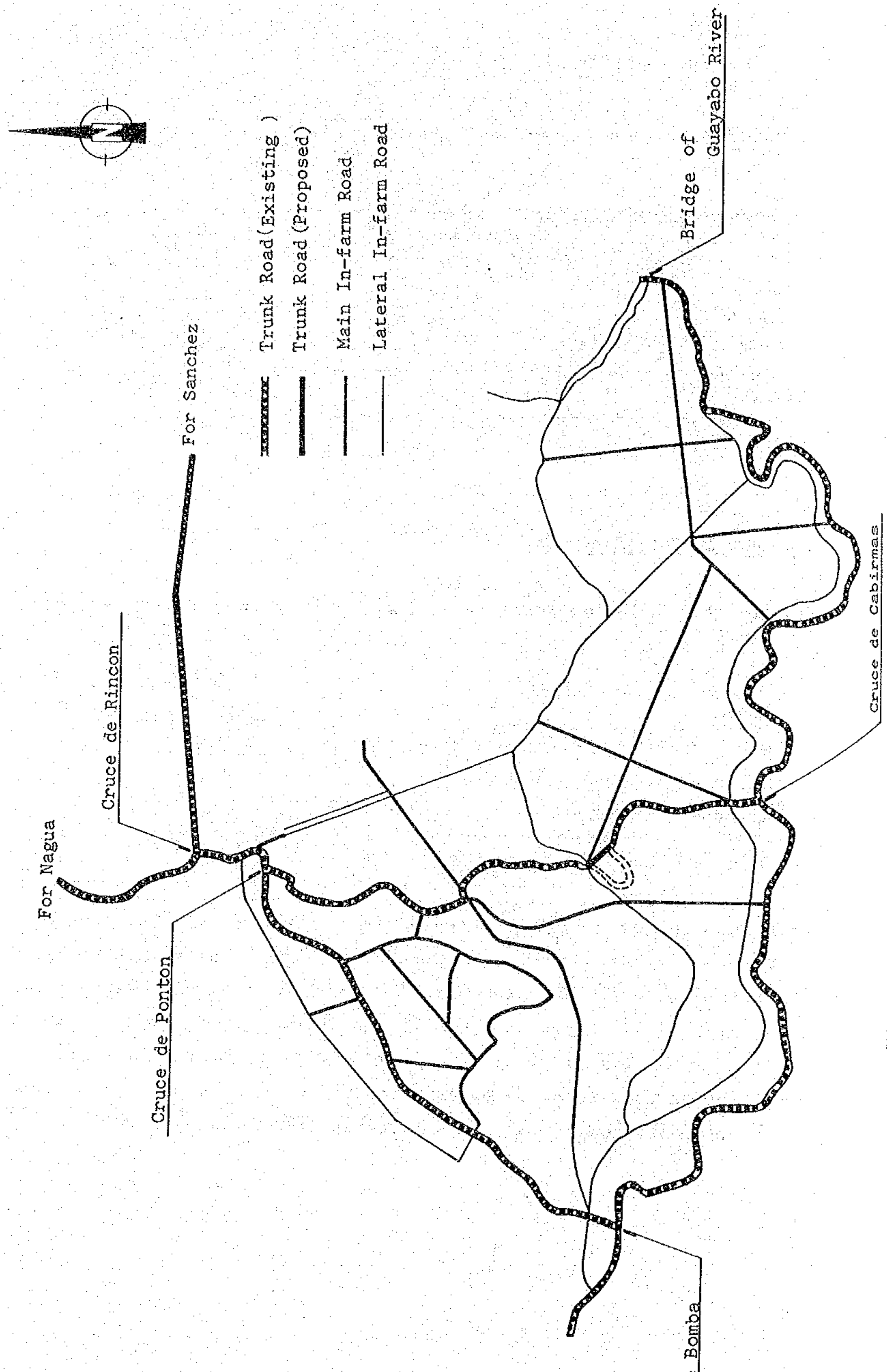

For Nagua
Cruce de Rincon
For Sanchez
Cruce de Ponton
Trunk Road(Existing)
Trunk Road(Proposed)
Main In-farm Road
Lateral In-farm Road
Bridge of
Guayabo River
Cruce de Cabirmas
Aguacate Bomba

Table J.8.1 ROAD NETWORK PLANNING

(Unit: m)

	WIDTH	LENGTH			
		Alternative A-1	Alternative A-2	Alternative B-1	Alternative B-2
1) Trunk Road	8.0 m				
To be Constructed		700	700	700	700
2) Trunk In-farm Road	6.0 m				
To be Constructed		18,400	18,400	18,400	18,400
To be Improved		28,300	28,300	25,500	25,500
Sub-total		46,700	46,700	43,900	43,900
3) Maintenance Road for Canal	4.0 m				
To be Constructed		64,250	63,700	56,850	56,300
To be Improved		3,600	3,600	1,800	1,800
Sub-total		67,850	67,300	58,650	58,100
4) Lateral In-farm Road	4.0 m				
To be Constructed		97,300	97,300	79,050	79,050

9. APPENDICES

APPENDIX J.2.1 MONTHLY MEAN TEMPERATURE AT BARRAQUITO

(℃)

Month / Year	Jan	Feb	Mar	Apl	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1975	23.0	23.4	24.2	26.1	26.7	28.1	27.5	27.2	26.5	26.0	24.8	22.3
1976	21.7	23.0	23.6	24.7	26.3	26.3	26.9	27.0	27.1	26.5	26.0	24.1
1977	23.5	24.4	25.0	27.5	27.5	27.7	27.1	26.8	27.0	26.5	25.6	24.4
1978	23.8	23.6	25.0	25.5	26.8	27.2	26.8	*	*	*	*	23.7
1979	23.0	23.6	23.8	25.0	26.0	26.9	27.4	26.8	26.9	*	25.1	23.3
1980	23.8	24.3	*	26.4	*	28.1	27.6	27.4	27.4	27.4	26.4	24.5
1981	23.6	24.0	25.0	24.9	26.5	27.0	27.2	27.0	27.7	27.0	25.8	24.3
1982	23.9	23.9	24.9	26.9	26.4	27.4	27.0	27.6	27.5	27.5	25.2	23.5
1983	24.5	24.6	26.3	26.4	27.2	28.3	27.6	27.5	27.4	26.9	26.1	*
1984	24.9	24.0	24.8	26.4	25.9	26.8	26.9	27.2	26.7	25.7	24.5	22.8
1985	22.9	23.2	23.7	24.9	*	*	*	*	*	*	*	*
Mean	23.5	23.8	24.6	25.6	26.5	27.4	27.2	27.2	27.1	26.7	25.5	23.7

APPENDIX J.2.2 MONTHLY MEAN RELATIVE HUMIDITY AT BARRAQUITO

(

Year \ Month	Jan	Feb	Mar	Apr	Mar	Jun	Jul	Aug	Sep	Oct	Nov	De
1868	-	-	-	-	-	-	79	82	81	78	86	90
1869	85	81	82	84	86	88	86	84	82	82	84	80
1970	78	80	75	71	78	79	79	80	81	80	78	79
1971	77	82	82	79	79	78	82	82	83	84	82	84
1972	89	86	-	85	79	83	82	84	82	82	86	88
1973	89	84	82	-	82	80	78	86	86	86	87	89
1974	89	86	83	82	81	83	86	88	86	88	89	92
1975	89	85	84	74	79	77	89	-	89	97	97	89
1976	92	97	90	82	82	84	84	84	84	86	92	88
1977	85	81	78	84	81	82	84	88	86	89	88	88
1978	84	85	85	85	84	85	86	-	-	-	-	86
1979	84	81	78	81	85	86	85	86	85	-	88	-
1980	85	81	-	80	-	93	85	86	85	85	83	86
1981	87	83	81	83	85	85	87	85	82	83	85	88
1982	88	86	78	75	83	85	85	83	87	82	85	85
1983	82	79	78	75	82	80	83	84	83	82	81	-
1984	81	87	89	73	-	82	81	81	87	85	85	83
1985	82	80	81	76	-	-	-	-	-	-	-	-
Mean	85.1	83.8	81.7	79.3	81.9	83.1	83.6	84.2	84.3	84.6	86.0	86

APPENDIX J.2.3 MONTHLY MEAN WIND VELOCITY AT BARRAQUITO

(m/s)

Month / Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1974	1.4	1.3	1.3	1.4	1.4	1.3	-	1.1	0.5	0.9	0.7	0.7
1975	1.2	1.4	1.2	2.0	1.5	2.1	2.0	1.7	1.3	1.1	1.1	1.1
1976	1.2	1.4	1.9	1.9	2.1	1.6	1.4	1.3	1.2	1.2	1.2	-
1977	-	-	-	-	-	-	-	-	1.3	1.0	1.1	1.1
1978	1.3	1.4	1.7	1.7	1.4	1.2	1.6	1.4	1.1	1.0	1.0	-
1979	-	-	-	-	-	-	-	-	-	-	-	-
1980	-	-	-	-	-	1.8	1.8	0.2	1.4	1.5	1.3	1.4
1981	1.6	1.8	2.3	2.3	1.6	1.0	0.2	1.6	1.7	1.3	1.2	1.2
1982	2.1	2.2	2.0	2.5	1.7	1.7	1.5	1.3	1.3	1.4	-	-
1983	-	-	-	-	-	-	-	-	-	-	-	-
1984	-	-	1.5	1.8	2.2	1.9	2.0	1.3	1.5	1.3	1.6	1.9
1985	1.4	1.4	1.4	1.5	-	-	-	-	-	-	-	-
Mean (m/sec)	1.5	1.6	1.7	1.9	1.7	1.6	1.5	1.2	1.3	1.2	1.2	1.2
" (km/day)	130	138	147	164	147	138	130	104	112	104	104	104

APPENDIX J.2.4 MONTHLY MEAN CLOUDINESS OKTAS AT BARRAQUITO

Year \ Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1968	-	-	-	-	-	-	5	6	6	6	6	6
1969	6	4	4	5	7	5	6	6	4	4	4	4
1970	3	3	4	4	4	4	3	3	3	3	3	3
1971	4	3	4	3	3	4	3	3	3	4	4	4
1972	4	4	-	4	4	4	4	4	4	4	4	4
1973	5	3	4	-	4	4	4	4	4	4	4	4
1974	3	4	4	-	4	3	4	4	4	4	5	4
1975	4	2	2	1	2	2	2	2	3	3	3	4
1976	2	3	3	3	3	4	4	3	3	4	4	4
1977	4	3	4	4	4	4	4	4	4	4	4	4
1978	4	4	4	4	4	4	4	4	4	4	4	3
1979	3	2	3	3	4	4	3	4	4	-	4	-
1980	3	3	-	3	-	3	3	3	3	3	3	3
1981	3	3	2	3	4	4	4	3	3	3	3	4
1982	3	3	2	4	3	4	4	4	4	2	2	-
1983	3	3	2	3	4	3	3	3	3	2	2	-
1984	3	3	2	2	3	3	3	3	3	3	3	3
1985	2	3	3	3	-	-	-	-	-	-	-	-
Mean	3.5	3.1	3.1	3.3	3.8	3.7	3.7	3.7	3.6	3.6	3.6	3.6
n/N	0.60	0.64	0.64	0.62	0.57	0.58	0.58	0.58	0.59	0.59	0.59	0.5[illegible]

APPENDIX J.2.5 SATURATION VAPOUR PRESSURE (ea) IN MBAR AS FUNCTION OF MEAN AIR TEMPERATURE (T) IN °C

Temperature °C	0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
ea mbar	6.1	6.6	7.1	7.6	8.1	8.7	9.3	10.0	10.7	11.5	12.3	13.1	14.0	15.0	16.1	17.0	18.2	19.4	20.6*	22.0

Temperature °C	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39
ea mbar	23.4	24.9	26.4	28.1	29.8	31.7	33.6	35.7	37.8*	40.1	42.4	44.9	47.6	50.3	53.2	56.2	59.4	62.8	66.3	69.9

1/ Also actual vapour pressure (ed) can be obtained from this table using available Tdewpoint data.
(Example: Tdewpoint is 18°C; ed is 20.6 mbar)

APPENDIX J.2.6 VAPOUR PRESSURE (ed) IN MBAR FROM DRY AND WET BULB TEMPERATURE DATA IN °C

Depression wet bulb T°C altitude 0-1 000 m												drybulb T°C	Depression wet bulb T°C altitude 1 000-2 000 m											
0	2	4	6	8	10	12	14	16	18	20	22		0	2	4	6	8	10	12	14	16	18	20	22
73.8	64.9	56.8	49.2	42.2	35.8	29.8	24.3	19.2	14.4	10.1	6.0	40	73.8	65.2	57.1	49.8	43.0	41.8	31.0	25.6	20.7	16.2	12.0	8.1
66.3	58.1	50.5	43.6	37.1	31.1	25.6	20.5	15.8	11.4	7.3		38	66.3	58.2	50.9	44.1	37.9	36.7	26.8	21.8	17.3	13.2	9.2	5.7
59.4	51.9	44.9	38.4	32.5	26.9	21.8	17.1	12.7	8.6	4.9		36	59.4	52.1	45.2	39.0	33.3	32.1	23.0	18.4	14.3	10.4	6.8	3.5
53.2	46.2	39.8	33.8	28.3	23.2	18.4	14.0	10.0	6.2			34	53.2	46.4	40.1	34.4	29.1	24.1	19.6	15.4	11.5	8.0	4.6	1.5
47.5	41.1	35.1	29.6	24.5	19.8	15.4	11.3	7.5	4.0			32	47.5	41.3	35.5	30.2	25.3	20.7	16.6	12.6	9.1	5.8	2.6	
42.4	36.5	30.9	25.8	21.1	16.7	12.6	8.8	5.3				30	42.4	36.7	31.3	26.4	21.9	17.7	13.8	10.2	6.9	3.8	0.9	
37.8	32.3	27.2	22.4	18.0	14.0	10.2	6.7	3.4				28	37.8	32.5	27.5	23.0	18.9	14.9	11.4	8.0	4.9	2.1		
33.6	28.5	23.8	19.4	15.3	11.5	8.0	4.7	1.6				26	33.6	28.7	24.1	20.0	16.1	12.5	9.2	6.0	3.2	0.5		
29.8	25.1	20.7*	16.6	12.8	9.3	6.0	2.9					24	29.8	25.3	21.1	17.2	13.9	10.3	7.2	4.3	1.6			
26.4	22.0	18.0	14.2	10.6	7.4	4.3	1.4					22	26.4	22.3	18.3	14.3	11.5	8.3	5.5	2.7	0.2			
23.4	19.3	15.5	12.0	8.7	5.6	2.7						20	23.4	19.5	15.9	12.6	9.5	6.6	3.9	1.3				
20.6	16.8	13.3	10.0	6.9	4.1	1.4						18	20.6	17.1	13.7	10.6	7.8	5.0	2.5	0.1				
18.2	14.6	11.4	8.3	5.4	2.7							16	18.2	14.9	11.7	8.9	6.2	3.6	1.3					
16.0	12.7	9.6	6.7	4.0	1.5							14	16.0	12.9	10.0	7.3	4.8	2.4	0.3					
14.0	10.9	8.1	5.3	2.8								12	14.0	11.2	8.4	5.9	3.6	1.4						
12.3	9.4	6.7	4.1	1.7								10	12.3	9.6	7.0	4.7	2.6	0.4						
10.7	8.0	5.5	3.1	0.8								8	10.7	8.2	5.8	3.7	1.6							
9.3	6.8	4.4	2.1									6	9.3	7.0	4.8	2.7	0.7							
8.1	5.7	3.4	1.6									4	8.1	6.0	3.8	1.8								
7.1	4.8	2.8	0.8									2	7.1	5.0	2.9	1.0								
6.1	4.0	2.0										0	6.1	4.1	2.1									

APPENDIX J.2.7 VALUES OF WEIGHTING FACTOR (1-w) FOR EFFECT OF WIND AND HUMIDITY ON ETO AT DIFFERENT TEMPERATURES AND ALTITUTDES

Temperature °C	2	4	6	8	10	12	14	16	18	20	22	24	26	28	30	32	34	36	38	40
(1-W) at altitude m																				
0	0.57	.54	.51	.48	.45	.42	.39	.36	.34	.32	.29	.27	.25	.23*	.22	.20	.19	.17	.16	.15
500	.56	.52	.49	.46	.43	.40	.38	.35	.33	.30	.28	.26	.24	.22	.21	.19	.18	.16	.15	.14
1 000	.54	.51	.48	.45	.42	.39	.36	.34	.31	.29	.27	.25	.23	.21	.20	.18	.17	.15	.14	.13
2 000	.51	.48	.45	.42	.39	.36	.34	.31	.29	.27	.25	.23	.21	.19	.18	.16	.15	.14	.13	.12
3 000	.48	.45	.42	.39	.36	.34	.31	.29	.27	.25	.23	.21	.19	.18	.16	.15	.14	.13	.12	.11
4 000	.46	.42	.39	.36	.34	.31	.29	.27	.25	.23	.21	.19	.18	.16	.15	.14	.13	.12	.11	.10

APPENDIX J.2.8 VALUES OF WEIGHTING FACTOR (W) FOR THE EFFECT OF RADIATION ON ETO AT DIFFERENT TEMERATURE AND ALTITUDES

Temperature °C	2	4	6	8	10	12	14	16	18	20	22	24	26	28	30	32	34	36	38	40
W at altitude m																				
0	0.43	.46	.49	.52	.55	.58	.61	.64	.66	.69	.71	.73	.75	.77*	.78	.80	.82	.83	.84	.85
500	.44	.48	.51	.54	.57	.60	.62	.65	.67	.70	.72	.74	.76	.78	.79	.81	.82	.84	.85	.86
1 000	.46	.49	.52	.55	.58	.61	.64	.66	.69	.71	.73	.75	.77	.79	.80	.82	.83	.85	.86	.87
2 000	.49	.52	.55	.58	.61	.64	.66	.69	.71	.73	.75	.77	.79	.81	.82	.84	.85	.86	.87	.88
3 000	.52	.55	.58	.61	.64	.66	.69	.71	.73	.75	.77	.79	.81	.82	.84	.85	.86	.87	.88	.89
4 000	.54	.58	.61	.64	.66	.69	.71	.73	.75	.77	.79	.81	.82	.84	.85	.86	.87	.89	.90	.90

APPENDIX J.2.9 EXTRA TERRESTRIAL RADIATION (Ra) EXPRESSED IN mm/day

Northern Hemisphere													Southern Hemisphere											
Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec	Lat	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec
3.8	6.1	9.4	12.7	15.8	17.1	16.4	14.1	10.9	7.4	4.5	3.2	50°	17.5	14.7	10.9	7.0	4.2	3.1	3.5	5.5	8.9	12.9	16.5	18.2
4.3	6.6	9.8	13.0	15.9	17.2	16.5	14.3	11.2	7.8	5.0	3.7	48	17.6	14.9	11.2	7.5	4.7	3.5	4.0	6.0	9.3	13.2	16.6	18.2
4.9	7.1	10.2	13.3	16.0	17.2	16.6	14.5	11.5	8.3	5.5	4.3	46	17.7	15.1	11.5	7.9	5.2	4.0	4.4	6.5	9.7	13.4	16.7	18.3
5.3	7.6	10.6	13.7	16.1	17.2	16.6	14.7	11.9	8.7	6.0	4.7	44	17.8	15.3	11.9	8.4	5.7	4.4	4.9	6.9	10.2	13.7	16.7	18.3
5.9	8.1	11.0	14.0	16.2	17.3	16.7	15.0	12.2	9.1	6.5	5.2	42	17.8	15.5	12.2	8.8	6.1	4.9	5.4	7.4	10.6	14.0	16.8	18.3
6.4	8.6	11.4	14.3	16.4	17.3	16.7	15.2	12.5	9.6	7.0	5.7	40	17.9	15.7	12.5	9.2	6.6	5.3	5.9	7.9	11.0	14.2	16.9	18.3
6.9	9.0	11.8	14.5	16.4	17.2	16.7	15.3	12.8	10.0	7.5	6.1	38	17.9	15.8	12.8	9.6	7.1	5.8	6.3	8.3	11.4	14.4	17.0	18.3
7.4	9.4	12.1	14.7	16.4	17.2	16.7	15.4	13.1	10.6	8.0	6.6	36	17.9	16.0	13.2	10.1	7.5	6.3	6.8	8.8	11.7	14.6	17.0	18.2
7.9	9.8	12.4	14.8	16.5	17.1	16.8	15.5	13.4	10.8	8.5	7.2	34	17.8	16.1	13.5	10.5	8.0	6.8	7.2	9.2	12.0	14.9	17.1	18.2
8.3	10.2	12.8	15.0	16.5	17.0	16.8	15.6	13.6	11.2	9.0	7.8	32	17.8	16.2	13.8	10.9	8.5	7.3	7.7	9.6	12.4	15.1	17.2	18.1
8.8	10.7	13.1	15.2	16.5	17.0	16.8*	15.7	13.9	11.6	9.5	8.3	30	17.8	16.4	14.0	11.3	8.9	7.8	8.1	10.1	12.7	15.3	17.3	18.1
9.3	11.1	13.4	15.3	16.5	16.8	16.7	15.7	14.1	12.0	9.9	8.8	28	17.7	16.4	14.3	11.6	9.3	8.2	8.6	10.4	13.0	15.4	17.2	17.9
9.8	11.5	13.7	15.3	16.4	16.7	16.6	15.7	14.3	12.3	10.3	9.3	26	17.6	16.4	14.4	12.0	9.7	8.7	9.1	10.9	13.2	15.5	17.2	17.8
10.2	11.9	13.9	15.4	16.4	16.6	16.5	15.8	14.5	12.6	10.7	9.7	24	17.5	16.5	14.6	12.3	10.2	9.1	9.5	11.2	13.4	15.6	17.1	17.7
10.7	12.3	14.2	15.5	16.3	16.4	16.4	15.8	14.6	13.0	11.1	10.2	22	17.4	16.5	14.8	12.6	10.6	9.6	10.0	11.6	13.7	15.7	17.0	17.5
11.2	12.7	14.4	15.6	16.3	16.4	16.3	15.9	14.8	13.3	11.6	10.7	20	17.3	16.5	15.0	13.0	11.0	10.0	10.4	12.0	13.9	15.8	17.0	17.4
11.6	13.0	14.6	15.6	16.1	16.1	16.1	15.8	14.9	13.6	12.0	11.1	18	17.1	16.5	15.1	13.2	11.4	10.4	10.8	12.3	14.1	15.8	16.8	17.1
12.0	13.3	14.7	15.6	16.0	15.9	15.9	15.7	15.0	13.9	12.4	11.6	16	16.9	16.4	15.2	13.5	11.7	10.8	11.2	12.6	14.3	15.8	16.7	16.8
12.4	13.6	14.9	15.7	15.8	15.7	15.7	15.7	15.1	14.1	12.8	12.0	14	16.7	16.4	15.3	13.7	12.1	11.2	11.6	12.9	14.5	15.8	16.5	16.6
12.8	13.9	15.1	15.7	15.7	15.5	15.5	15.6	15.2	14.4	13.3	12.5	12	16.6	16.3	15.4	14.0	12.5	11.6	12.0	13.2	14.7	15.8	16.4	16.5
13.2	14.2	15.3	15.7	15.5	15.3	15.3	15.5	15.3	14.7	13.6	12.9	10	16.4	16.3	15.5	14.2	12.8	12.0	12.4	13.5	14.8	15.9	16.2	16.2
13.6	14.5	15.3	15.6	15.3	15.0	15.1	15.4	15.3	14.8	13.9	13.3	8	16.1	16.1	15.5	14.4	13.1	12.4	12.7	13.7	14.9	15.8	16.0	16.0
13.9	14.8	15.4	15.4	15.1	14.7	14.9	15.2	15.3	15.0	14.2	13.7	6	15.8	16.0	15.6	14.7	13.4	12.8	13.1	14.0	15.0	15.7	15.8	15.7
14.3	15.0	15.5	15.5	14.9	14.4	14.6	15.1	15.3	15.1	14.5	14.1	4	15.5	15.8	15.6	14.9	13.8	13.2	13.4	14.3	15.1	15.6	15.5	15.4
14.7	15.3	15.6	15.3	14.6	14.2	14.3	14.9	15.3	15.3	14.8	14.4	2	15.3	15.7	15.7	15.1	14.1	13.5	13.7	14.5	15.2	15.5	15.3	15.1
15.0	15.5	15.7	15.3	14.4	13.9	14.1	14.8	15.3	15.4	15.1	14.8	0	15.0	15.5	15.7	15.3	14.4	13.9	14.1	14.8	15.3	15.4	15.1	14.8

APPENDIX J.2.10 CONVERSION FACTOR FOR EXTRA-TERRESTRIAL RADIATION (Ra) TO NET SOLAR RADIATION (Rns) FOR A GIVEN REFLECTION OF 0.25 AND DIFFERENT RATIONS OF ACTUAL TO MAXIMUM SUNSHINE HOURS $(1-\alpha)(0.25 + 0.50n/N)$

n/N	0.0	.05	.10	.15	.20	.25	.30	.35	.40	.45	.50	.55	.60	.65	.70	.75	.80	.85	.90	.95	1.0
$(1-\alpha)(0.25 + 0.50n/N)$	0.19	.21	.22	.24	.26	.28	.30	.32	.34	.36	.37	.39	.41	.43	.45	.47	.49*	.51	.52	.54	.56

APPENDIX J.2.11 EFFECT OF TEMPERATURE f(T) ON LONGWAVE RADIATION (Rnl)

T°C	0	2	4	6	8	10	12	14	16	18	20	22	24	26	28	30	32	34	36
$f(T) = \sigma Tk^4$	11.0	11.4	11.7	12.0	12.4	12.7	13.1	13.5	13.8	14.2	14.6	15.0	15.4	15.9	16.3*	16.7	17.2	17.7	18.1

APPENDIX J.2.12 EFFECT VAPOUR PRESSURE f(ed) ON LONGWAVE RADIATION (Rnl)

ed mbar	6	8	10	12	14	16	18	20	22	24	26	28	30	32	34	36	38	40
$f(ed) = 0.34 - 0.044\sqrt{ed}$	0.23	.22	.20	.19	.18	.16	.15	.14	.13*	.12	.12	.11	.10	.09	.08	.08	.07	.06

APPENDIX J.2.13 EFFECT OF THE RATIO ACTUAL AND MAXIMUM BRIGHT SUNSHINE HOURS f(n/N) ON LONGWAVE RADIATION (Rnl)

n/N	0	.05	.1	.15	.2	.25	.3	.35	.4	.45	.5	.55	.6	.65	.7	.75	.8	.85	.9	.95	1.0
$f(n/N) = 0.1 + 0.9\, n/N$	0.10	.15	.19	.24	.28	.33	.37	.42	.46	.51	.55	.60	.64	.69	.73	.78	.82*	.87	.91	.96	1.0

APPENDIX J.2.14 ADJUSTMENT FACTOR (c) IN PRESENTED PENMAN EQUATION

	RHmax = 30%				RHmax = 60%				RHmax = 90%			
Rs mm/day	3	6	9	12	3	6	9	12	3	6	9	12
Uday m/sec	Uday/Unight = 4.0											
0	.86	.90	1.00	1.00	.96	.98	1.05	1.05	1.02	1.06	1.10	1.10
3	.79	.84	.92	.97	.92	1.00	1.11	1.19	.99	1.10	1.27	1.32
6	.68	.77	.87	.93	.85	.96	1.11	1.19	.94	1.10	1.26	1.33
9	.55	.65	.78	.90	.76	.88	1.02	1.14	.88	1.01	1.16	1.27
	Uday/Unight = 3.0											
0	.86	.90	1.00	1.00	.96	.98	1.05	1.05	1.02	1.06	1.10	1.10
3	.76	.81	.88	.94	.87	.96	1.06	1.12	.94	1.04	1.18	1.28
6	.61	.68	.81	.88	.77	.88	1.02	1.10	.86	1.01	1.15	1.22
9	.46	.56	.72	.82	.67	.79	.88	1.05	.78	.92	1.06	1.18
	Uday/Unight = 2.0											
0	.86	.90	1.00	1.00	.96	.98	1.05	1.05	1.02	1.06	1.10	1.10
3	.69	.76	.85	.92	.83	.91	.99*	1.05*	.89	.98	1.10*	1.14*
6	.53	.61	.74	.84	.70	.80	.94	1.02	.79	.92	1.05	1.12
9	.37	.48	.65	.76	.59	.70	.84	.95	.71	.81	.96	1.06
	Uday/Unight = 1.0											
0	.86	.90	1.00	1.00	.96	.98	1.05	1.05	1.02	1.06	1.10	1.10
3	.64	.71	.82	.89	.78	.86	.94*	.99*	.85	.92	1.01*	1.05*
6	.43	.53	.68	.79	.62	.70	.84	.93	.72	.82	.95	1.00
9	.27	.41	.59	.70	.50	.60	.75	.87	.62	.72	.87	.96

APPENDIX J.2.15 MONTHLY RAINFALL AT BARRAQUITO (mm)

	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANUAL
1968							102.8	122.3	103.6	52.9	435.0	300.2	1,116.8
1969	84.6	19.9	18.0	151.0	363.4	250.5	126.8	104.8	107.7	184.1	72.0	81.2	1,564.6
1970	40.9	159.9	57.7	41.4	279.8	160.4	161.9	351.8	180.3	551.8	116.4	256.9	2,359.2
1971	64.2	210.2	84.3	234.2	114.9	43.1	213.5	161.0	112.7	81.8	78.7	120.8	1,519.4
1972	79.1	171.3	178.7	129.7	175.3	214.5	186.3	258.6	198.8	182.3	105.7	137.3	2,017.6
1973	60.2	127.7	210.8	77.4	11.8	171.8	154.8	218.5	157.7	155.3	131.7	196.8	1,674.2
1974	163.2	212.6	85.7	142.3	189.1	234.2	141.9	246.6	173.6	235.4	85.8	205.2	2,115.6
1975	17.4	22.3	89.5	9.0	256.4	49.6	103.4	61.0	166.1	175.5	374.2	237.7	1,562.1
1976	31.4	167.1	124.4	229.3	165.9	310.8	138.9	162.3	64.6	248.6	129.9	79.0	1,752.2
1977	25.8	19.9	40.1	371.1	206.5	72.5	291.9	317.1	101.6	176.0	381.1	289.3	2,292.9
1978	121.6	71.2	172.1	201.6	281.8	145.1	191.3	176.4	219.3	116.9	154.4	86.9	1,938.6
1979	92.8	48.2	116.2	254.7	654.2	244.3	189.7	491.9	190.1	251.6	350.9	211.6	3,096.2
1980	91.7	85.0	60.4	121.5	423.8	189.8	218.4	300.1	104.9	158.6	40.0	220.6	2,014.8
1981	189.2	255.3	150.7	180.6	348.6	133.4	269.5	220.8	107.4	159.8	117.1	219.8	2,352.2
1982	96.9	137.8	29.8	42.7	409.9	310.1	177.7	160.6	157.8	99.5	230.1	139.4	1,992.3
1983	66.3	51.3	32.1	101.8	343.9	264.9	289.1	243.8	169.4	195.3	220.3	113.1	2,141.1
1984	100.1	163.1	91.3	82.8	275.0	386.0	147.2	59.7	113.6	161.6	221.2	34.0	1,886.1
1985	55.6	98.4	164.8	59.4									378.2
MEAN	81.2	118.9	103.3	143.0	281.3	192.5	182.7	215.1	142.9	187.5	190.9	177.0	2,016.3

	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANUAL
1969	67.7	15.8	14.4	11.2	271.1	200.4	101.4	83.8	86.2	147.4	57.6	65.0	1,232.0
1970	34.7	127.9	46.2	33.1	201.0	128.3	129.5	233.5	121.4	159.2	93.1	129.5	1,437.4
1971	51.8	168.2	68.2	187.4	91.9	34.5	170.8	128.8	90.2	65.4	63.0	96.9	1,216.8
1972	63.3	137.0	143.0	103.8	140.2	170.9	149.0	188.6	159.0	145.8	84.6	109.8	1,595.0
1973	48.2	102.2	159.4	31.9	9.4	137.2	123.8	174.8	126.2	124.2	105.4	157.4	1,330.1
1974	130.9	153.5	39.2	113.0	151.3	187.4	115.1	197.3	138.9	188.1	68.6	164.2	1,648.3
1975	13.9	17.8	71.6	7.2	195.4	39.7	82.7	48.8	132.9	140.4	215.7	74.6	1,040.7
1976	25.1	133.7	99.5	166.2	121.0	168.6	11.1	129.8	5.7	198.9	103.9	63.2	1,372.7
1977	20.6	15.9	32.0	198.5	165.2	58.0	228.4	210.9	81.3	140.8	205.1	105.2	1,461.9
1978	97.3	57.8	137.7	161.3	224.5	116.1	153.0	141.0	174.2	93.5	123.5	69.5	1,548.7
1979	74.2	38.6	93.0	161.2	231.2	175.0	141.7	232.4	110.9	185.8	136.4	89.0	1,669.4
1980	73.4	68.0	48.3	97.2	264.1	151.8	174.7	239.7	83.9	126.9	32.0	176.5	1,536.5
1981	151.4	118.3	120.6	144.5	266.5	105.1	215.6	176.6	86.2	127.8	111.9	163.5	1,787.9
1982	77.5	110.2	23.8	34.2	258.0	165.0	142.2	128.5	126.2	79.6	182.4	107.2	1434.8
1983	53.6	41.0	65.7	81.4	220.4	197.9	231.3	195.0	135.5	156.5	121.9	90.2	1,589.6
2YEAR P.E.R.	56.7	75.8	67.8	103.6	156.3	120.3	145.0	161.4	111.6	133.0	104.3	104.9	1,446.9
5YEAR P.E.R.	32.4	37.6	37.3	57.0	77.0	74.3	113.0	114.8	85.0	102.1	67.7	78.0	1,283.6
10YEAR P.E.R.	23.4	22.7	25.5	36.7	53.2	57.7	98.7	93.7	72.2	89.0	52.8	66.8	1,205.7

ANNEX K: DRAINAGE PLAN

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ANNEX K: DRAINAGE PLAN

1. Basic Consideration

Basic consideration on the drainage plan is the timely removal of surplus water from the land so that damage to crops is minimized. This surface drainage will be made through actual systems of the Yuna River and the Caño Gran Estero until it will discharge to the Samana and Escocesa Bays.

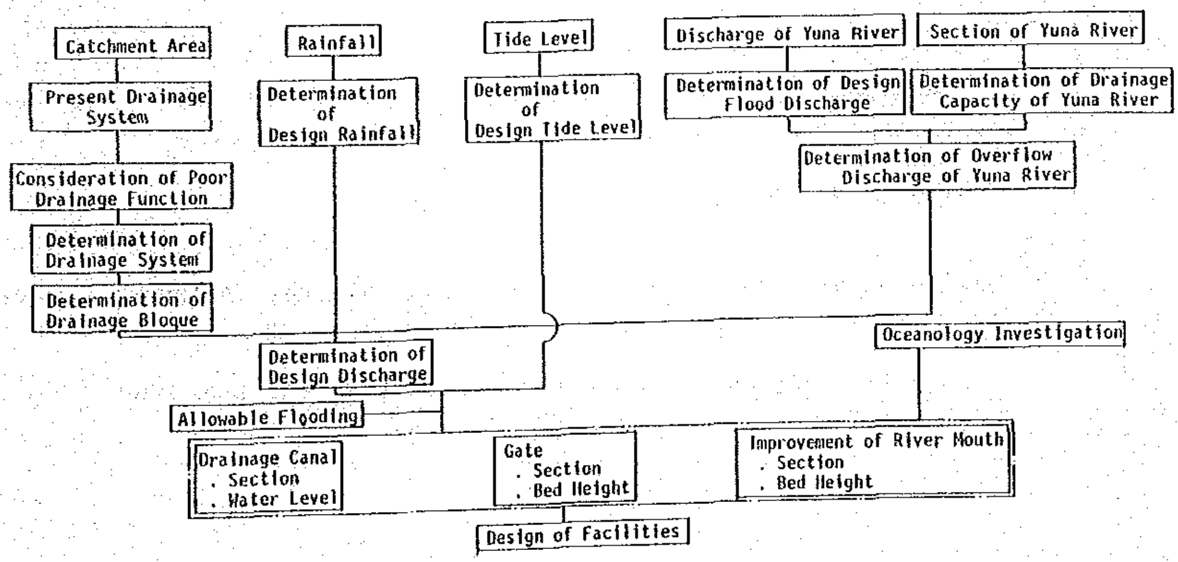
The improvement of present pour drainage constitutes the principal objective of the plan which has been designed to remove excess rainfall produced by a 1 : 5 year return period.

In view that the development plan does not aim at Flood Mitigation of the Yuna River but at the Agricultural Development of the study area, the improvement works of the Yuna River has not been considered.

The drainage plan covers only the proposed development area and the rest of area will be left without any improvement works.

2. Planning Methodology

The establishment of the drainage plan has been made in line with the following flow chart.



3. Design Criteria

3.1 Project Level

(1) Design Drainage Discharge

The design drainage discharge was established for 5 years return period which is currently employed by the INDRHI.

(2) Design Daily Rainfall

Design daily rainfall to determine the drain section was established based on the maximum daily rainfall for 5 years return period; daily rainfall is expected to be removed within the same day. For the design of tide gate, three days duration of rainfall was employed.

(3) Allowable Flooding Depth and Duration

Allowable flooding depth and duration are as follows:

Allowable flooding depth	:	30 cm
Allowable flooding duration	:	24 hours

(4) Design Inner Water Level

Design inner water level at the flooding was determined as follows:

Design inner water level =
Minimum elevation of the paddy field + allowable flooding depth

(5) Design Outer Water Level

Design outer water levels were determined in the following manner:

Cano Gran Estero System: design high tide level of the Escocesa Bay

Guayabo River System : mean water level at the confluence of the Yuna River and the Guayabo River

(6) Inflow from Outside the Development Area

The proposed inflow of excess water from outside the development area was estimated to be equivalent to the overflow which was determined by the flood discharge of the Yuna River for 1 : 5 return period and the draining capacity of the Yuna River.

3.2 Design Daily Rainfall

(1) Design Daily Rainfall

Maximum consecutive rainfall for 5 years return period at different pluvial stations are as follows:

Continuous maximum rainfall for 5 years return period (mm)

Consecutive Rainfall	Sanchez	Nagua	Villa Riva	Barraquite	Study Area
Maximum Daily	151.3	164.9	124.8	136.7	141.6
Maximum Two Days	206.3	201.6	161.8	169.2	183.7
Maximum Three Days	229.7	229.2	173.1	190.1	204.5

Consequently, design daily rainfalls for drainage plan were established as follows:

Daily : 141.6 mm
Two consecutive days : 183.7 mm
Three consecutive days : 204.5 mm

(2) Rainfall Pattern

Rear mountain type rainfall pattern was employed for the sake of safety.

(3) Distribution of Rainfall

Distribution of rainfall are in the followings:

First day	:	20.8 mm
Second day	:	42.1 mm
Third day	:	141.6 mm

(4) Hourly Distribution of Rainfall

Hourly distribution of rainfall was computed using following formula:

$$R_t = R_{24} \left(\frac{t}{24}\right)^K$$

where, R_t : Total Rainfall on T hours

R_{24} : Daily Rainfall

K : Coefficient (employed 0.5 which was estimated by Sharman)

3.3 Design Outer Water Level

3.3.1 Tide Level

Design outer water level applied for the Caño Gran Estero System was determined based on data obtained from the Playa El Diamante Station.

HWL = +0.40 m

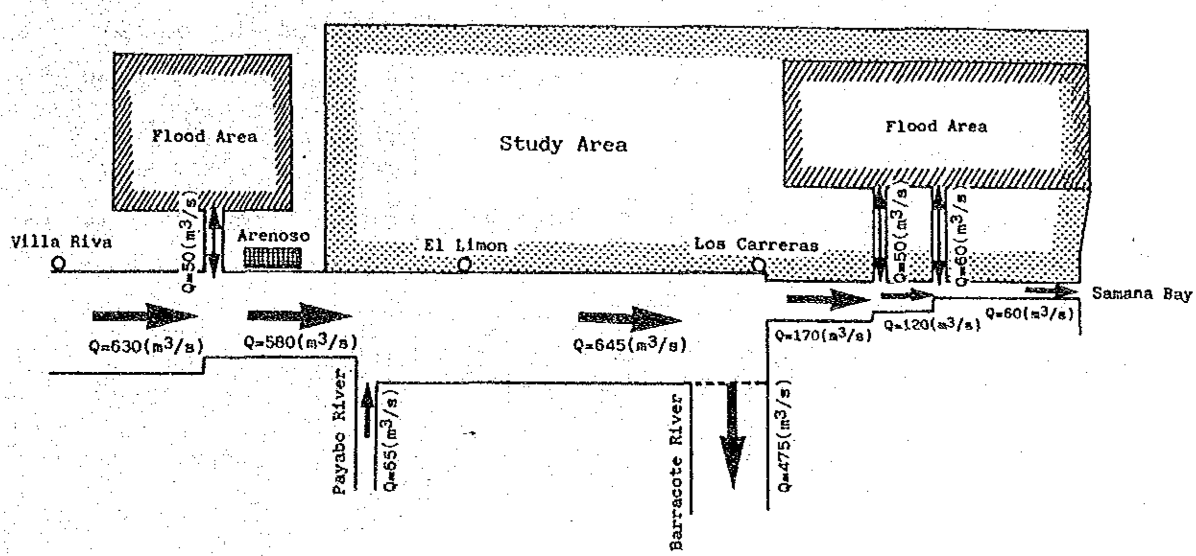
3.3.2 Water Level of the Yuna River

Design outer water level for the Guayabo River System was determined by present water level at the confluence of the Yuna River with the Guayabo River.

In the course of the field works, water level of at the said confluence was observed, which is summarized as below:

Mean Water Level ... +0.89 m
High Water Level ... +1.46 m
Low Water Level ... +0.58 m
(Oct. 1985 - Dec. 1985)

Flood discharge of the Yuna River for 5 years return period is illustrated as follows:



4. Drainage Plan

4.1 General Description of the Plan

For the purpose of improving actual poor drainage system, a drainage plan has been formulated, which is composed of:

- To construct a main drainage canal, which will collect excess water of development area to drain into the Escocesa Bay through the Caño Gran Estero.

- To improve and cut off the Caño Gran Estero so that its draining capacity may be improved. The construction of a tide gate and a training dike is also considered so as to prevent the outlet of the Caño Gran Estero from being closed with the accumulation of drifting sand.

- To improve the Guayabo River together with the installation of a drainage gate at the confluence with the Yuna River.

4.2 Description of Structures

The following structures are included in the drainage plans.

Structures	Alternative-A	Alternative-B
Main Drainage Canal	44.3 km	44.3 km
Secondary Drainage Canal	28.2 km	19.2 km
Drainage Gate	2	1
Tide Gate	1	-
Training Dike	1	1

4.3 Drainage Canal

4.3.1 Canal Network

Drainage canal network, as illustrated in Fig. K.4.1, has been delineated in accordance with following considerations.

- Actual rivers and drains will be used as far as possible and these systems will constitute the trunk of the network.

4.3.2 Division of the Drainage Basin

The division of the drainage basin is as shown in Fig. K.4.2.

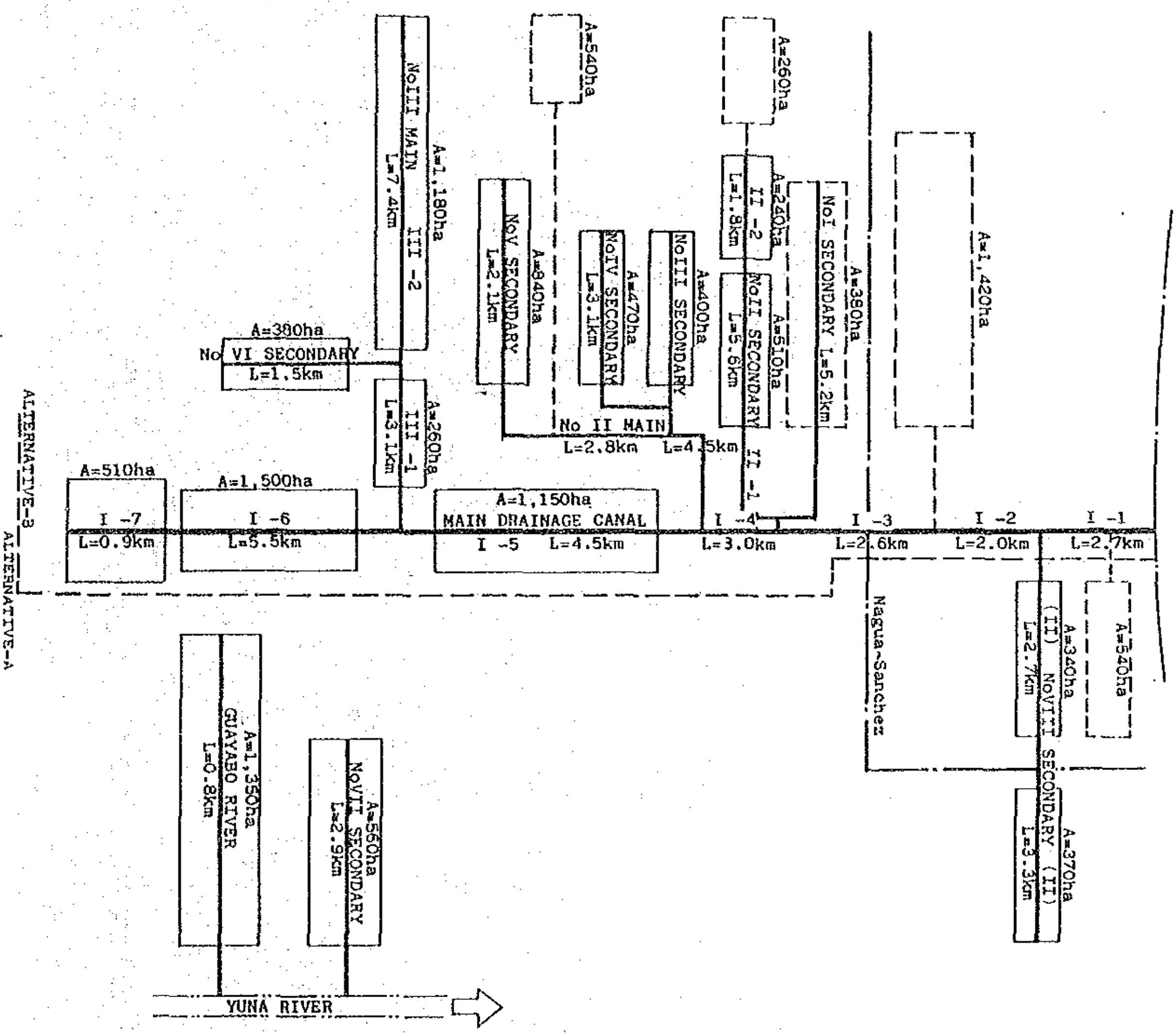


FIG. K.4.1 DRAINAGE CANAL NETWORK

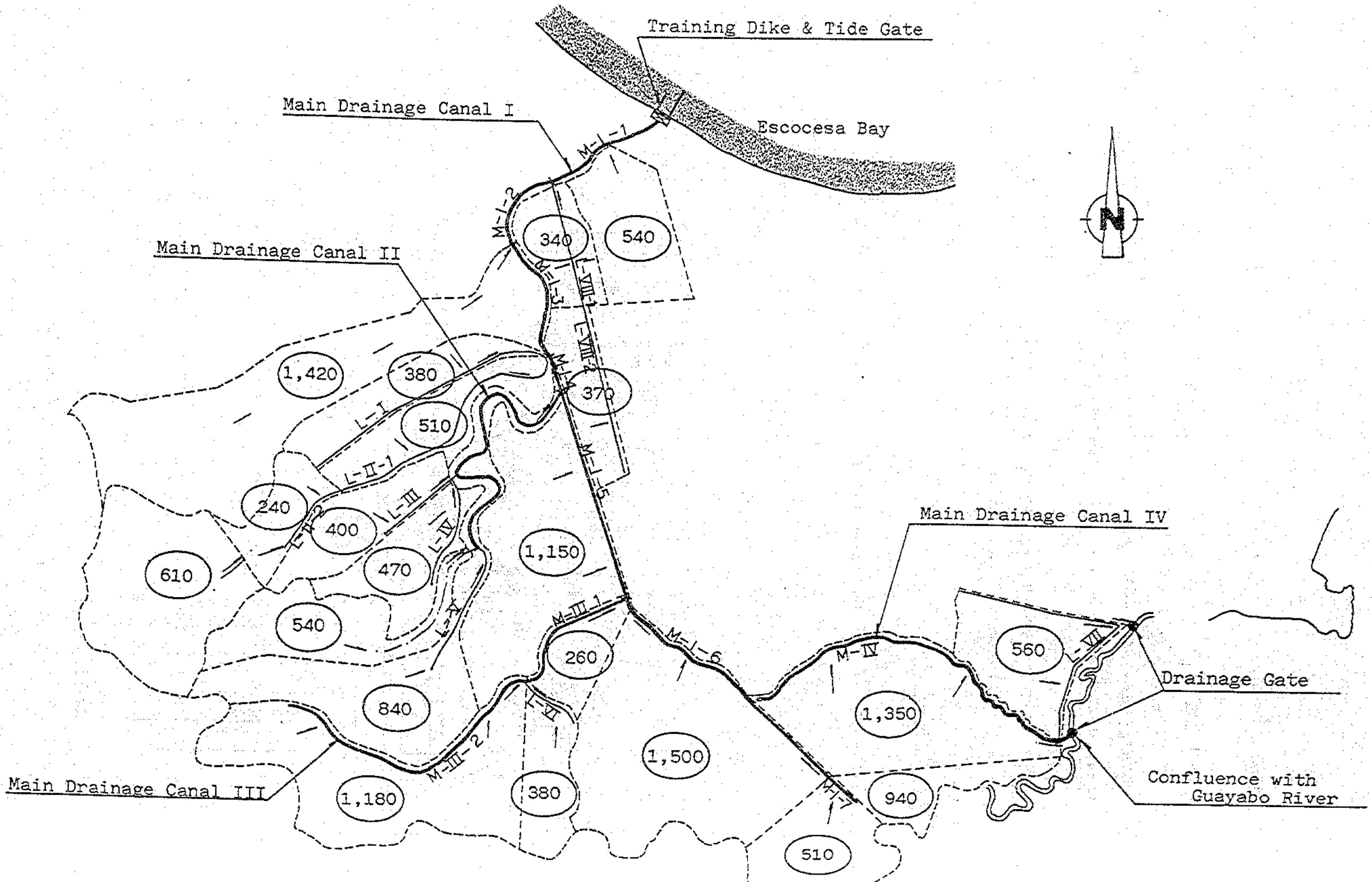


FIG. K.4.2 DIVISION OF THE DRAINAGE BASIN

4.3.3 Design Drainage Discharge

The rational formula was applied for the establishment of design drainage discharge.

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

where, Q : peak discharge (m^3/s)

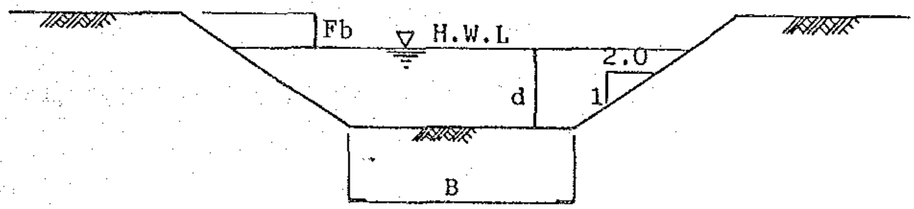
f : runoff coefficient

r : average rainfall intensity for the duration of flooding (mm/hr)

A : area to be drained (km^2)

4.3.4 Section of Drainage Canal

The standard section of a drainage canal is as illustrated below.



Fb; Freeboard

The freeboard will be maintained as high as 30 cm or more.

4.3.5 Main Drainage Canal

(1) Longitudinal Plan of the Main Drainage Canal No. 1

For the purpose of efficient utilization of water resources as well as protection of salt wedge, the installation of a tide gate at the outlet of the Cano Gran Estero has been proposed for Alternative A-I and Alternative A-II; the bottom of the gate which will be a fixed structure is to be determined its elevation within the longitudinal plan of the canal.

The gradient of the river bed was designed to be at 1 : 15,000, which had been determined aiming at adequate removal of excess water stagnated within swamps of El Guayabo.

The elevation of river bed was set out to be -3.0 m, equivalent to that of the Cano Gran Estero.

(2) Other Main & Secondary Drainage Canal

Gradient of others main and secondary drainage canals were determined, depending on actual land slope.

In Table K.4.1 drainage basin and design discharge are summarized.

TABLE K.4.1 CALCULATION OF DESIGN DISCHARGE

Canals	Catchment Area (ha)	Total Catchment Area (ha)	Length (km)	Maximum Length (km)	Land Elevation of Upper Reach (m)	Land Elevation of Lower Reach (m)	Concentration Time (hr)	Intensity Rainfall (mm/hr)	Discharge (m^3/s)
Main Canal									
I-7	510	510	1.5	4.0	3.0	1.8	7.2	10.7	6.1
I-6	1,500	2,010	5.5	9.5	7.5	1.8	11.3	8.6	19.2
I-5	1,150	4,980	4.5	15.0	9.0	1.8	20.4	6.4	35.4
I-4	-	7,400	1.3	20.6	9.0	1.8	33.9 (24.0)	5.9	48.5
I-3	-	8,870	2.6	20.6	9.0	1.8	33.9 (24.0)	5.9	58.1
I-2	1,420	10,560	2.0	25.2	9.0	1.8	45.3 (24.0)	5.9	69.2
I-1	710	11,270	2.7	27.9	9.0	1.8	53.4 (24.0)	5.9	73.9
II-2	70	1,450	2.8	14.8	7.5	3.0	26.5 (24.0)	5.9	9.5
II-1	100	2,420	4.5	19.3	7.5	1.8	35.2 (24.0)	5.9	15.9
III-2	1,180	1,180	7.4	7.4	10.0	3.0	6.7	11.2	14.6
III-1	260	1,820	3.1	10.5	10.0	3.0	10.8	8.8	17.8
IV (Guayabo)	1,350	1,350	8.0	8.0	1.5	0.8	30.2 (24.0)	5.9	8.9
Secondary Canal									
I	380	380	5.2	5.2	3.8	1.8	8.1	10.2	4.3
II-2	240	850	1.8	8.8	9.0	4.5	11.5	8.5	8.0
II-1	510	1,360	5.6	14.4	9.0	1.8	19.1	6.6	10.0
III	400	400	2.1	2.1	4.0	3.0	2.9	17.0	7.5
IV	470	470	3.1	3.1	4.5	3.0	4.2	14.1	7.4
V	840	840	3.1	3.1	4.0	3.5	8.1	10.2	9.5
VI	380	380	1.5	1.5	4.0	3.0	1.7	22.2	9.4
VII	560	560	2.9	2.9	1.5	1.0	(24.0)	5.9	3.7
VIII-2	370	370	3.3	3.3	3.0	1.0	(24.0)	5.9	2.4
VIII-1	340	710	2.7	6.0	3.0	1.0	(24.0)	5.9	4.7

TABLE K.4.2 DETERMINATION OF CANAL SECTION

Canals	Design Discharge (m^3/s)	Length of Canal (km)	Gradient	Depth (m)	Width of Canal Bed (B)	Area (m^2)	Perimeter (m)	Velocity (m/s)	Capacity of Canal (m^3/s)
Main Canal									
I-7	6.1	1.5	1/15,000	3.4	1.5	28.2	16.7	0.29	8.1
I-6	19.2	5.5	1/15,000	3.4	10.0	57.1	25.2	0.35	20.1
I-5	35.4	4.5	1/15,000	3.4	20.0	91.1	35.2	0.38	35.1
I-4	48.5	0.8	1/15,000	3.4	30.0	125.1	45.2	0.40	50.4
I-3	58.1	2.6	1/15,000	3.4	35.0	142.1	50.2	0.41	58.1
I-2	69.2	2.0	1/15,000	3.4	42.5	167.6	57.7	0.41	69.6
I-1	73.9	2.7	1/15,000	3.4	45.0	176.1	60.2	0.41	73.9
II-2	9.5	2.8	1/ 6,000	2.0	8.0	24.0	16.9	0.41	9.8
II-1	15.9	4.5	1/ 6,000	2.0	15.0	38.0	23.9	0.44	16.7
III-2	14.6	7.4	1/ 1,500	2.0	6.0	20.0	14.9	0.78	15.6
III-1	17.8	3.1	1/15,000	2.5	20.0	62.5	31.2	0.32	20.3
IV (Guayabo)	8.9	8.9	1/20,000	2.0	15.0	38.0	23.9	0.24	9.1
Secondary Canal									
I	4.3	5.2	1/ 2,000	1.5	3.0	9.0	9.7	0.53	4.8
II-2	8.0	1.8	1/ 2,000	2.0	3.0	14.0	11.9	0.62	8.7
II-1	10.0	5.6	1/ 2,000	2.0	4.0	16.0	12.9	0.64	10.3
III	7.5	2.1	1/ 2,000	2.0	2.5	13.0	11.4	0.60	7.9
IV	7.4	3.1	1/ 1,500	2.0	2.0	12.0	10.9	0.69	8.2
V	9.5	3.1	1/ 6,000	2.0	8.0	24.0	16.9	0.41	9.8
VI	9.4	1.5	1/ 1,500	2.0	3.0	14.0	11.9	0.71	10.0
VII	3.7	2.9	1/20,000	2.0	5.0	18.0	13.9	0.21	3.8
VIII-2	2.4	3.3	1/ 6,000	2.0	2.0	12.0	10.9	0.34	4.1
VIII-1	4.7	2.7	1/ 6,000	2.0	3.0	14.0	11.9	0.35	5.0

4.3.6 Tertiary Drainage Canal

Tertiary drainage canal will be installed at the interval of every 700 ha and the section was determined based on daily removal of collected rainfall. The following four types have been proposed.

Type	Width of Canal Bed	x	Width of Canal Top	x	Depth
Type I	0.5		2.5		1.0 m
Type II	1.0		3.0		1.0 m
Type III	1.5		3.5		1.0 m
Type IV	2.0		4.0		1.0 m

Discharge by the gradient of canal are as follows:

(m^3/s)

Type	1/1,000	1/2,000	1/3,000	1/5,000
Type I	1.03	0.73	0.59	0.45
Type II	1.37	0.97	0.79	0.61
Type III	1.73	1.22	1.00	0.77
Type IV	2.09	1.48	1.21	0.92

Therefore, the section of canal was determined according to gradient of canal and drain basin.

(ha)

Type	1/1,000	1/2,000	1/3,000	1/5,000
Type I	150	110	90	60
Type II	200	140	120	90
Type III	260	180	150	110
Type IV	310	220	180	140

ANNEX L: FACILITIES PLAN

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ANNEX L: FACILITIES PLAN

1. Outline of Facilities

1.1 Irrigation Works

1.1.1 Main Intake Facilities

Main intake facilities to be included within the development plan are as featured below:

(1) Headworks

Alternatives in which this installation is proposed	:	A-I, B-I
Location	:	Arenoso
Type	:	Floating type
Height of weir	:	3.9 m
Length of movable weir	:	68.5 m
Intake rate	:	5.9 m^3/s

(2) Pumping Station

Alternatives in which this installation is proposed	:	A-II, B-II
Location	:	Arenoso
Type	:	Vertical type mixed flow pump
Diameter and quantity	:	ϕ900 x 3
Actual pump head	:	4.3 m
Discharge capacity	:	5.9 m^3/s

1.1.2 Secondary Intake Structures

Secondary intake facilities are proposed in Alternatives A-I, A-II, Irrigation water will be taken exclusively by means of pumps (inclined axial mixed flow pump). Design criteria of these pumps are as presented below:

	Discharge (m3/s)	Actual Head (m)	Diameter and Quantity
No. 1 Pumping Station	0.46	5.5	ϕ350 mm x 2
No. 2 "	0.21	4.8	ϕ350 mm x 1
No. 3 "	0.41	3.1	ϕ350 mm x 2

1.1.3 Irrigation Canals

The length of irrigation canal for each Alternative is as summarized in the table below:

Unit: m

	Alternatives			
	A-I	A-II	B-I	B-II
Main Canal	62,650	62,100	56,550	56,000
Type A	35,450	34,900	34,200	33,650
Type B	27,200	27,200	22,350	22,350
Secondary Canal	242,600	242,600	200,900	200,900
Type A	102,700	102,700	86,100	86,100
Type B	112,100	112,100	90,900	90,900
Type C	27,800	27,800	23,900	23,900
Total	305,250	304,700	257,450	256,900

1.1.4 Diversion Works and Check Gate

The quantity of diversion works and check gate to be included in each Alternative is as follows:

Works	Alternatives			
	A-I	A-II	B-I	B-II
Diversion Works	95	95	80	80
large size	6	6	6	6
small size	89	89	74	74
Check Gate	30	30	25	25
large size	5	5	5	5
small size	25	25	20	20

1.2 Drainage Works

1.2.1 Drainage Canals

The length of canals for each Alternative is as listed below:

Unit: m

	Alternatives			
	A-I	A-II	B-I	B-II
Main Canal	44,300	44,300	44,300	44,300
Secondary Canal	31,300	31,300	22,400	22,400
Tertiary Canal	114,700	114,700	99,900	99,900
Total	190,300	190,300	166,600	166,600

1.2.2 Tide Gate

The installation of tide gate is proposed in Alternatives A-I and A-II.

Location: Outlet of the Caño Gran Estero

Gate : B 13.50 m x H 4.00 m x 3 sets

1.2.3 Training Dike

The installation of training dike is proposed in all Alternatives.

Location: Outlet of the Caño Gran Estero

Length : Right bank - 120 m

Left bank - 200 m

Total: 320 m

1.2.4 Outlet of Drainage Canal

The outlet of drainage canal is proposed in the following two types. The quantity of works is the same for each Alternative.

Outlet with gate	:	17
Outlet without gate	:	19
Total		36

1.3 Road Works

1.3.1 Roads

The length of road to be constructed is summarized in each Alternative as below:

Unit: m

	Alternatives			
	A-I	A-II	B-I	B-II
Trunk Road	700	700	700	700
Trunk In-farm Road	46,700	46,700	43,900	43,900
To be constructed	18,400	18,400	18,400	18,400
To be improved	28,300	28,300	25,500	25,500
Lateral In-farm Road I	67,850	67,300	58,650	58,100
To be constructed	64,250	63,700	56,850	56,300
To be improved	3,600	3,600	1,800	1,800
Lateral In-farm Road II	97,300	97,300	79,050	79,050
Total	212,550	212,000	182,300	181,750

1.3.2 Bridges and Culverts

The quantity of bridges and culverts to be proposed in each Alternative is presented in the table below.

Component	Alternatives			
	A-I	A-II	B-I	B-II
Bridge	9	9	8	8
Box Culvert	15	15	12	12

2. Preliminary Study of Main Intake Facilities

Preliminary study has been made with respect to the following three proposals of irrigation water intake method, namely:

- Regulating reservoir,
- Headworks and
- Pumping station

In this section, technical justification for the withdrawal of regulation reservoir proposal as well as comparative study on the location of headworks proposal is described.

2.1 Regulating Reservoir Proposal

2.1.1 Location

The location of a regulating reservoir was proposed near the existing El Aguacate Pumping Station, in the western part of the study area and in close vicinity to Arenoso (refer to Fig. L.2.1).

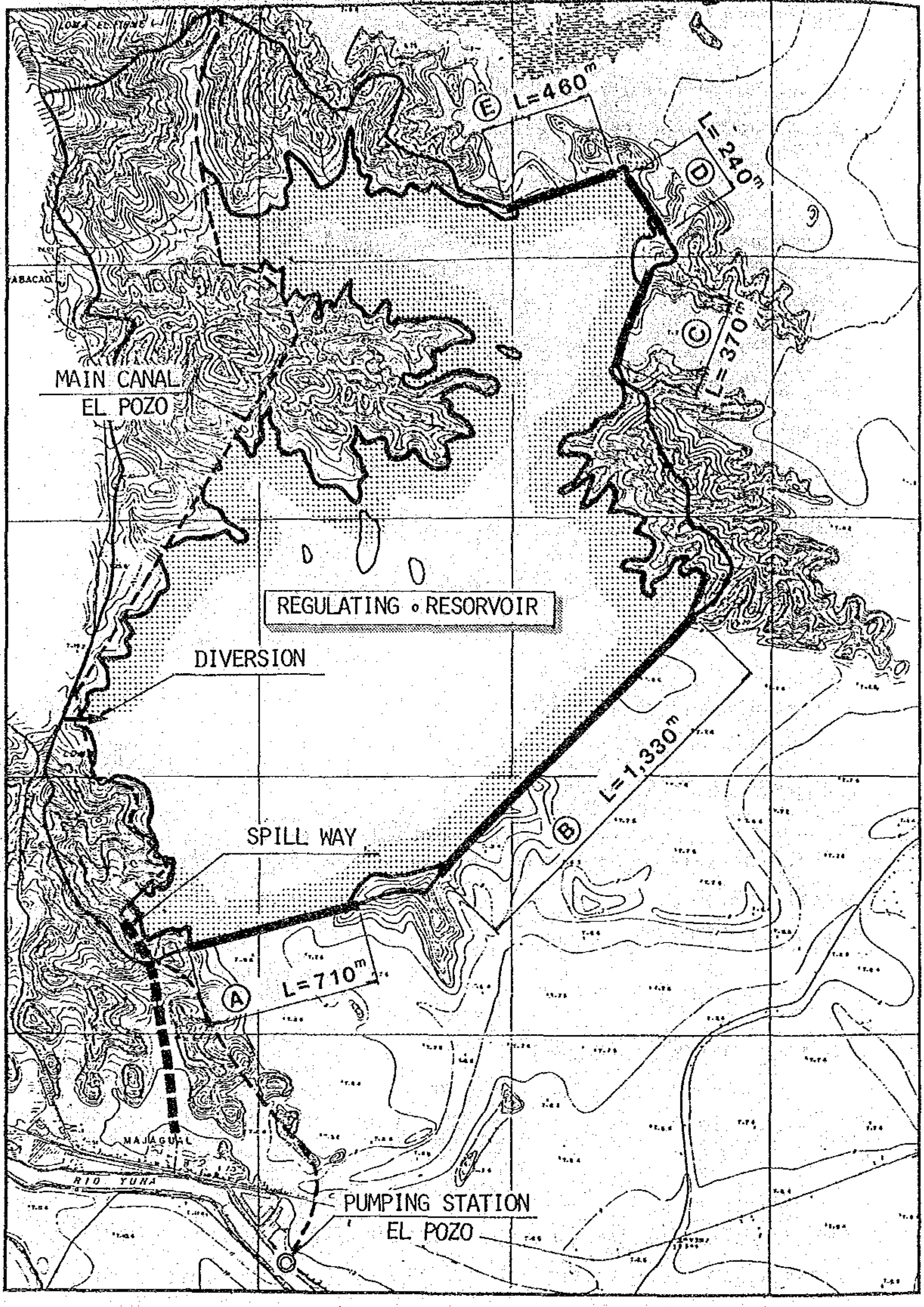


FIG L.2.1 PLAN OF REGULATING RESERVOIR

2.1.2 Full Water Level

In due consideration of embankments of main canals designed for the El Pozo Project and for the purpose of alleviating uplift pressure to be expected to the lining structure of main irrigation canal, the full water level was set out to be 14.00 m.

2.1.3 Storage Volume

The storage volume to comply with water level in consultation with the topographic map with a scale 1/10,000.

Water Level (m)	Height (m)	Area ($x10^3m^2$)	Average Area ($x10^3m^2$)	Volume ($x10^3m^3$)	Cumulative Volume ($x10^3m^3$)
7.6	0	0	0	0	0
8.0	0.4	350	175	70	70
9.0	1.0	2,270	1,310	1,310	1,380
10.0	1.0	3,220	2,745	2,745	4,125
12.0	2.0	3,540	3,380	6,760	10,885
14.0	2.0	4,100	3,820	7,640	18,525

2.1.4 Volume of Sediment

Given annual volume of sediment to be 100 m^3/km^2, the total sediment to be accumulated for 100 years was calculated as follows:

$$V = 100\ m^3/km^2/year \times 6.61\ km^2 \times 100\ years = 66,100\ m^3$$

As a result, the design sediment level was established to be EL 8.00 m.

2.1.5 Dam

(1) Type

In view that impervious materials are easily available around the location and the height of dam is supposed comparatively low, a homogeneous type earth dam was proposed.

(2) Slope Gradient

In compliance with height of dam, embankment materials, geological structure of sub-base, etc., the following slope gradients were considered:

1) Dams to be constructed on poor ground : A, B, C

Upper part - 1 : 3.5
Lower part - 1 : 3.0

2) Dams to be constructed on favorable ground: D, E

Upper part - 1 : 2.5
Lower part - 1 : 2.0

(3) Width of Crest

According to the "Design of Small Dam" prepared by Bureau of Reclamation, United States Department of the Interior, the width of crest is calculated in the following manner:

$$W \geq 0.2H + 3.0 \text{ (m)}$$

where, W : width of crest
H : height of dam, 16.0 - 7.0 = 9.0 (m)

Then,

$$W \geq 0.2 \times 9.0 + 3.0 = 4.8 \text{ (m)}$$

Considering that the crest would be also used as a road, the width of 6.0 m was proposed.

(4) Freeboard

The freeboard of a dam with its height less than 10 m is computed as below:

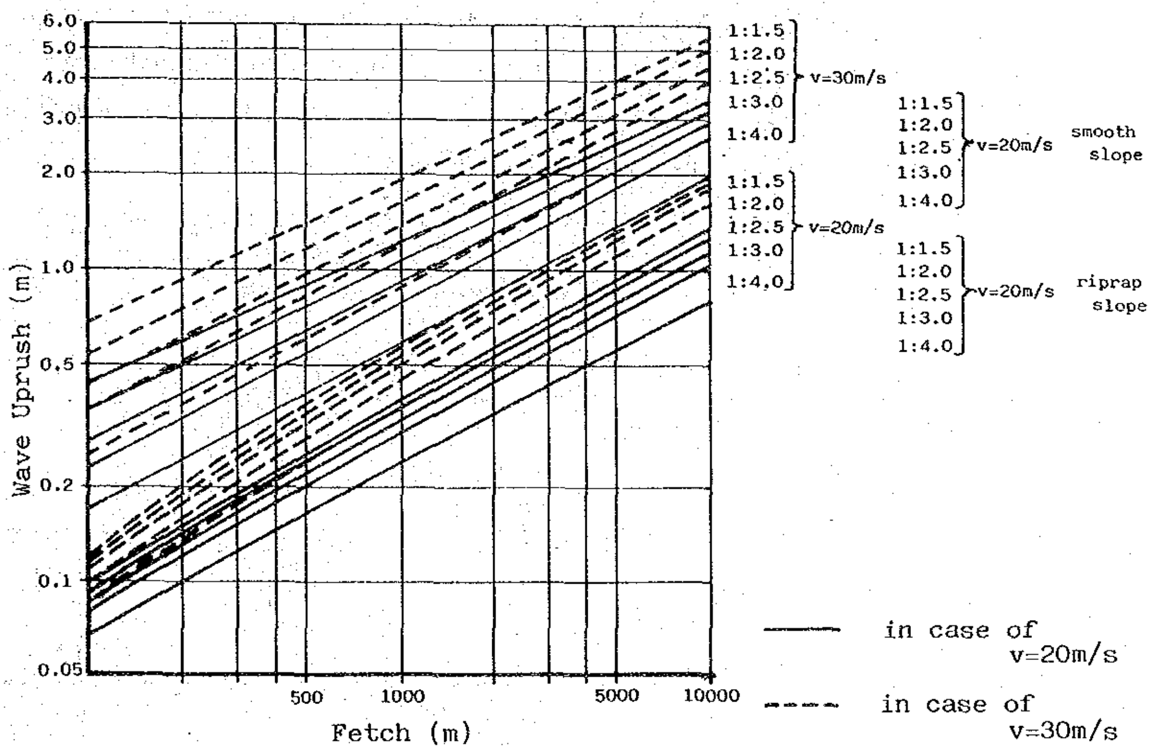
$$Fb = 0.05H + \text{wave height}$$

where, Fb: freeboard
H : height between design flooding level and sub-base (14.60 - 7.00 = 7.60 m)

If the wave height will be less than 1 m, it will be raised to 1 m.

The wave height is computed by means of the S.M.B. and the Saville Methods as illustrated below:

HEIGHT OF WAVE CAUSED BY WIND



Given, V = 30 m/s, fetch = 1,750 m, slope gradient = 1 : 3.5 and the slope = riprap slope.

Then, the wave height of 0.65 m is gotton to consult with the above graph. This value, being less than 1 m, becomes 1 m as mentioned above.

As a result, the freeboard is computed as follows:

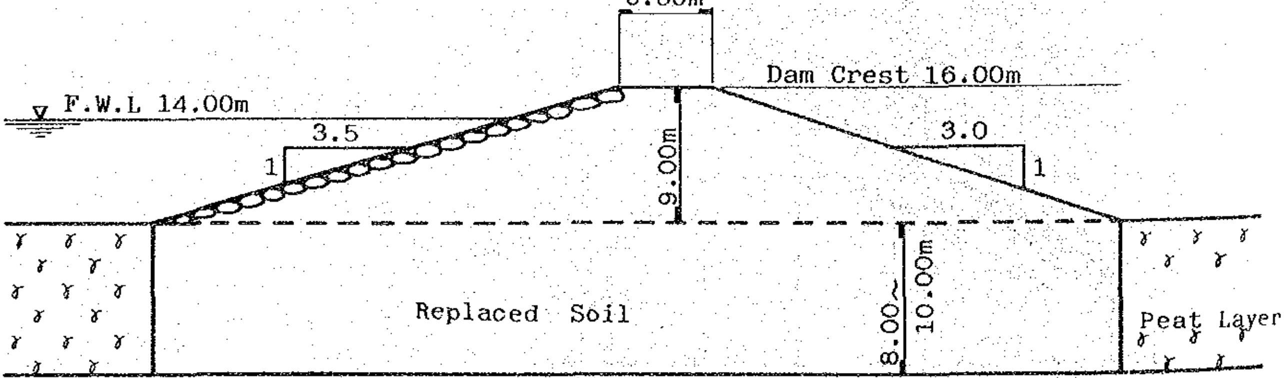
$$Fb = 0.05 \times 7.60 + 1.00 = 1.38 \text{ m}$$

Finally, the freeboard from the normal water level is:

$$Fb = 1.38 + 0.60 \fallingdotseq 1.98 = 2.00 \text{ cm}$$

(5) Standard Profile and Improvement Works on Sub-Base

Three dams of A, B and C were proposed to be constructed on poor ground formed principally be peat soils. In this connection, the standard profile of these dams were delineated including improvement works of the said poor ground; improvement works by applying displacement method was proposed taking the thickness of peat soil layer (8 - 10 m) and the coefficient of water permeability (10^{-2} - 10^{-3} cm/sec) into account. Then, the standrd profile is illustrated as below:



2.1.6 Spillway

(1) Design Flood Discharge

In establishing the design flood discharge, a return period of 1 : 200 was employed. The Rational Formula was applied for the calculation.

$$Qp = \frac{1}{3.6} \cdot f \cdot Rt \cdot A$$

where, Qp : peak runoff discharge (m^3/s)
f : runoff ratio
Rt : rainfall intensity during the time of concentrate (mm/hr)
A : catchment area (km^2)

1) Runoff Ratio (f)

The runoff ratio composed of that at hill area and water surface area was computed as below:

	Area (km^2)	Runoff Ratio
Hill Area	2.56	f1 = 0.5
Water Surface Area	4.05	f2 = 1.0
Total	6.61	

$$\text{Then, } f = \frac{2.56 \times 0.5 + 4.05 \times 1.0}{6.61} = 0.8$$

2) Rainfall Intensity (Rt)

Rainfall intensity is computed as follows:

$$Rt = \frac{R24}{24} \left(\frac{24}{T}\right)^n$$

where, R_{24} : daily rainfall = 214 mm/day
T : time of concentration
n : coefficient 0.5

$$T = 1/W$$

where, l : length from hyraulically most distant point of damsite

W : velocity (km/hr)
$W = 72\ (H/l)^{0.6}$
$= 72\ (0.082/5.0)^{0.6}$
$= 6.11$ km/hr

H : height from hydraulically most distant point to damsite
$= 0.090 - 0.008 = 0.082$ km

$$\therefore T = 5.0/6.11 = 0.82 \text{ (hr)}$$

$$\therefore Rt = \frac{214}{24} \left(\frac{24}{0.82}\right)^{0.5} = 48.2 \text{ (mm/hr)}$$

3) Design Flood Discharge

$$Qp = \frac{1}{3.6} \times 0.80 \times 48.2 \times 6.61 = 70.8 \ m^3/s$$

The design flood discharge was summed up the design discharge of the El Pozo Project to the said unit runoff discharge.

Then, $Qp' = 70.8 + 5.5 = 76.3 \fallingdotseq 77.0 \ m^3/s$

(2) Overflow Depth and Length of Crest

The relation between the overflow depth and the length of crest was presented as below: