Topographical survey, soil and geological investigation works were undertaken for the proposed site identified as more likely to be viable. These sites were identified as Sites D, LD and E. (refer to attached drawing Dr-1)

F. 2. 4 Alternative Study

a) Results of Water Balance Study

In order to establish the optimum scale of the water resources facilities, the water balance study for the potential water resources sites were carried out on a 10-day basis for the duration of 17 years as shown in Appendix C. The Study under the conditions of double cropping (200 percent of cropping intensity) was conducted in the irrigable area of 590 ha with three times water shortage for the duration of 17 years. The major features of potential water resources are resulted as follows;

Site	Watershed	Annual Av. Runoff	Annual Water Requirement R. M	W. P	I. W	Effective Storage
	(sq.km)	(MCM)	(MCM)	(MCM)	(MCM)	(MCM)
(Inagawan River)						
Site C	110.7	99.3	0.5	13.6	4.3	0.21
Site D	118.1	105.9	0.6	13.6	4.3	0.20
Site LD	118.5	106.3	0.6	13.6	4.3	0.20
(Pinagsaluran River)						
Site Eu	14.5	13.0	0.1	-	4.3	1.65
Site El	15.0	13.5	0.1	-	4.3	1.61

(Note) R.M : River Maintenance
W.P : Water Permit of Existing Irrigation System
I. W ; Irrigation Water

In addition, the site EuM is a preferable site for a small scale water resources development such as mountain stream diversion works. The site which has 0.18 MCM effective storage capacity in maximum by its topographical restriction, is available for the following irrigable area under three times shortage for the duration of 17 years.

Wet season : Paddy 430 ha + Upland 160 ha
Dry season : Upland crop and vegetable 177 ha

b) Dimension and Construction Costs

The facility dimensions and construction costs for each potential water resources based on the results of water balance mentioned above were proposed and estimated in accordance with the design concept stipulated in Chapter F.3 and the method in Appendix Kand one presented in Table F.2.2 and F.2.3. (refer to the attached drawings Dr-2 ~ Dr.7)

c) Site Selection

The selection of optimum water resources site for the project shall be made from various view points such as irrigable area, easiness of construction, environment, economical construction cost including operation and maintenance cost.

1) Evaluation Criteria

The evaluation criteria for selection was established as follows:

Item	Score	Contents
Irrigation Conditions	5	200% Cropping intensity
	3	200 ~ 150% Cropping intensity
	1	less than 150% Cropping intensity
Construction conditions	5	Short construction period, reliable construction
	3	Two to three years of construction term, many unknown factors for construction
	1	Long construction period, more than three years, unreliable and/or difficult construction
Environment conditions	5	Less or few effect to the surrounding natural resources such as river water and forest
	3	Rather large effect
	1	Large effect
Construction cost	5	Lower construction cost
	3	Medium construction cost
	1	Higher construction cost
Operation and Maintenance conditions	5	Economical and easy O & M
	3	Intermediate
	1	Expensive and difficult O & M

2) Site Evaluation

Based on the criteria established, the potential water resources sites were evaluated as follows:

	Site Eu	Site El	Site D	Site C	Site EuM	Site LD
Watershed (sq.km)	14.5	15.0	118.1	110.7	13.9	118.5
Intake Type	Gravity Intake					Pumping Intake
Cropping Intensity	200%				130%	200%
App. Construction Cost (MP)*1 App. O & M Cost (MP)*2	444 -	611 -	1,552 -	545 -	176 -	342 77
(Evaluation) - Irrigation conditions - Construction conditions - Environment conditions - Construction cost - O & M conditions Total Score	 5 3 3 3 5 19	 5 1 3 3 5 17	 5 1 1 1 3 11	 5 1 1 3 3 13	 1 5 5 5 5 21	 5 3 5 3 1 17
Ranking	2	2	6	5	1	3

*1: Refer to Appendix K Table K.4.1
*2: refer to Appendix K

The table indicates that implementation of Site EuM which has the lowest cropping intensity among the potential water resources sites, only 130% is judged to be viable because of its good conditions. While, in case of double cropping (200% cropping intensity), Site Eu has higher priority for water resources development for the project.

Table F.2.1 General Features of Potential Sites on Water Resources Development

(1) Site Name	Site A	Site B	Site C	Site D	Site E	Site F
(2) Location						
a) Province	Palawan	Palawan	Palawan	Palawan	Palawan	Palawan
b) City	Puerto Princesa	Puerto Princesa	Puerto Princesa	Puerto Princesa	Puerto Princesa	Puerto Princesa
(3) River						
a) Name	Inagawan	Inagawan	Inagawan	Inagawan	Pinagsa-luran	Branch of P-inagsaluran
b) Watershed (Km2)	84.4	105.4	110.7	118.1	15.0	3.7
c) Average Annual Rainfall (mm/y)	1,590	1,590	1,590	1,590	1,590	1,590
d) Assumed Average Runoff Coefficient	0.5	0.5	0.5	0.5	0.5	0.5
e) Assumed Average Annual Runoff (MCM)	67.1	83.8	88.0	93.9	11.9	2.9
f) Riverbed Elevetion (m)	130.0	54.5	23.5	21.3	29.0	45.5
g) Distance from the Guaging Station No2 (m)	14.0	6.7	1.8	1.3	---	---
(4) Site Conditions						
a) Storage Capacity by Topography (MCM)	931.3	30.4	5.4	28.3	19.0	1.5
[W.L (m)]	300.0	100.0	50.0	70.0	70.0	70.0
b) Watershed Vegetation	thick forest	thick forest	thick forest	thick forest	thick forest	thick forest
c) Approx. River Runoff (cms) as of Feb.1994	no observat-ion	1.0-1.5	1.0-1.5	1.0-1.5	0.2~ 0.3	no water flow
d) Riverbed Foundation	no observat-ion	shallow riv-er deposit/ outcrop of freh sand-stone	comparativ-ely shallow river depos-ite/outcrop of shale	rather deep river depos-ite/outcrop of shale	Miocene, Qua-ternary sed-iment/rather thick river deposit	Miocene, Qua-ternary sed-iment

(Note)
1) Average Annual Rainfall (mm/y) : Aborlan Rainfall Data (1977-1993)
2) The storage capacity of site A was based on the topo-map scaled 1/50,000 and the other capacities , scaled 1/4,000 .

Table F.2.2
Alternative Plan of Water Resources Sites

Site		Site Eu	Site El	Site D	Site C	Site EuW	Site LD	Remarks
(1) Intake Type		Gravity w/ Reservoir					Pump w/ Weir	
(2) Water Resources								
a) River Name		Pinagsaluran	Pinagsaluran	Inagavan	Inagavan	Pinagsaluran	Inagavan	
b) Watershed	(Km2)	14.5	15.0	118.1	110.7	13.9	118.5	
c) Riverbed Elevation	(m)	34.5	29.0	21.2	23.5	34.5	19.5	
(3) Reservoir								
a) Required E. Storage	(MCM)	1.65	1.61	0.20	0.21	0.20	0.20	
b) Sediment Volume	(MCM)	0.44	0.45	2.36	2.21	0.11	—	
c) Dead Volume	(MCM)	0.44	0.45	2.36	2.21	0.11	0.06	
d) H.W.L (MSL)	(m)	54.00	46.50	40.00	42.50	45.00	25.50	
e) L.W.L (MSL)	(m)	46.00	37.80	39.00	42.00	41.00	21.50	
f) W. Surface at H.W.L	(ha)	29	28	33	31	8	9	
(4) Major Feature of Dam/Weir								
a) Dam Type		Filltype Dam	Filltype Dam	Concrete Dam	Concrete Dam	Filltype Dam	Concrete Weir	
b) Dam Crest Elevation	(m)	58.00	50.50	44.00	47.00	50.00	31.50	
c) Dam Height	(m)	28.0	25.5	46.0	30.0	20.0	14.5	
d) Dam Crest Length	(m)	875	868	355	155	239	221	
e) Design Flood Discharge	(c.m.s)	430	440	1600	1550	420	990	
f) Intake Discharge 1	(c.m.s)	0.84	0.84	0.84	0.84	0.84	—	
Intake Discharge 2	(c.m.s)	—	—	0.45	0.45	—	0.45	
(5) Major Feature of Pump								
a) Type of Pump		—	—	—	—	—	Vertical Pump	
b) Design Head	(m)	—	—	—	—	—	ϕ450×3sets	
c) Design Discharge	(c.m.s)	—	—	—	—	—	0.84	
d) Output of Pump	(KW)	—	—	—	—	—	190×3 sets	
(6) Approx. Direct Construction Cost								
a) Dam/Weir	(M.P)	437	607	1514	483	169	188	
b) Leading Canal	(M.P)	7	4	38	62	7	38	
b) Pump	(M.P)	—	—	—	—	—	116	
c) Total Construction Cost	(M.P)	444	611	1552	545	176	342	

(Note) Intake Discharge 1 :Discharge for the project

Intake Discharge 2 :Discharge for the existing project & river maintenance flow

Conditions:

Irrigable Area (ha) 590

Type I Area (ha) 430 (Paddy+Upland crop)

Type II Area (ha) 160 (Vegetable+Upland crop)

Table F.2.3
Earthquake Analysis

	Hypocenter			Site	
	LAT (N2) (°)	LONG (E2) (°)	Ms Magnitude	R (Km)	A (cm/sec2)
1	12.67	120.96	5.2	436.8	0
2	11.63	121.88	5.6	434.2	0
3	12.19	120.66	5.5	374.1	0
4	12.67	120.49	5.2	407.2	0
5	8.92	121.19	5.2	298.8	0
6	12.25	121.04	5.4	406.5	0
7	12.69	121.93	5.3	511.0	0
8	12.23	120.30	5.1	354.4	0
9	12.90	120.20	5.1	414.2	0
10	11.58	121.49	5.4	394.9	0
11	11.51	121.39	5.9	381.3	0
12	11.36	121.18	5.2	352.6	0
13	11.00	120.00	6.1	225.9	0
14	13.00	120.20	5.0	424.2	0
15	8.00	121.00	6.3	319.9	0
16	11.00	120.00	5.7	225.9	0
17	12.60	121.60	5.7	477.7	0
18	13.00	120.00	5.0	415.2	0
19	13.00	120.30	5.0	429.1	0
20	12.00	119.00	5.8	276.7	0
21	9.00	118.00	5.5	89.0	0.035793
22	11.00	120.00	5.0	225.9	0
23	12.28	121.17	5.2	418.8	0
24	13.10	120.80	5.0	466.0	0
25	13.02	120.83	5.0	460.3	0
26	13.20	121.50	5.1	520.2	0
27	13.45	120.33	5.0	474.8	0
28	13.80	120.70	5.1	528.6	0
29	13.50	120.50	5.0	488.8	0
30	13.60	120.54	5.2	500.8	0
31	13.78	120.71	5.4	527.1	0
32	13.28	121.36	5.5	517.8	0
33	13.76	120.84	5.0	531.8	0
34	13.40	120.80	5.3	494.6	0
35	13.98	120.74	5.1	548.5	0
36	13.82	119.91	5.0	497.8	0
37	13.57	120.06	5.2	476.8	0
38	13.96	120.87	5.0	553.1	0
39	13.67	120.55	5.1	508.3	0
40	11.52	121.89	5.3	428.7	0
41	13.80	120.80	5.3	533.7	0
42	10.00	122.00	5.1	383.9	0
43	10.73	121.78	5.0	379.6	0
44	10.47	121.50	6.3	340.7	0
45	13.10	121.57	5.7	516.6	0
46	13.23	120.58	5.7	468.2	0
47	12.80	119.90	5.4	390.4	0

(Note)

① The earthquake data are originated from PAGASA.

② Dam Site Latitude N1= 9.55 (°)

Longitude E1= 118.58 (°)

A:Mean peak acceleration (in cm/sec2)

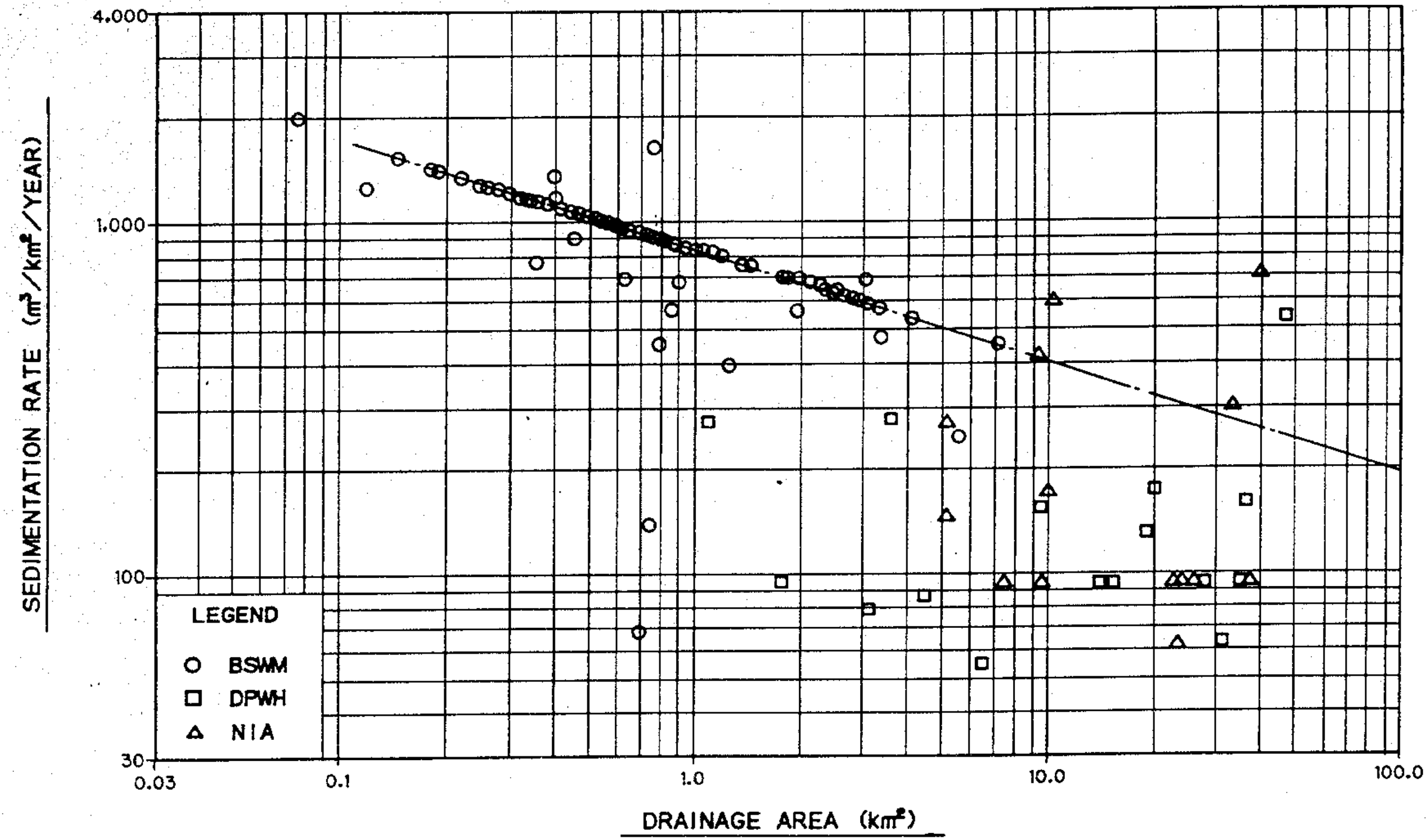
Log A = 0.41Ms-Log(R+0.032×10^0.41Ms)-0.034R+1.3

R:Shortest distance between the site and the hypocenter

R =(ER× π /180)√((N1-N2)^2+(E1-E2)^2) (Km)

Where ER = 6,377 Km

Figure F.2.1 Sedimentation Rate - Drainage Area Relationship



Source : The Water Plan Study on The Small Water
Impounding Management (SWIM)/SEp. 1989 JICA

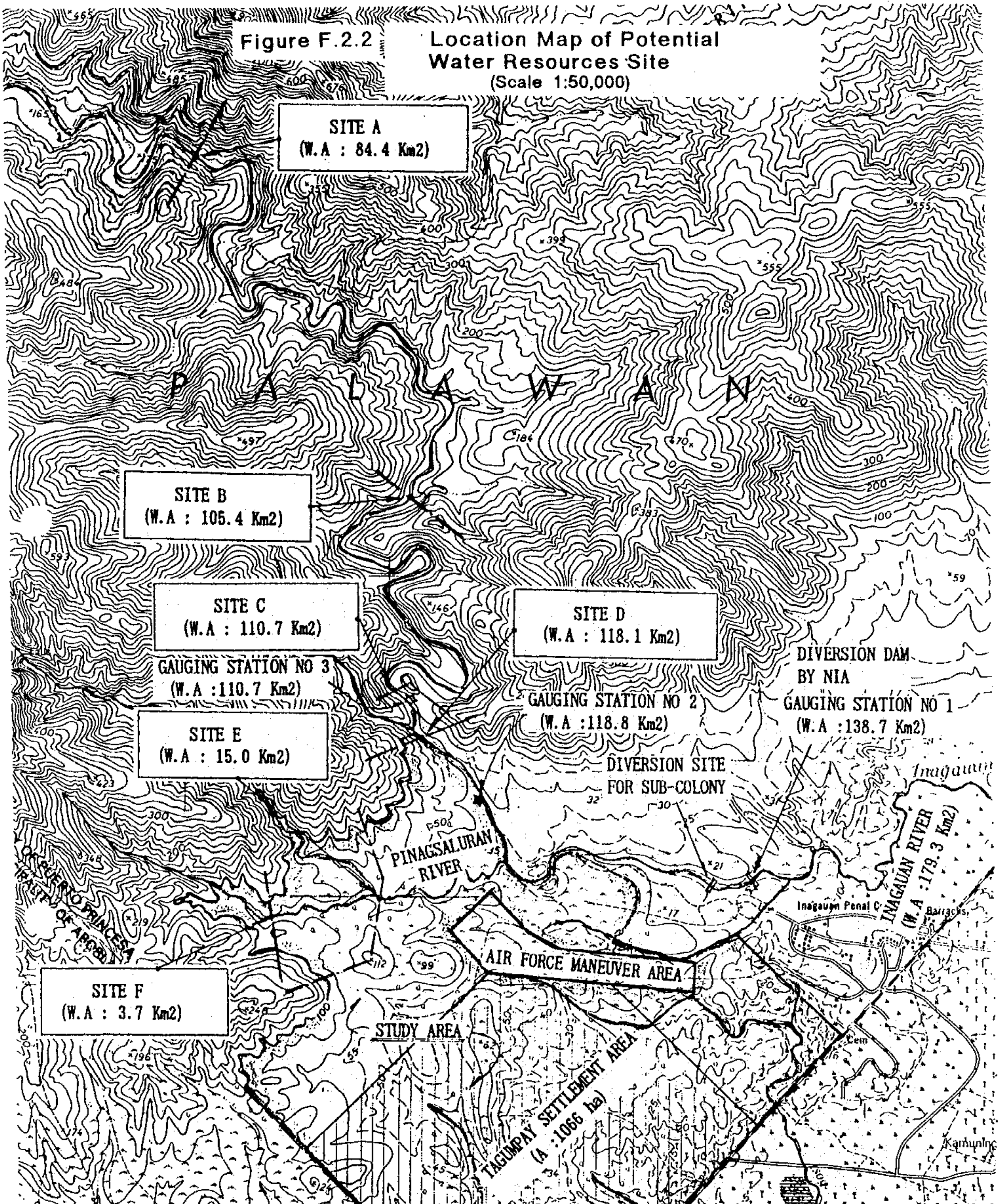


Figure F.2.2 Location Map of Potential Water Resources Site
(Scale 1:50,000)

F.3 DESIGN CONCEPT OF WATER RESOURCES FACILITIES

F.3.1 Design Concept

a) Reservoir Plan

1) Reservoir Capacity

The reservoir capacity and area of each potential water resources site were measured based on the topographical map with a scale of 1 to 4,000 as shown in Figure F.3.1 to F.3.6.

2) Sediment Volume

As described in Para. F.2.2 Sedimentation, the specific sediment volume for the Pinagsaluran and Inagawan river basins were assumed at 300 and 200 cu.m/year/sq.km, respectively, and applied the design period of 100 years of sediment accumulation for reservoir.

However, sediment volume will not be accounted at Site LD, diversion dam type because this type pf facility provides sand sluiceway. Site EuM is also planned with a sand sluiceway at the right river course, thus, the design period of 25 years of sediment accumulation will be employed.

For the above considerations, the design sediment volume for each water resources site are as follows:

	Site Eu	Site El	Site C	Site D	Site LD	Site EuM
Watershed (sq.km)	14.5	15.0	110.7	118.1	118.5	13.9
Sediment (MCM)	0.44	0.45	2.21	2.36	-	0.11

3) Design Water Level

Based on the effective storage volume and sediment volumes computed, the normal water level (NWL) and low water level (LWL) for each water resources site is determined from its reservoir capacity and area curve, as follows:

		Site Eu	Site El	Site C	Site D	Site LD	Site EuM
E. Storage	(MCM)	1.65	1.61	0.21	0.20	0.20	0.20
Sediment V	(MCM)	0.44	0.45	2.21	2.36	-	0.11
Dead V	(MCM)	0.44	0.45	2.21	2.36	0.06	0.11
Total Storage	(MCM)	2.09	2.06	2.42	2.56	0.26	0.31
NWL	(MSL)	54.00	46.50	42.50	40.00	25.50	45.00
LWL	(MSL)	45.60	37.80	42.00	39.00	21.50	41.00

b) Seismic Force

The main parts of the Philippine Island excluding Palawan Island are enclosed within Philippine trench and East Luzon trough at the east side, and North Luzon trough, Manila trench, Negros trench, Sulu trench and Cotabato trench at the west side. Philippine fault and Mindanao fault with north-south direction run across the middle portion of the enclosed zone.

Frequent earthquakes have occurred in the enclosed zone within the western and eastern trenches and troughs. (refer to Figure F.3.7)

Based on the report on the Estimates of Regional Ground-Motion Hazard in the Philippines published by the PHIVOLCS, peak horizontal ground accelerations on rock foundation for a 10% probability of being exceeded in 50 years range from a low of 0.11 g in the Visayas to a high of 0.30 g in the vicinity of Casiguran fault zone in eastern Luzon. Further estimates for soft soil conditions range between 0.27 g for the Visayas and 0.80 g along the Casiguran fault zone.

As to the Palawan Island, the ground acceleration for rock foundation by earthquake belongs to non-affected zones and those for soft soil to not more than 0.30 g zones. (refer to Figure F.3.8 to F.3.10)

Based on earthquake data (within the area between 117 to 112 degrees of east longitude and 8 to 13 degrees latitude, 34 years duration from 1960 to 1993 not less than 3.4 surface-wave magnitude, refer to Figure F.3.11) collected, ground-motion analysis was made, applying the Fukushima and Tanaka's attenuation equation.

The results indicate that the peak horizontal ground accelerations in the Study Area are hardly small, only 3.7 E-5g. Therefore, it is acceptable to apply the minimum design value of 0.05 g of earthquake force K in the structural design (refer to Table F.3.1)

c) Design Flood Discharge

The flood discharge with a 100 years return period is generally applied for the dam design flood discharge at NIA and DPWH. Since there are no available data of long term runoff for flood analysis in or surrounding the study area, the design flood discharge is assumed based on the following methods.

- By the use of flood formulas derived from DPWH's design guidelines criteria and standards.
- By the use of design flood discharge of the NIA's existing diversion dam.
- By the use of rational method based on the Aborlan daily rainfall data.

1) DPWH's Flood Formula

Rare case and occasional case of flood formula are adopted with storage dam and weir design respectively, as follows:

For dam : $Q\,(\text{rare}) = 155 \cdot A / \sqrt{A+13}$ (cu.m/sec)
For weir : $Q\,(\text{occasional}) = 85 \cdot A / \sqrt{A+11}$ (cu.m/sec)

Where A: watershed (sq.km)

2) Design Flood Discharge of NIA's Diversion Dam

Watershed A = 138.7 (sq.km)
Design discharge Q = 420 (cu.m/sec)
Then applying Creager's formula,

$Q = 35.7 / \sqrt{A}$ (cu.m/sec)
Where A: watershed (sq.km)

3) Rational Method

$$Q = 0.2778 \cdot C \cdot I \cdot A \quad \text{(cu.m/sec)}$$

where

C: coefficient of runoff which depends on the topographical character of the drainage area, 0.60

I: rainfall intensity for a duration equal to the time of concentration (mm/hr)

	Site Eu	Site El	Site C	Site D	Site LD	Site EuM
I (mm/ha)	82 *1	82*1	57*1	57*1	50*2	82*1

*1; 100 years return period
*2; 50 years return period

A: watershed

The design flood discharges for the potential water resources sites are estimated applying the above methods as follows;

	Site Eu	Site El	Site C	Site D	Site LD	Site EuM
Watershed (km^2)	14.5	150	110.7	118.1	118.5	13.9
Discharge (m^3/sec)						
1)	430	440	1,550	1,600	890*1	420
2)	140	140	380	390	390	140
3)	200	210	1,060	1,130	990	190
Max. Discharge	430	440	1,550	1,600	990	420

*1: applying occasional formula

d) Dam Type

The dam type shall be determined taking into account such various conditions as topography, geology, available construction materials, construction method, environment, safety structure and economy in addition to objective and scale of facility.

(Site Eu, El)

- Dam span and height rates of the sites are quite large, 30 to 35. So concrete type dam is not economical.
- The foundations of sites are not suitable for high concrete dam due to thick weathered bed rock and overburden layer.
- There are sufficient borrow areas for embankment materials near the site.

From the above reasons, the fill-type dam is applicable for the sites. In addition, Site El shall be provided with deep cutoff wall for reduction of seepage flow due to thick river deposit, about 20 m depth with high permeability ($k = n \times 10^3$ cm/sec).

(Site D, C)

- Dam span and height rate of the site is five (5) to eight (8), a narrow valley.
- The bed rocks below the river deposit at the sites are comparatively fresh, so as to have sufficient bearing capacity for concrete dam.
- The construction cost of a spillway is quite high, if provided as a separate structure from the dam body, due to big design flood discharge of around 1,500 cu.m/sec.

From the above reasons, the concrete type dam is recommendable for the sites.

(Site LD)

A concrete diversion dam shall be applied for the Site LD depending on its facility function (pumping intake by low dam).

(Site EuM)

- Site EuM is almost located along the Site Eu axis.

- There are two rivers along Site Eu axis, Pinagsaluran river (9.8 sq.km watershed) which is located at the left portion and its tributary (4.1 sq.km watershed) located at the middle portion.
- The dam crest elevation of Site EuM will be less than the top elevation of the middle bank which exists between the two rivers mentioned above, in order to minimize the construction cost of facility.
- The spillway structure with concrete type will be provided on the left side river, Pinagsaluran river which forms a narrow valley with fresh bed rock under the condition of lower 45.0 m elevation, and while on the right river, the fill-type dam will be provided due to long crest length.

e) Freeboard, Slope and Crest Width

1) Freeboard

A 2.0 m freeboard from the high water level is adopted for the fill type dam to protect it from over topping, while 1.0 m is applied for the concrete type dam.

2) Slope of Dam

The results of embankment material investigation and laboratory tests indicate the properties of each material as follows;

(Core Materials)

The materials are composed of GC, SC, CH and MH in the unified soil classification, of which, GC and SC materials are predominant. The materials have such properties as 20 to 50% of field moisture content, 1.2 to 1.8 ton per cu.m of maximum dry density, 2 to 4 t per sq.m of cohesion and 26 to 30° of internal friction angle, $n \times 10^{6}$ to $n \times 10^{7}$ cm per sec of permeability coefficient, and 17 to 46% of plasticity index. Therefore, the materials are judged to be comparatively good with, such characteristic as high density, imperviousness, strong shearing strength, cohesiveness and easy construction.

(Random Materials)

The materials belong to GC, SC, SM and SW in the unified soil classification containing more sand and gravel particles than the core materials. The materials which have 10 to 30% of field moisture content and 1.4 to 1.9 ton per cu.m of maximum dry density are expected to be more strong than the core materials in the shearing strength.

(Filter Materials)

The properties of materials which can be borrowed from the river deposit of Pinagsaluran are GC to GW with 2.7 specific gravity in unified soil classification. Before banking the materials at the filter zone, clayey and silty materials shall be screened from the filter materials.

(Riprap Materials)

The boulders from the diluvial terrace along the Inagawan main river are used as riprap materials. Judging from the boring core samples which are classified to sand stone, amphibolite and peridotite with 2.5 to 3.1 ton per cu.m for bulk specific gravity, 0.7 to 8% of absorption and 2 to 6% of soundness, the quality of riprap materials will be equivalent with and/or more than the boring core samples.

Since the properties of embankment materials are considered to be good in addition to the weak earthquake force in the Study Area, the slopes of upstream and downstream of fill dam will be employed to be 1 to 2.80 and 1 to 2.30 respectively, referring to the following table.

Material Core Zone		Homogeneous		Zone type-1		Zone type-2	
		US	DS	US	DS	US	DS
GC,	GM	1:3.0	1:2.0	1:2.5	1:2.0	1:2.0	1:2.0
SC,	SM	1:3.0	1:2.0	1:2.5	1:2.0	1:2.0	1:2.0
CL,	ML	1:3.5	1:2.5	1:3.0	1:2.5	1:2.0	1:2.0
CH,	MH	1:4.0	1:2.5	1:3.5	1:2.5	1:2.0	1:2.0

Note: US : upstream slope
DS : downstream slope
Zone Type-1 : wide core type
Zone Type-2 : narrow core type
Material of random zone and other zone: GW, GP, SW, SP
Source : "Design of Small Dam", USBR

While regarding a concrete dam at the Inagawan river, the dam will receive large hydrostatic, hydrodynamics and silt pressures due to high elevation of sediment and dead water compared with its dam height. Based on the results of stability analysis under the middle third condition varying the upstream and downstream dam slope as an example (refer to Figure F.3.12), 1 to 0.20 of upstream slope and 1 to 0.80 of downstream slope are planned.

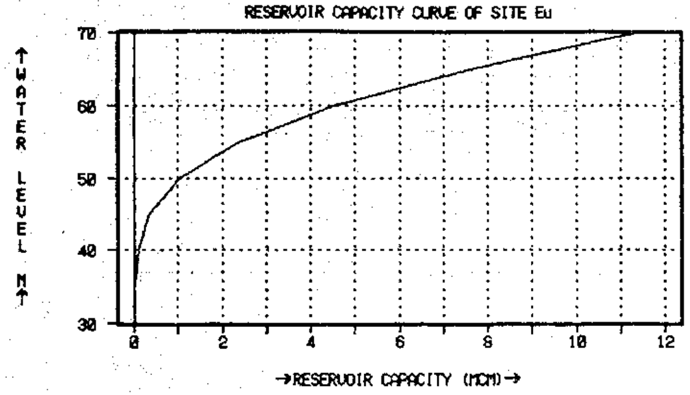
3) Dam Crest Width

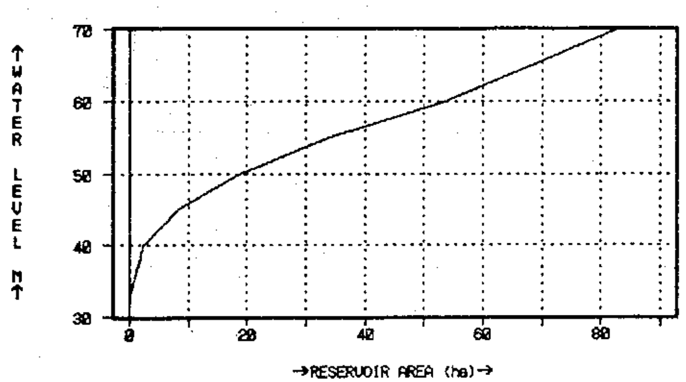
8.0 m of the crest width for a fill dam and 3.0 m for a concrete dam will be applied referring the previous studies and considering O & M.

F. 3. 2 Major Features

Based on the topographical maps with a scale of 1 to 1,000 and the design concepts described above, the preliminary design for each water resources site were carried out as shown in Table F.2.2 and attached drawings Dr-2 to Dr-6.

Figure F.3.1
Reservoir Capacity and Area Curve
(Site-Eu)

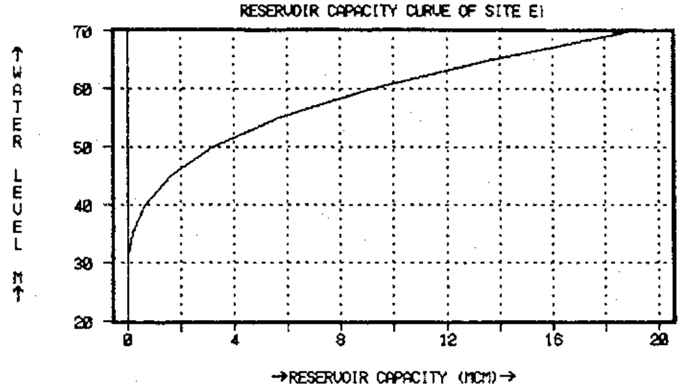


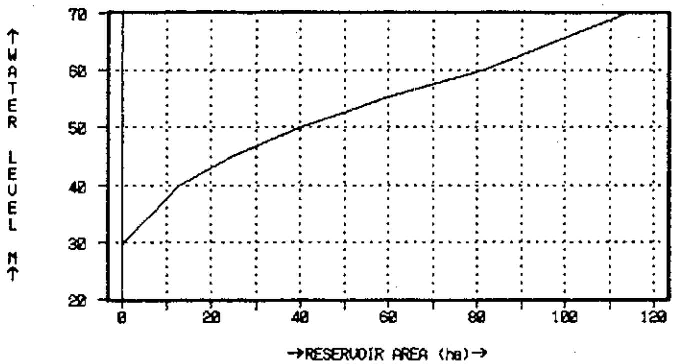


SITE Eu H-A.Q

WL(m)	H(m)	A(ha)	Av.A(ha)	Q(MCM)	Ac.Q(MCM)
33.00	0.00	0.00			0.000
35.00	2.00	0.74	0.37	0.007	0.007
40.00	5.00	2.48	1.61	0.081	0.088
45.00	5.00	8.34	5.41	0.271	0.358
50.00	5.00	18.66	13.50	0.675	1.033
55.00	5.00	34.10	26.38	1.319	2.352
60.00	5.00	53.68	43.89	2.195	4.547
65.00	5.00	68.25	60.97	3.048	7.595
70.00	5.00	82.68	75.47	3.773	11.368

Figure F.3.2
Reservoir Capacity and Area Curve
(Site-El)



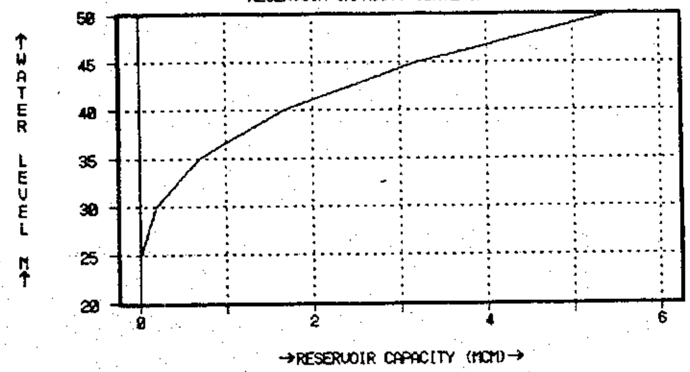


SITE El H-A.Q

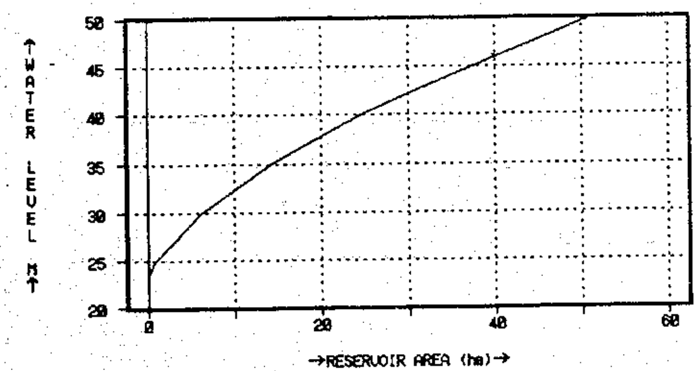
WL(m)	H(m)	A(ha)	Av.A(ha)	Q(MCM)	Ac.Q(MCM)
29.00	0.00	0.00			
30.00	1.00	0.36	0.18	0.002	0.002
35.00	5.00	6.65	3.51	0.175	0.177
40.00	5.00	12.89	9.77	0.488	0.666
45.00	5.00	24.32	18.60	0.930	1.596
50.00	5.00	40.51	32.42	1.621	3.216
55.00	5.00	59.01	49.76	2.488	5.704
60.00	5.00	81.50	70.26	3.513	9.217
65.00	5.00	97.67	89.59	4.479	13.697
70.00	5.00	113.77	105.72	5.286	18.983

Figure F.3.3
Reservoir Capacity and Area Curve
(Site-C)

RESERVOIR CAPACITY CURVE OF SITE C



RESERVOIR AREA CURVE OF SITE C

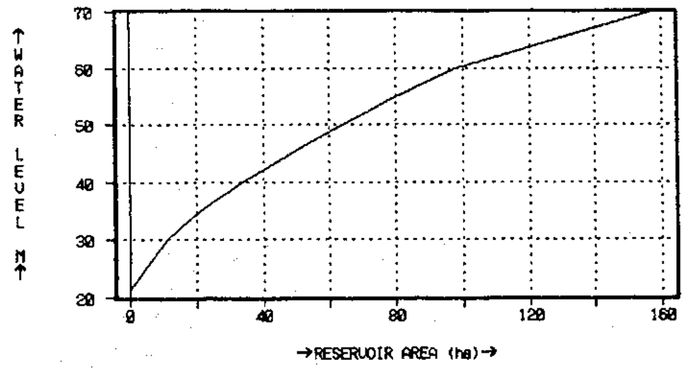


SITE C H-A.Q

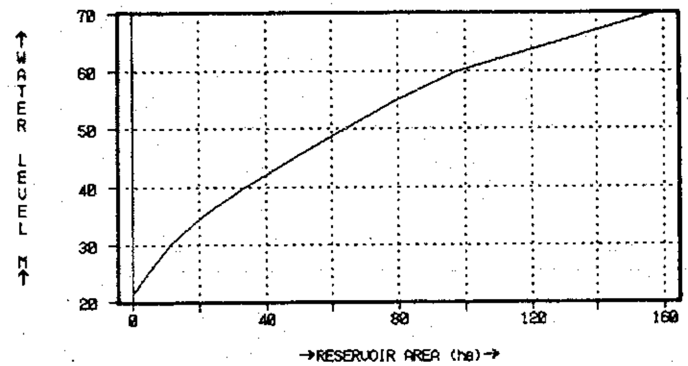
WL (m)	H (m)	A (ha)	Av. A (ha)	Q (MCM)	Ac. Q (MCM)
23.50	0.00	0.00			0.000
25.00	1.50	0.93	0.46	0.007	0.007
30.00	5.00	6.19	3.56	0.178	0.185
35.00	5.00	14.14	10.16	0.508	0.693
40.00	5.00	24.58	19.36	0.968	1.661
45.00	5.00	37.34	30.96	1.548	3.209
50.00	5.00	51.04	44.19	2.210	5.419

Figure F.3.4
Reservoir Capacity and Area Curve
(Site-D)

RESERVOIR AREA CURVE OF SITE D



RESERVOIR AREA CURVE OF SITE D

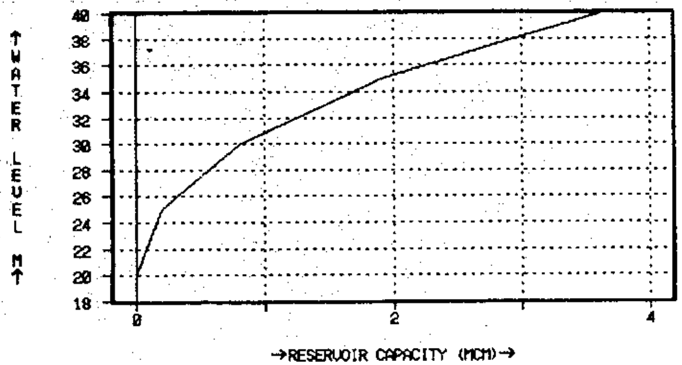


SITE D H-A.Q

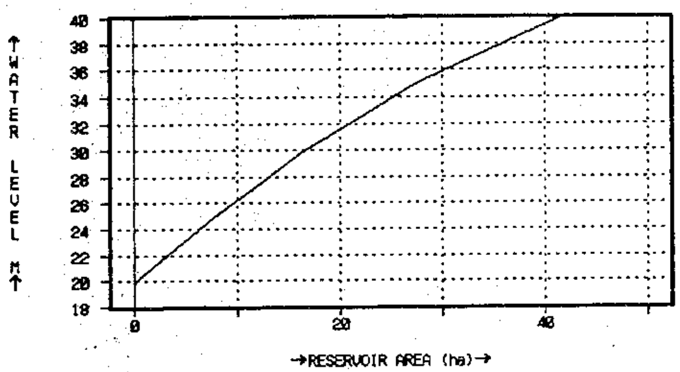
WL (m)	H (m)	A (ha)	Av. A (ha)	Q (MCM)	Ac. Q (MCM)
21.30	0.00	0.00			0.000
25.00	3.70	4.65	2.33	0.086	0.086
30.00	5.00	11.06	7.86	0.393	0.479
35.00	5.00	21.09	16.08	0.804	1.283
40.00	5.00	33.47	27.28	1.364	2.647
45.00	5.00	48.53	41.00	2.050	4.697
50.00	5.00	64.28	56.41	2.820	7.517
55.00	5.00	79.87	72.08	3.604	11.121
60.00	5.00	98.14	89.01	4.450	15.571
70.00	10.00	157.24	127.69	12.769	28.340

Figure F.3.5
Reservoir Capacity and Area Curve
(Site-LD)

RESERVOIR CAPACITY CURVE OF SITE LD



RESERVOIR AREA CURVE OF SITE LD

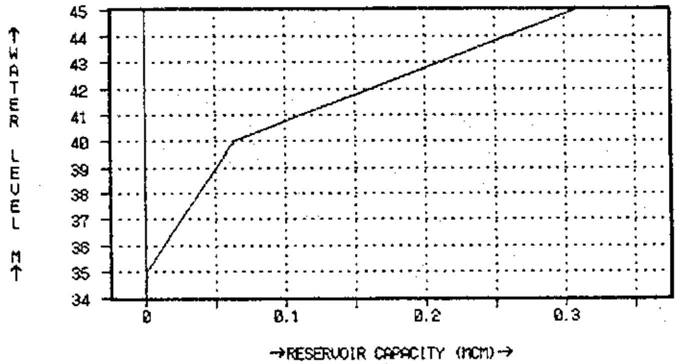


SITE LD H-A.Q

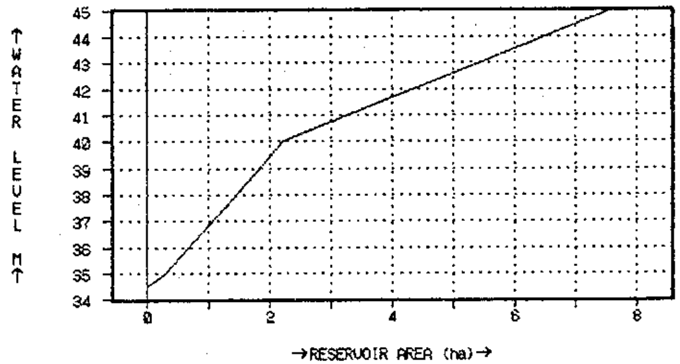
WL(m)	H(m)	A(ha)	Av.A(ha)	Q(MCM)	Ac.Q(MCM)
19.80	0.00	0.00			0.000
20.00	0.20	0.18	0.09	0.000	0.000
25.00	5.00	7.77	3.97	0.199	0.199
30.00	5.00	16.64	12.20	0.610	0.809
35.00	5.00	27.37	22.00	1.100	1.909
40.00	5.00	41.60	34.49	1.724	3.634

Figure F.3.6
Reservoir Capacity and Area Curve
(Site-EuM)

RESERVOIR CAPACITY CURVE OF SITE EuM



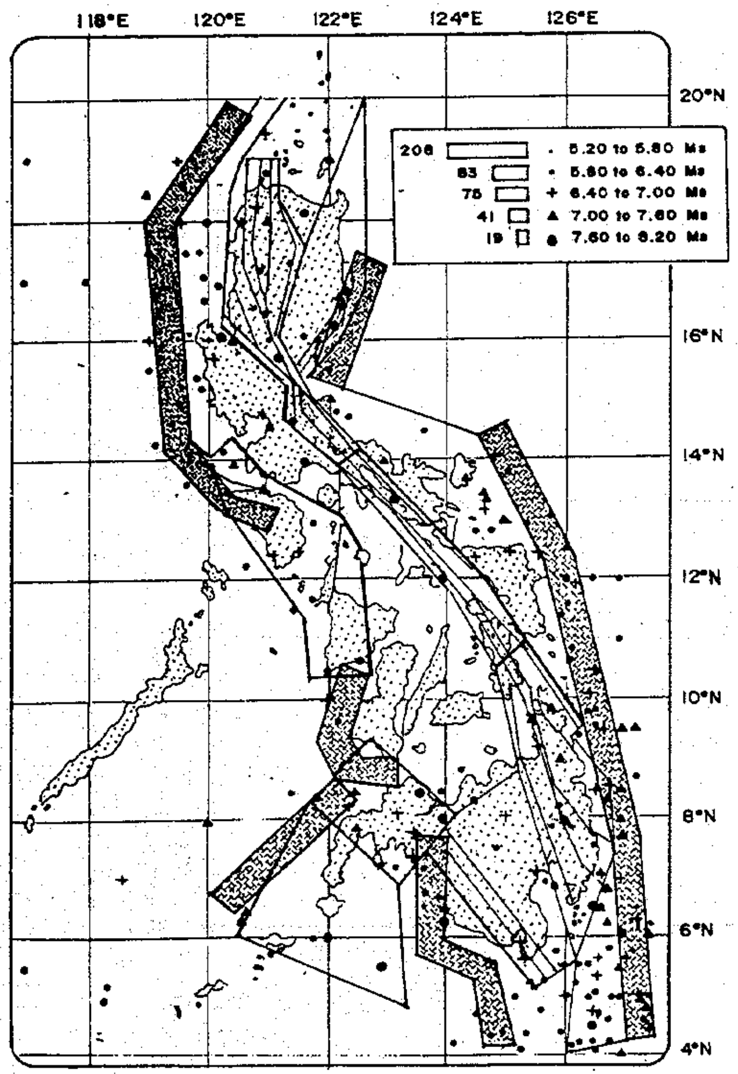
RESERVOIR AREA CURVE OF SITE EuM



SITE EuM H-A.Q

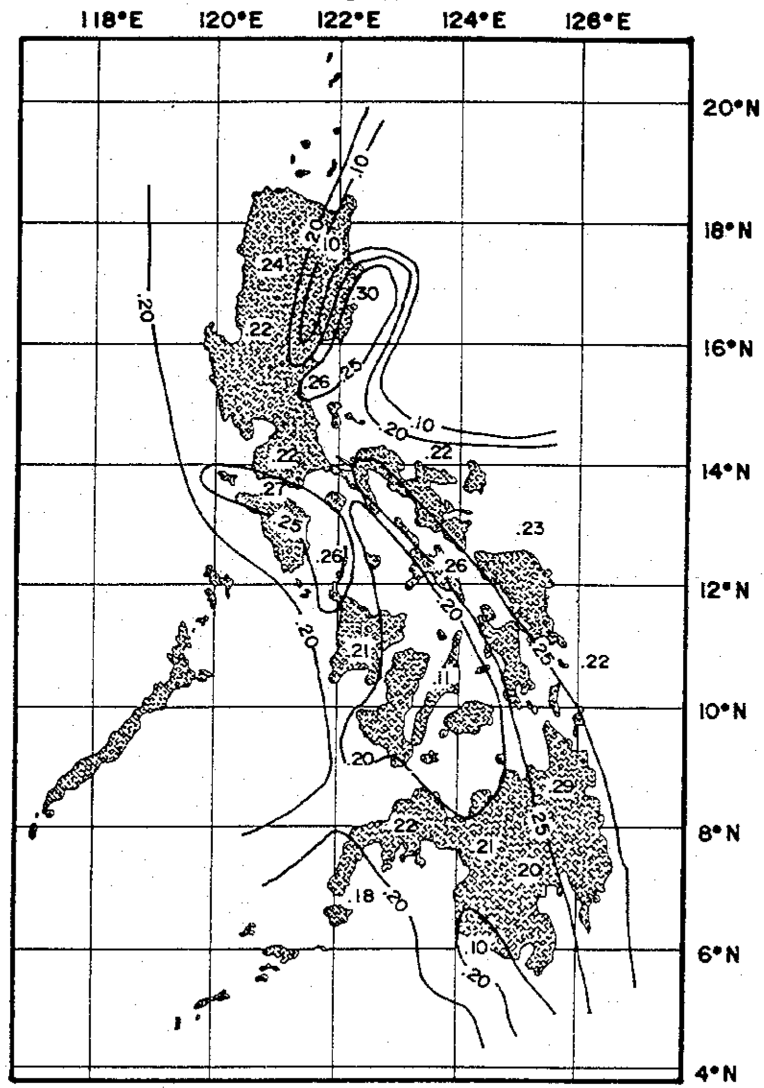
WL(m)	H(m)	A(ha)	Av.A(ha)	Q(MCM)	Ac.Q(MCM)
34.50	0.00	0.00			0.000
35.00	0.50	0.30	0.15	0.001	0.001
40.00	5.00	2.21	1.26	0.063	0.064
45.00	5.00	7.58	4.90	0.245	0.308

Figure F.3.7
Main Shock Earthquakes
in the Philippines



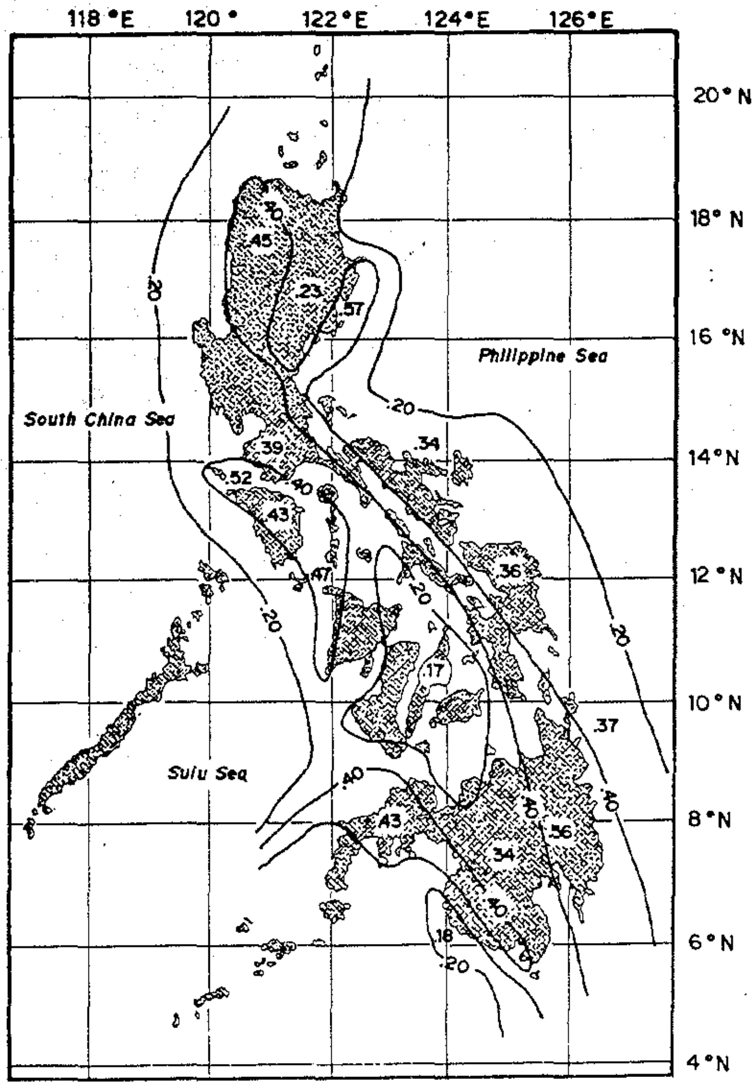
SOURCE : REPORT ON ESTIMATES OF THE REGIONAL GROUND-MOTION HAZARD IN THE PHILIPPINES (PHIVOLCS)

Figure F.3.8
Map of Peak Horizontal Acceleration of Gravity
(a 10 percent Provability of Exceedance in 50 Years)
in Rock



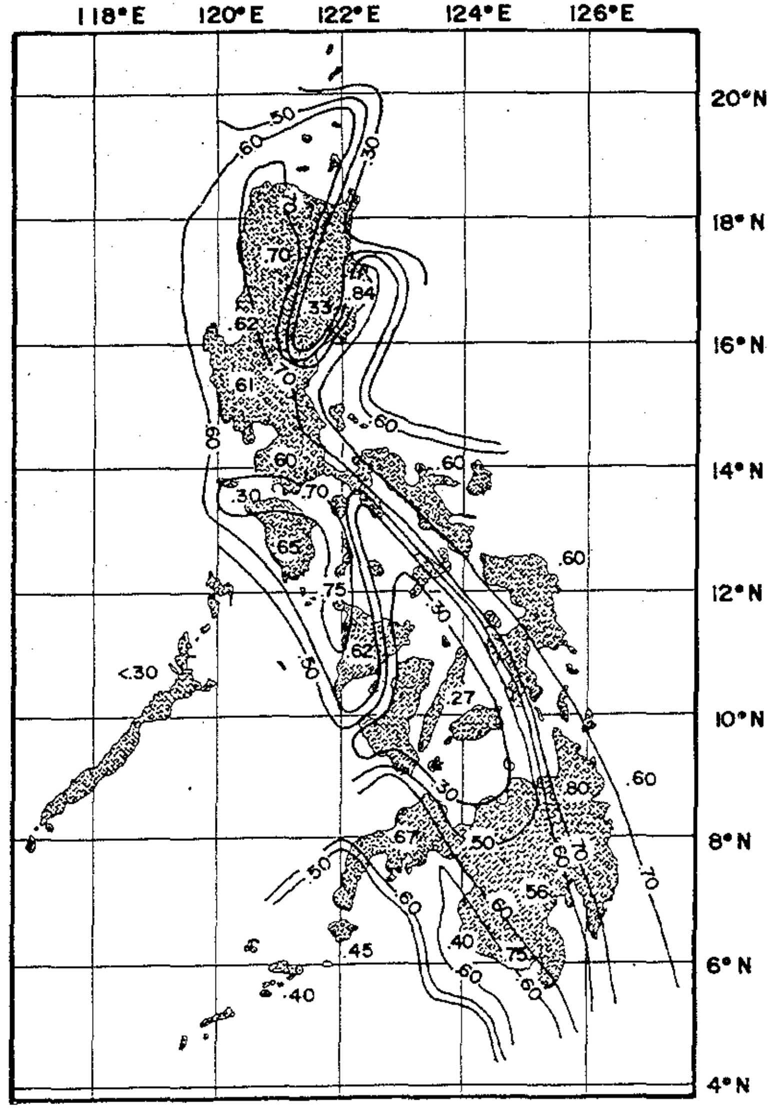
SOURCE : REPORT ON ESTIMATES OF THE REGIONAL GROUND-MOTION HAZARD IN THE PHILIPPINES (PHIVOLCS).

Figure F.3.9
Map of Peak Horizontal Acceleration of Gravity
(a 10 percent Provability of Exceedance in 50 Years)
in Medium Soil



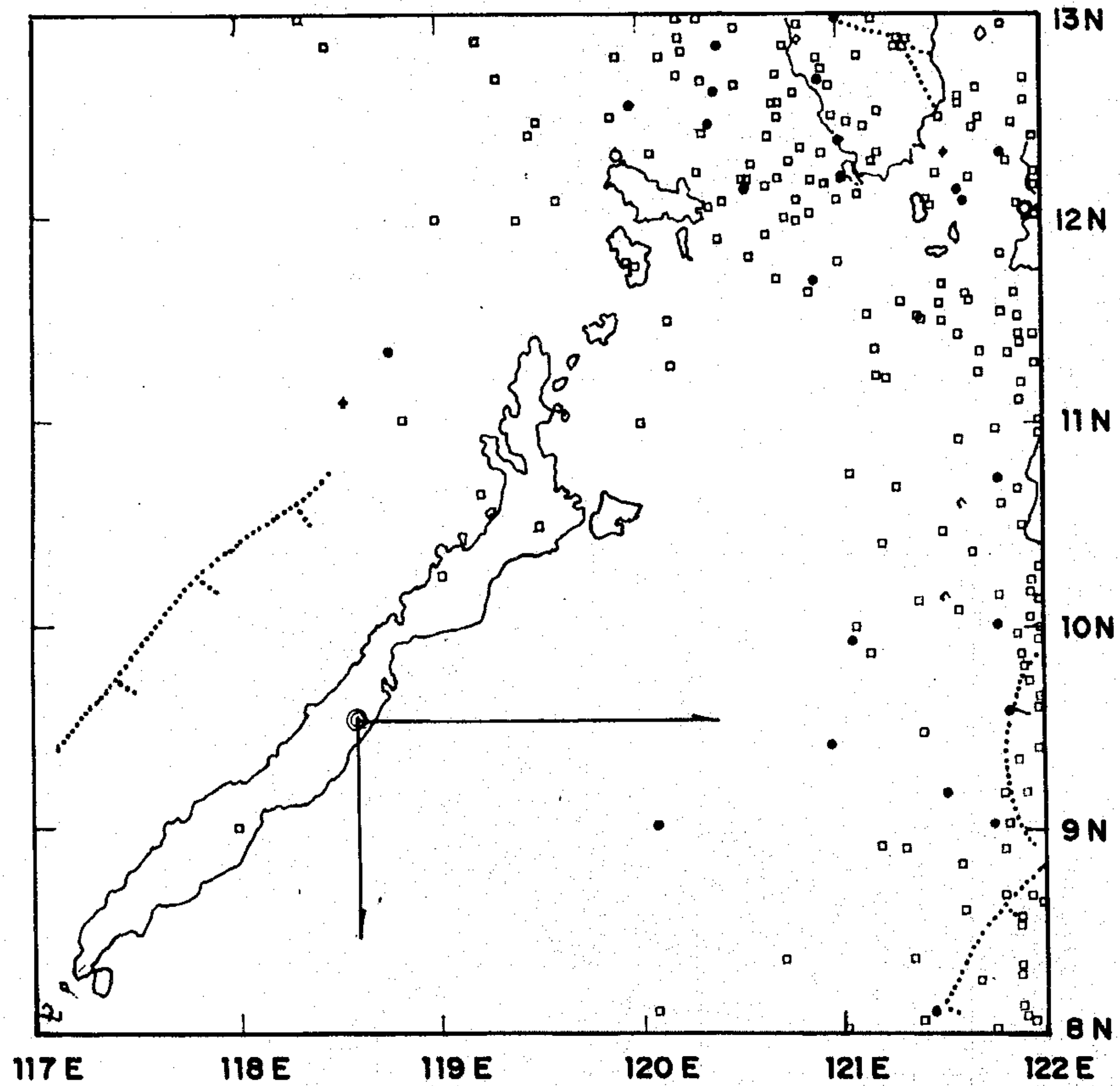
SOURCE : REPORT ON ESTIMATES OF THE REGIONAL GROUND-MOTION HAZARD IN THE PHILIPPINES (PHIVOLCS).

Figure F.3.10
Map of Peak Horizontal Acceleration of Gravity
(a 10 percent Provability of Exceedance in 50 Years)



SOURCE : REPORT ON ESTIMATES OF THE REGIONAL GROUND-MOTION HAZARD IN THE PHILIPPINES (PHIVOLCS).

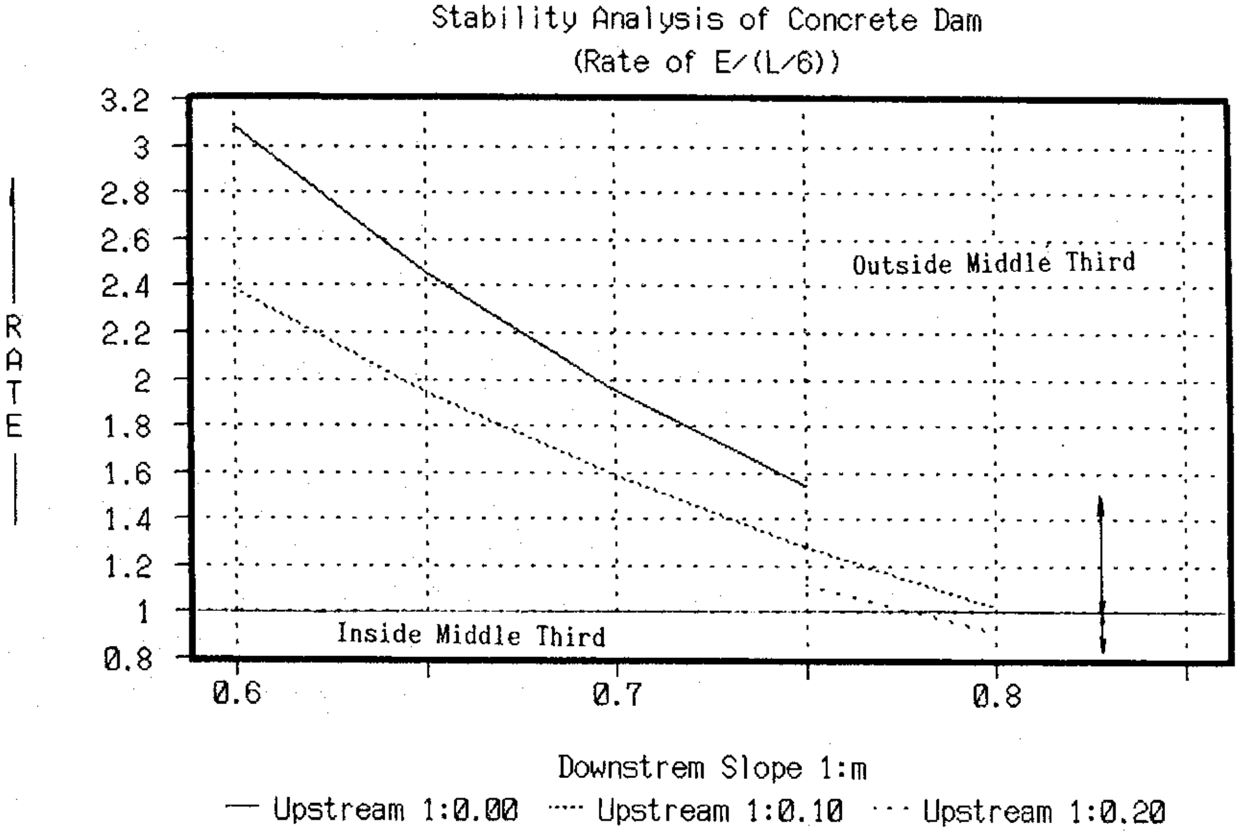
Figure F.3.11 Seismicity Map of Palawan Islands and Vicinity



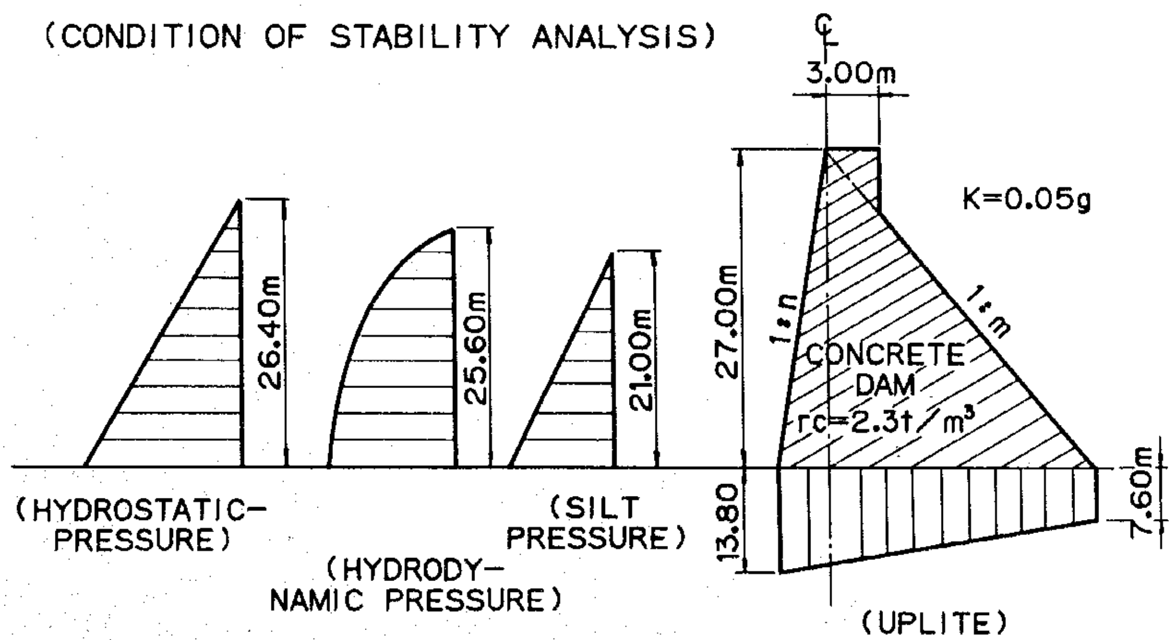
TIME: 1960-1993 MAGNITUDE: 3.4 DEPTH: 009

SOURCE: PHIVOLCS

Figure F.3.12 Example Stability Analysis of Concrete Dam



(Note) RATE=Ecentric Distance /Half of Middle Third Length



F. 4 WATER RESOURCES FACILITIES AND WATERSHED CONSERVATION

Since the island of Palawan is mostly covered by natural and dense forest, the Philippine Government executed laws and acts to protect these forests.

The population density of Palawan is small, hence, many people migrate to the province. There is a possibility that people will cut trees to manage and maintain their life. The irrigated agricultural plan proposed in this Study will contribute in the control of illegal cutting of trees and the conservation of the forest, since the proposed plan will directly increase agricultural productivity and income and indirectly conserve the land.

However, from the micro-point of view, the water resources facilities proposed (reservoir and traces of borrow pits, etc.) may affect the watershed environment. But because the scale of the proposed facilities are small, the effect to the environment will be minimal. To continually preserve and maintain the environment, it is necessary that the following countermeasures be taken into consideration. (refer to Table F.4.1)

Table F.4.1 Countermeasures for Watershed Conservation

Facility	Change of Conditions	Elements to be affected	Countermeasure
1) Reservoir	Submergency (8 ha in Stage I 29 ha in Stage II)	Forest, plants and insects	Detail investigation of submerged area Preserved measeures
	Driftwood and flouting sweepings	Water quality	O&M for clearing the obstructs
	River discharge	Discharge	Secure of river maintenance flow
	Unstable land slope of reservoir area	Topography & Geography	Detail geological investigation on detail design stage
2) Road	Bare ground of cutting portion	Scene	Vegetation and rehabilitaion of scene
3) Traces of Borrow pit area	Bare ground by cutting works	Scene	Vegetation and rehabilitaion of scene Conversion to farm land
Surrunding area	Bare ground by cutting works Unstability of land slope	Scene Topography and geolography	Vegetation and rehabilitaion of scene Stabilization measure for land slope

APPENDIX G. IRRIGATION AND DRAINAGE

G. 1 PRESENT CONDITIONS

G. 1. 1 Irrigation Condition

a) Present Situation

National irrigation development reached 49% of the 3.1 million ha potential irrigable land equivalent to 1,533,000 ha in 1992. Inadequate and untimely funding support restricts the faster development of irrigation system.

Irrigated cropping intensity averaged 136%. The Management Turnover Program is pursued with 1,370 Irrigators' Associations (IAs) now given contracts in systems management covering maintenance, Irrigation Service Fee (ISF) collection and turnover of whole or part of National Irrigation System (NIS). Collection efficiency of water charge on current account is averaged 43% in 1993, but 47% in 1992.

The priorities for the short, medium and long-term development programs are geared to support food production and enhance socioeconomic growth in the rural areas. The trusts in project development include accelerating the completion of on-going projects, adequate packaging of future projects and improved project management systems and practices. Thereby, emphasis will be on short gestating and medium scale projects while major projects will be started only when financial support is assured.

The targets of NIA's 10 years Development Plan (1993 - 2002) are the generation of 437,620 ha of new areas, rehabilitation of 586,680 ha in existing systems, minor restoration in 62,260 ha and reforestation of 24,000 ha in critical watersheds. The level of irrigation development will be about 57% by 1998 and 63% at the end of the plan period. Under the Plan, rice deficit is projected during the entire period, decreasing from 434,960 ton in 1993 to 139,500 ton in 2002, with the completion of on-going projects in 1993 and the impact of the new projects which would be felt beginning the middle of the decade.

b) Present Provincial Situation

Palawan province has 17,800 ha irrigation areas, equivalent to 33% of 54,500 ha potential irrigable land. These irrigation systems are composed of national, communal and individual pump irrigation systems under NIA, private systems and other government agencies. These facts indicate that Palawan has a low level of irrigation development, next to Mindanao Island.

However, under the SPIADP, a total of 15 communal irrigation projects will be constructed and five existing communal irrigation systems will be rehabilitated within the next four years thereby increasing the percentage of irrigation development of the province. About 410 ha irrigation service area, or about 10% of the total 4,370 ha irrigable land in Puerto Princesa City is generated. Puerto Princesa City may be one of the area having the lowest irrigation condition in the nation. About 350 ha out of 410 ha is located south of the national road forming southeast boundary of the Study Area. The features of said existing irrigation systems are as follows;

Project Name	Service Area	Household	Water Right	Planted Area (ha, in 1993) Wet Season	Dry Season
	(ha)		(lit/sec)		
Inagawan Sub-colony	80	Colony	100	80	60
Inagawan CIS	270	about 90	330	220	200

Note: Water source is the Inagawan river

c) The Study Area

In the Study Area, no irrigation system is available and rainfed farming is adopted except in certain areas where water is available from the three (3) natural springs with 2 to 3 lit/min yield.

d) Present Cropping Calendar

The major crops in the existing irrigation area are wet and dry season paddy. (refer to Figure G.1.1)

e) Approximate Unit water Requirement

Regarding water requirement of paddy, NIA's 7th Operation & Maintenance Plan which is prepared for the O&M plan of the existing systems in Narra, about 40 km southwestward from the Study Area based on the O&M activities shows the following:

	Dry Season	Wet Season
- Type of soil in the service area	clay loam	clay loam
- Soil saturation requirement	110 mm	110 mm
- Submergence requirement	50 mm	50 mm
- Percolation	4 mm/day	3.5 mm/day
- Evaporation	5 mm/day	5 mm/day
- Evapotranspiration	7.5 mm/day	7.5 mm/day
- Farm waste & delivery losses	30% of CWR	30% of CWR
- System efficiency	50%	45%

(Note) CWR : Crop Water Requirement

f) Results of Field Tests

For the purpose of determining the field percolation rate in the paddy land and intake rate in the upland, field tests were carried out during the dry and rainy season, by using the quick percolation rate measuring apparatus and cylinder infiltrometer, respectively. The following are the results;

(Percolation Test for Paddy Field)

Dry Season;

- Date	Feb. 1994
- Location	Farm lot No.44 in Tagumpay Settlement
- Meteorology	Fine weather, Temperature 33°C Breeze
- Type of soil	Loamy clay (dark brown)
- Percolation rate	3.1 mm/day (average)

Rainy Season;

- Date	Aug. 24, 1994
- Location	Existing paddy field
- Meteorology	Fine, cloudy later, Temp. 28 to 30°C
- Percolation rate	7.9 mm/day in average (0 to 14 mm/d)

(Intake Rate Tests for Upland)

Dry Season;

- Date: Feb. 20, 1994
- Meteorology: Fine weather, Temperature 33 - 35°C Breeze
- Location: Farm lot in Tagumpay settlement

	No.48	No.19	No.56
- Type of soil	clay loam w/gravel (light)	sandy loam (gray)	clay loam w/gravel (light brown)
- Test method	cylinder	cylinder	cylinder
- Basic intake rate (Ib)	18.4 (mm/day)	96.3 (mm/day)	20.4 (mm/day)

Rainy Season;

- Date: Sep. 5, 1994
- Location: 1) Western upper part of bombing range; 2) Near Kamuning along the highway
- Meteorology: Fine, cloudy later, Temp. 30°C

	Site 1)	Site 2)
- Type of soil	Silty loam	Sandy loam
- Basic intake rate	3.3 mm/day	3.3 mm/day

(refer to Table G.1.1 and Figure G.1.2 to G.1.4)

The percolation rates in the dry and rainy season were 3.1 and 2.0 mm/day, respectively, at the same place measured. These rates corresponded with the standard figure in Palawan area, but the average rate of 8.0 mm/day in the rainy season was higher than the ordinal figure in the paddy field, which may occur due to unconsolidated foundation of the land, that is, newly developed area.

With regard to the intake rate, surface irrigation method which is applicable for less than 50 mm/day percolation rate will be adopted on the clayey lands, but for the sandy lands the application of sprinkler irrigation method or other methods including perforated pipe irrigation method will be considered, as the rate is more than 75 mm per day.

g) Inagawan CIS O&M

The Inagawan CIS was constructed by NIA after receiving the water right for the project from NWRB on April 1985. In the initial stage, NIA assured the operation and maintenance works for the system. But in January 1993, the Inagawan-Kamuning Irrigators' Service Association organized by 90 farmers took charge of such works as collection of water charge, maintenance of on-farm facilities (cutting weeds and cleaning silt in canal) and informing the water demand schedule to NIA, while NIA is responsible for the operation and maintenance of the diversion dam including repair of the canal systems. The composition of the organization is as follows:

President	1 person,	Vice president	1 person
Secretary	1 person,	Board Member	5 persons
Ordinary Member	82 persons		

G.1.2 Drainage Condition

The Study Area has generally varying elevations from 5 to 100 m from the mountainous area toward the sea.

Four (4) major water courses of streams and creeks with direction of south-east, about 35 km in total length excluding the Inagawan and Pinagsaluran rivers, run across the area. These water courses have water flow in the rainy season although no and/or few flow can be observed in the dry season.

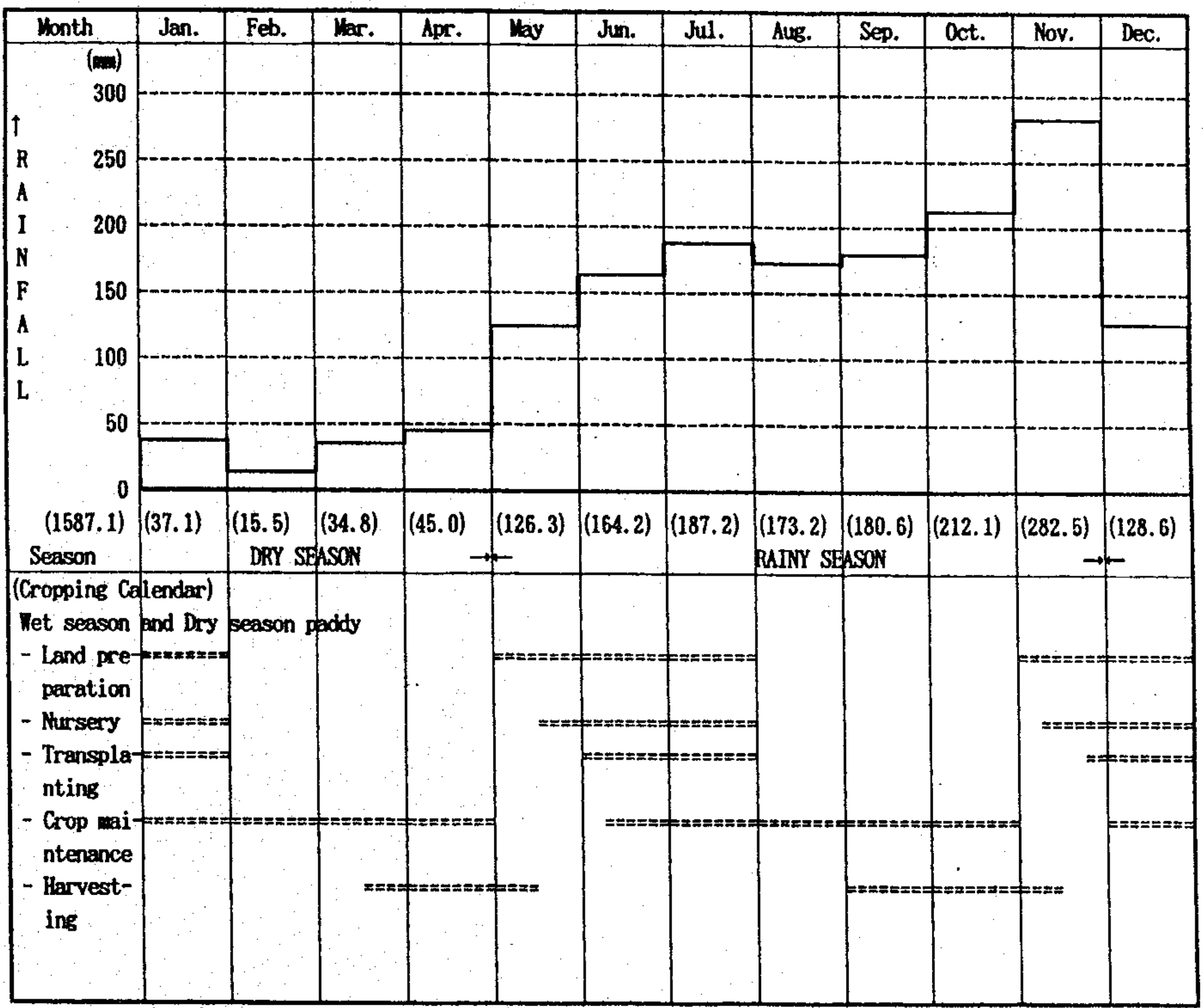
Smooth passage of vehicle is quite difficult during and/or after the rain in the Study Area due to its soil property, but two (2) or three (3) days after the rain stop, these conditions are improved. This fact shows that the existing rivers and creeks function sufficiently as drainage system in the Area. Therefore, these streams and creeks can be used as part of the drainage system in the Study Area.

Table G.1.1 **Percolation Tests at Paddy Field**

Date ;Aug.24,1994

Field No	Number	Reading Gauge Start (mm/day)	Reading Gauge End (mm/day)	Percolation (mm/day)
1	1st	276	290	14
	2nd	290	303	13
	3rd	303	315	12
	Av.			13.0
2	1st	62	64	2
	2nd	64	67	3
	3rd	67	68	1
	Av.			2.0
3	1st	23	26	3
	2nd	26	30	4
	3rd	30	33	3
	Av.			3.3
4	1st	151	159	8
	2nd	159	167	8
	3rd	167	172	5
	Av.			7.0
5	1st	313	370	57
	2nd	370	415	45
	3rd	415	451	36
	Av.			46.0
6	1st	57	74	17
	2nd	74	87	13
	3rd	87	100	13
	Av.			14.3
7	1st	79	87	8
	2nd	87	94	7
	3rd	94	100	6
	Av.			7.0
8-2	1st	48	62	14
	2nd	62	73	11
	3rd	73	83	10
	Av.			11.7
9	1st	103	121	18
	2nd	121	135	14
	3rd	135	147	12
	Av.			14.7
10	1st	134	144	10
	2nd	144	151	7
	3rd	151	153	2
	Av.			6.3
11	1st	17	18	1
	2nd	18	17	-1
	3rd	17	16	-1
	Av.			-0.3

Figure G.1.1 Present Cropping Calender



(Note)

Data source for cropping calendar : SPIADP

Rainfall : Aborlan rainfall (1977-1993)

Figure G.1.2 Location Map of Field Test

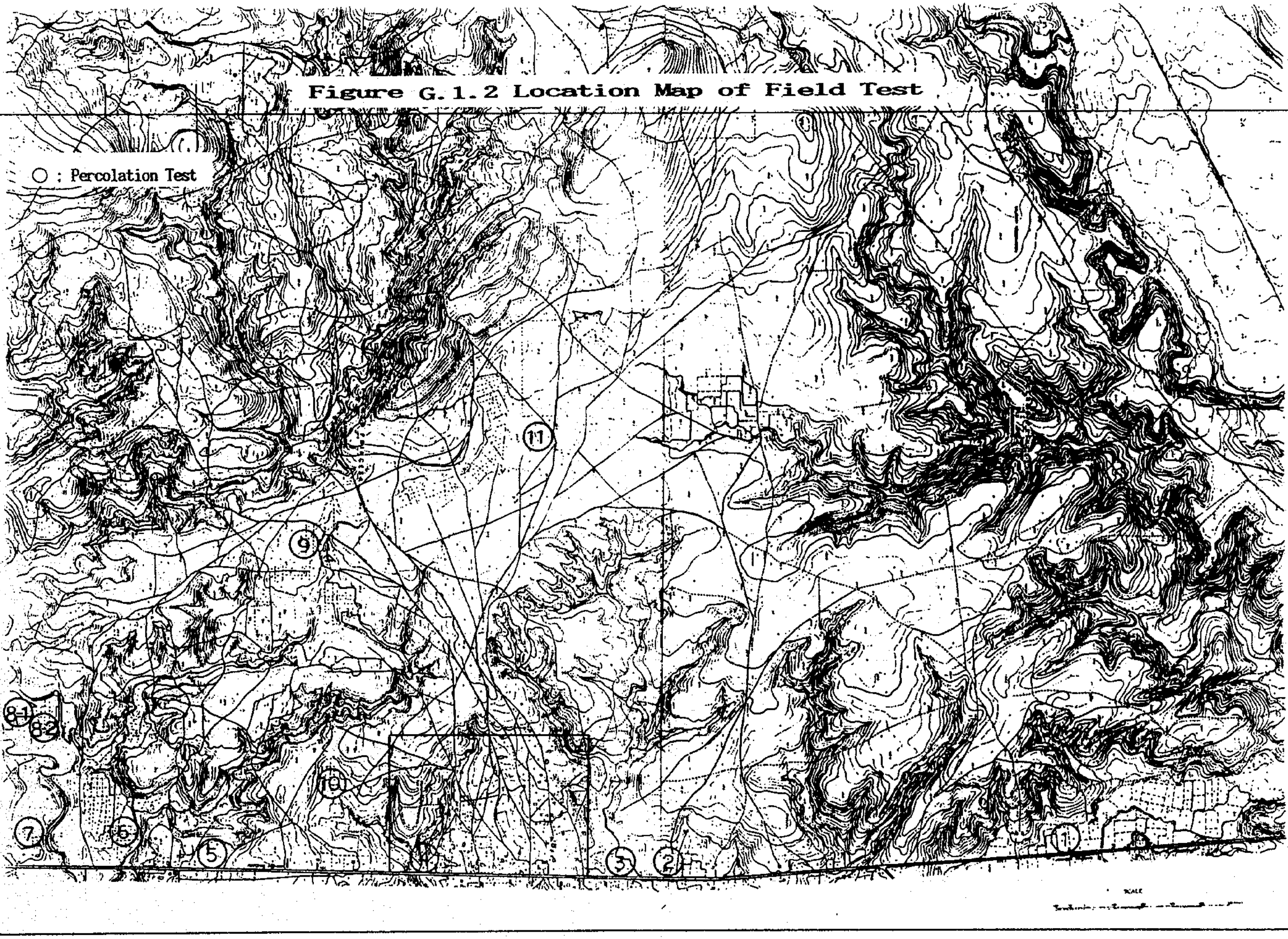
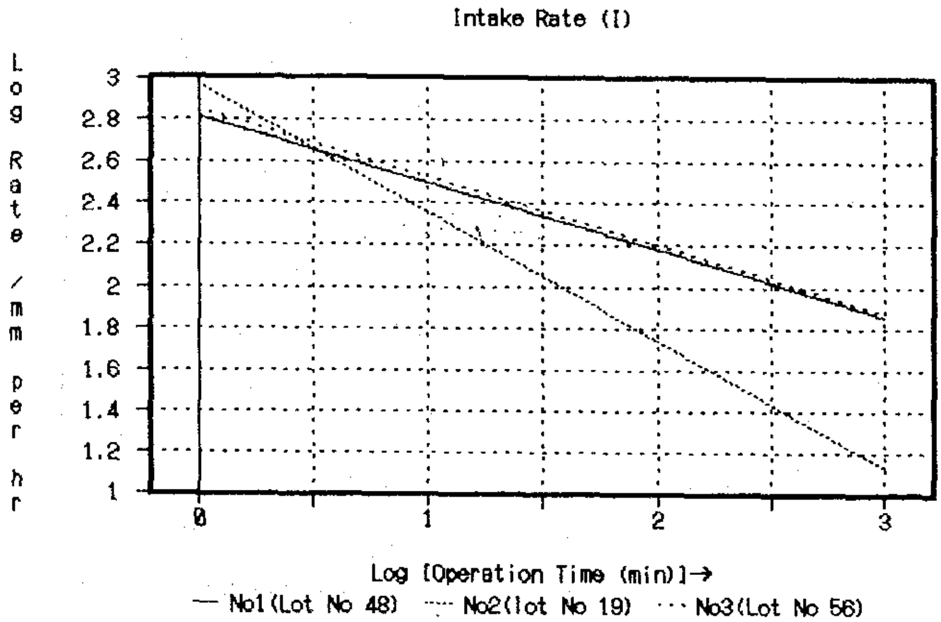


Figure G.1.3 Intake Rate Test in Dry Season



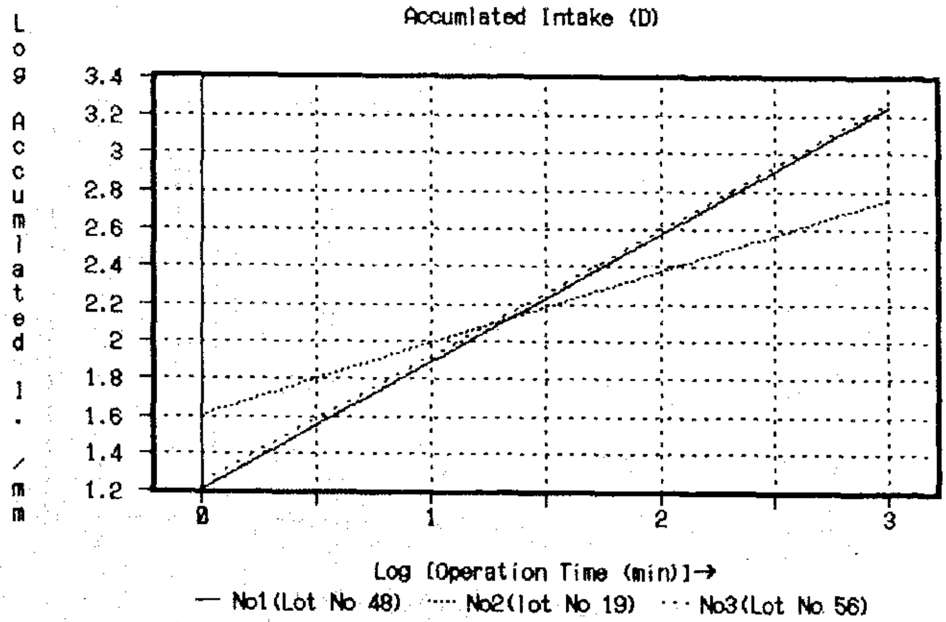
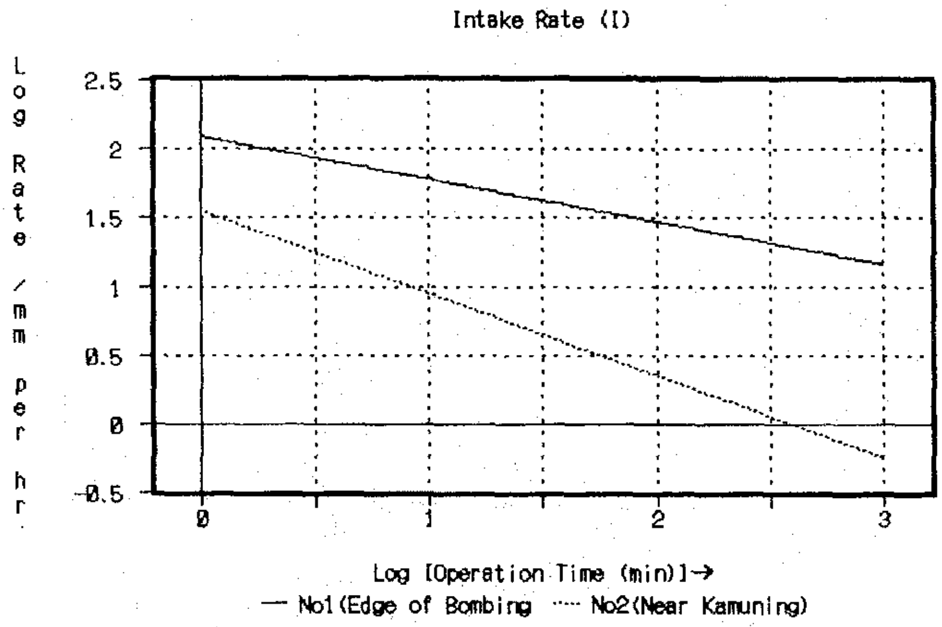
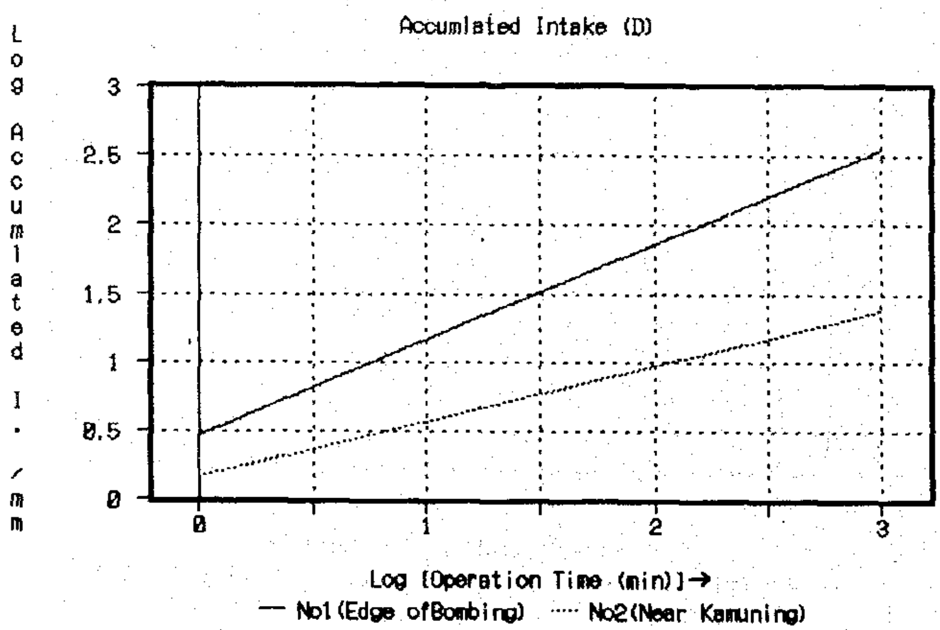


Figure G.1.4 Intake Rate Test in Rainy Season





G. 2 DEVELOPMENT PLAN

G. 2. 1 General

Out of about 2,000 ha of the total Study Area which relies on the rainfed farming without any irrigation facilities, only 80.5 ha including paddy field of 48.2 ha are utilized as agricultural farm land. The results of inquiry survey conducted in Work-1 stage indicate that the introduction of irrigation facilities is a priority requirement for the farmers. Therefore, for the water resources development, an irrigation component is essentially required for the agricultural development in the Study Area.

Regarding the irrigation development plan in the Study Area, staple and cash crops such as paddy, upland crops and vegetables are proposed to be irrigated. The fruit trees will be planted in rainfed areas due to restrictions as to topography and elevation conditions.

Proposed irrigation component includes the water resources and irrigation system up to 30 to 50 ha block, and on-farm facilities for effective utilization of the system.

Generally, furrow irrigation method will be adopted for upland field crops and vegetables, but in unapplicable areas surface irrigation with portable pump may be required.

G. 2. 2 Irrigation Water Requirement

The diversion water requirement (DWR) for irrigation is estimated based on the proposed two cropping calendars, Type I (Wet season paddy + Dry season upland crop, vegetable) and Type II (Full season upland crop and vegetable). DWR is the total amount of water diverted from a source for evapotranspiration, percolation, field loss, conveyance loss and operation loss less effective rainfall in the field. In addition to the water for growing period mentioned above, water for nursery beds and land preparation is required for paddy cultivation.

DWR is estimated by the following formula;

- Net water requirement = Crop consumptive use + Percolation +
  (NWR) Water requirement for land preparation
- Diversion water requirement = Net water requirement - Effective rainfall +
  (DWR) Losses (Field, Conveyance and Operation)

a) Crop Consumptive Use (Cu)

The estimate for crop consumptive use was conducted based on the FAO Irrigation and Drainage Paper No.24 owing to no available data observed.

Where, Crop consumptive use (Cu) = Evapotranspiration (ETr) × Crop factor(Kc)

The evapotranspiration was estimated based on the Aborlan climate data (PAGASA, 17 years duration from 1977 to 1993) by applying the modified Penman Method as follows:

Evapotranspiration (ETR)

Month	1	2	3	4	5	6	7	8	9	10	11	12
Kc	3.6	4.1	5.2	5.4	4.7	3.8	3.7	3.8	3.8	3.6	3.6	3.7

The crop factors for main crops were employed as follows:

10d	1	2	3	4	5	6	7	8	9	10	11	12
Pad.	1.08	1.10	1.10	1.08	1.05	1.02	0.98	0.93	0.88			
V. A	0.52	0.60	0.77	0.97	1.05	1.08	1.07	0.97	0.75			
V. B	0.53	0.54	0.57	0.62	0.72	0.83	0.88	0.91	0.92	0.89	0.84	0.78
Bean	0.52	0.58	0.67	0.80	0.94	1.02	1.07	1.05	0.93	0.65	0.30	
Corn	0.49	0.56	0.67	0.89	1.02	1.08	1.07	1.00	0.78			

Note *10d* : *10 days* *Pad.*: *Paddy*
V. A : Vegetable A (Tomato and others)
V. B : Vegetable B (Watermelon and others)

b) Percolation

The results of field test for the representative paddy field in the Study Area were 3.1 mm/day in the dry season and 2.0 to 3.0 mm/day in the rainy season. Therefore the field percolations for dry and wet season will be designed at 3.0 and 2.5 mm/day, respectively.

c) Water for Land Preparation

Soil saturation and submergence water during land preparation period in the paddy field are required for continuous 30 days before plantation. Further, the evaporation and percolation requirement during these period shall be taken into account.

Item	Dry Season (Dec. to May)	Wet Season (Jun. to Nov.)
- Soil saturation requirement	110 mm	110 mm*1
- Submergence requirement	50 mm	50 mm*1
- Evaporation	4.9 mm/day	3.4 mm/day*2
- Percolation	3.0 mm/day	2.5 mm/day
Total	397 mm (400 mm)	337 mm (340 mm)

(Note) *1 : Based on 7th Operation & Maintenance Plan by NIA System in Palawan
*2 : Based on Aborlan climate data (average evaporation)

Consequently 400 and 340 mm of land preparation water are adopted for the dry and wet season respectively. The area of nursery bed will be 5% of the paddy field and seedling be made for 30 days before plantation of paddy. The water requirement for seedling is included with the land preparation water described above.

d) Effective Rainfall

The effective rainfall is the quantity of rain effectively used in the irrigation service area. Since there are no available data observed concerning the effective rainfall, the estimate of effective rainfall is made in accordance with NIA's guideline for planning and design, such that the effective rainfall shall be less than 80 mm per 10 days.

e) Irrigation Efficiency

A part of the irrigation water will be lost during the conveyance and operation from the source to the field, and lost in the field at a rate depending upon irrigation method and field conditions. Referring to the NIA's guideline for planning and design, and 7th Operation and Maintenance Plan, the irrigation efficiency is established as follows:

Items	Irrigation	Efficiency
Efficiency	For Paddy	For Vegetable & Upland Crops
Field	0.80	0.70
Conveyance	0.80	0.80
Operation	0.80	0.80
Overall	0.50	0.45

f) Unit Net Water Requirement and Diversion Water Requirement

The following three (3) cropping patterns are introduced to the beneficial area.

Type	Irrigation	Efficiency
Type 1	Paddy	Vegetable & Upland
Type 2	Vegetable and Upland crops (Full season)	
Type 3	Tree crops(Full season)	
(The former two (2) types are to be irrigated)		

The net water requirement, and average diversion water requirement per 1,000 ha considering effective rainfall and irrigation efficiency, for the two (2) typical cropping patterns are as follows;

Net Water Requirement and Average Diversion Water Requirement

(Unit: MCM/1,000 ha)

Month	Rainfall	Type 1		Type 2	
		N. W	D. W	N. W	D. W
Jan.	0.35	0.96	1.60	1.13	1.97
Feb.	0.15	1.12	2.18	0.67	1.21
Mar.	0.33	0.64	1.18	0.08	0.13
Apr.	0.44	0.02	0.02	-	-
May	1.25	-	-	-	-
Jun.	1.63	1.91	1.44	0.32	0.03
Jul.	1.88	2.48	1.13	0.92	0.28
Aug.	1.73	2.02	1.26	1.17	0.48
Sep.	1.81	1.66	0.86	0.63	0.14
Oct.	2.12	0.26	0.00	0.04	0.00
Nov.	2.83	-	0.00	0.31	0.02
Dec.	1.29	0.49	0.45	0.92	0.97
Total	15.81	11.62	10.12	6.19	5.23

Note: N.W: Net water requirement
D. W: Diversion water requirement

G. 2. 3 Irrigation Area

a) Irrigable Area

Since the Study Area has undulated topography, the area of steep slope and water resources sites are omitted from the proposed beneficial area as reported in Appendix D.2 Land Use.

The area above 40 m MSL mostly consists of lands with steep slope of more than 8% because of it's topographical condition. The irrigable area therefore shall be selected below 40 m MSL. Based on the topographical map with a scale of 1 to 4,000, the canal alignment was proposed. Such areas as more than 8 % of land slope land and unsuitable area for agricultural farm shall be subtracted from the applicable area of irrigation below 40 m MSL, resulting in the irrigable area of 590 ha in gross as shown below;

Applicable area for irrigation	895 ha	
* Irrigable area	590 ha	
- Type 1 crops	430 ha	(Wet season paddy + Dry season vegetable and upland)
- Type 2 crops	160 ha	(Vegetable and Upland)
* Non-applied area	305 ha	
- Type 3 crops	90 ha	(Tree crop)
- Forest	215 ha	

b) Average Diversion Water Requirement

Based on the irrigable area mentioned above, the average diversion water requirements are estimated in order to formulate the water resources development. The net irrigable area is employed to be equivalent to 90% of the gross irrigable area subtracting lands for road and irrigation systems. The average annual diversion water requirements are as follows;

	Cropping Pattern		
	Type 1	Type 2	Total
(1) Net Area (ha)	387	144	531
(2) Average annual D.W.R.			
• 200% of Cropping Intensity			
Unit Water Requirement (MCM/1,000 ha)	10.12	5.23	
Total Water Requirement (MCM)	3.92	0.75	4.67
• 150% of Cropping Intensity			
Unit Water Requirement (MCM/1,000 ha)	7.41	3.09	
Total Water Requirement (MCM)	2.87	0.44	3.31

G. 2. 4 Drainage Plan

a) General

The removal of excess irrigation water and rainfall from the soil surface is necessary to prevent crop damage. The drainage plan will be made with the following concepts.

- The natural streams and rivers in the Study Area shall be utilized for drainage system as much as possible.
- The drainage canals will be non-lined in general.
- The capacity of drainage canal will be designed under the conditions with runoff coefficient of 80 % and two days drainage period for the maximum daily rainfall with a five years return period referring to the NIA's planning guide line.
- The density of drainage canal in the irrigation area will be about 20 m/ha.

b) Drainage Modulus

In accordance with the concepts described above, the design drainage modulus is determined as follows;

Design rainfall	:	138.6 mm/day (Probably rainfall with a return period of five (5) years)
Drainage modulus	:	80 % of the design rainfall for 2 days drain = 55.4 mm/day (= 6.4 lit/sec/ha)

Table G.2.1 Net Water Requirement (Wet Season Paddy)

Month	Jun.			Jul.			Aug.			Sep.			Oct.		
10 days	1	2	3	1	2	3	1	2	3	1	2	3	1	2	3
Cropping Pattern															
Element															
- Accumlated days	10	20	30	40	50	61	71	81	92	102	112	122	132	142	153
- Irrigating days				10	20	31	41	51	62	72	77				
- Crop Factor(Kc)				1.08	1.10	1.10	1.08	1.05	1.02	0.98	0.93	0.88			
					1.08	1.10	1.10	1.08	1.05	1.02	0.98	0.93	0.88		
						1.08	1.10	1.10	1.08	1.05	1.02	0.98	0.93	0.88	
							1.08	1.10	1.10	1.08	1.05	1.02	0.98	0.93	0.88
Average Kc				1.08	1.09	1.09	1.09	1.08	1.06	1.03	1.00	0.95	0.93	0.91	0.88
- ETr (mm/day)				3.7	3.7	3.7	3.8	3.8	3.8	3.8	3.8	3.8	3.6	3.6	3.6
- ETc (mm/day)	0.00	0.00	0.00	4.00	4.03	4.05	4.14	4.11	4.04	3.92	3.78	3.62	3.35	3.26	3.17
- Percolation (mm/day)	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5
- ETc+P. (mm/day)	2.50	2.50	2.50	6.50	6.53	6.55	6.64	6.61	6.54	6.42	6.28	6.12	5.85	5.76	5.67
Land Preparation															
- ETr (mm/day)	3.4	3.4	3.4	3.4	3.4	3.4									
- Percolation (mm/day)	2.5	2.5	2.5	2.5	2.5	2.5									
- ETc+P. (mm/day)	5.9	5.9	5.9	5.9	5.9	5.9									
Initial Leaching (mm)			110												
Submergency (mm)			50												
Equation															
- Initial Leaching	0.215	0.215	0.224	0.141	0.098	0.107									
- Land Preparation	0.172	0.484	0.797	0.825	0.516	0.188									
- Normal Irrigation				0.172	0.484	0.813	1.000	1.000	1.000	1.000	1.000	0.672	0.359	0.047	
Water Requirement															
- Initial Leaching	34.40	34.40	35.84	22.56	15.68	17.12									
- Land Preparation	10.15	28.56	47.02	48.68	30.44	12.20									
- Normal Irrigation	0.00	0.00	0.00	11.17	31.62	58.53	66.42	66.14	71.91	64.24	62.81	41.12	20.99	2.71	0.00
NWR (mm/10 days)	44.5	63.0	82.9	82.4	77.7	87.9	66.4	66.1	71.9	64.2	62.8	41.1	21.0	2.7	0.0
NWR (mm/month)	190.4			248.0			204.5			168.2			23.7		

(Note) NWR : Net Water Requirement

Table G.2.2 Net Water Requirement (Dry Season Bean)

Month	Nov.			Dec.			Jan.			Feb.			Mar.		
10 days	1	2	3	1	2	3	1	2	3	1	2	3	1	2	3
Cropping Pattern															
Element															
- Accumlated days		5	15	25	35	46	56	66	77	87	97	105	115	125	136
- Growing days		5	15	25	35	46	56	66	77	87	97	105			
- Crop Factor (Kc)		0.52	0.58	0.67	0.80	0.94	1.02	1.07	1.05	0.93	0.65	0.30			
			0.52	0.58	0.67	0.80	0.94	1.02	1.07	1.05	0.93	0.65	0.30		
				0.52	0.58	0.67	0.80	0.94	1.02	1.07	1.05	0.93	0.65	0.30	
					0.52	0.58	0.67	0.80	0.94	1.02	1.07	1.05	0.93	0.65	0.30
Average Kc		0.52	0.55	0.59	0.64	0.75	0.86	0.96	1.02	1.02	0.93	0.73	0.63	0.48	0.30
- ETr (mm/day)	3.6	3.6	3.6	3.7	3.7	3.7	3.6	3.6	3.6	4.1	4.1	4.1	5.2	5.2	5.2
- ETc (mm/day)	0.00	1.87	1.98	2.18	2.38	2.77	3.09	3.45	3.67	4.17	3.79	3.00	3.26	2.47	1.56
- Percolation (mm/day)	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
- ETc+P. (mm/day)	0.00	1.87	1.98	2.18	2.38	2.77	3.09	3.45	3.67	4.17	3.79	3.00	3.26	2.47	1.56
Equation															
- Normal Irrigation	0.000	0.096	0.339	0.661	0.952	1.000	1.000	1.000	1.000	1.000	1.000	1.000	0.887	0.532	0.194
Water Requirement															
- Normal Irrigation	0.00	0.90	6.71	14.43	22.63	30.42	30.87	34.47	40.39	41.72	37.93	24.03	28.90	13.14	3.33
NWR (mm/10 days)	0.0	0.9	6.7	14.4	22.6	30.4	30.9	34.5	40.4	41.7	37.9	24.0	28.9	13.1	3.3
NWR (mm/month)		7.6			67.5			105.7			103.7			45.4	

(Note) NWR ; Net Water Requirement

G. 3 Areal Rotational Irrigation System

In the first stage development, the capacity of the proposed water resources facility for farming will be a cropping intensity of 130%. That is, during the dry season, only one third of the irrigation area of 590 ha can be irrigated. The remaining two-thirds can not be planted with any crop under irrigation condition.

In order to distribute equitable benefits to the beneficiaries, the areal rotational irrigation system shall be proposed in the irrigation area. The Irrigators Association (IA), which will be trained and assisted by NIA, is proposed to implement the areal rotational irrigation system and will be executed based on the following procedures.

The area will be divided into three (3) blocks, namely, Block-A with an area of 193.0 ha, Block-B, with an area of 196.6 ha, and Blocked-C, with an area of 200.4 ha, based on the irrigation canal system and topographical condition. (refer to Figure G.3.1)

In the first year, all the irrigable areas can be irrigated during the wet season. However, during the dry season, only one-third of the area can be irrigated due to water resources limitation. During this period, only Block-A irrigation area could be irrigated. The other Blocks (B and C), can not be irrigated, thus no farm output will be produced.

In the second year, all the irrigable areas can be irrigated during the wet season. However, during the dry season, only Block-B irrigation area will enjoy irrigation water. The other two (2) Blocks (A and C) will not get irrigation water for cropping.

In the third year, again, all the irrigable areas can be irrigated during the wet season, however, during the dry season, only Block-C area will avail of irrigation water.

During the fourth year, the areal rotational irrigation system will return to the first year procedure of water allocation. For the succeeding years, the rotational procedure presented beforehand shall be followed and maintained. (refer to Figure G.3.2)

Figure G.3.1 Block Area of Areal Rotational Irrigation System
(in case of cropping intensity 130%, on Stage I Development)

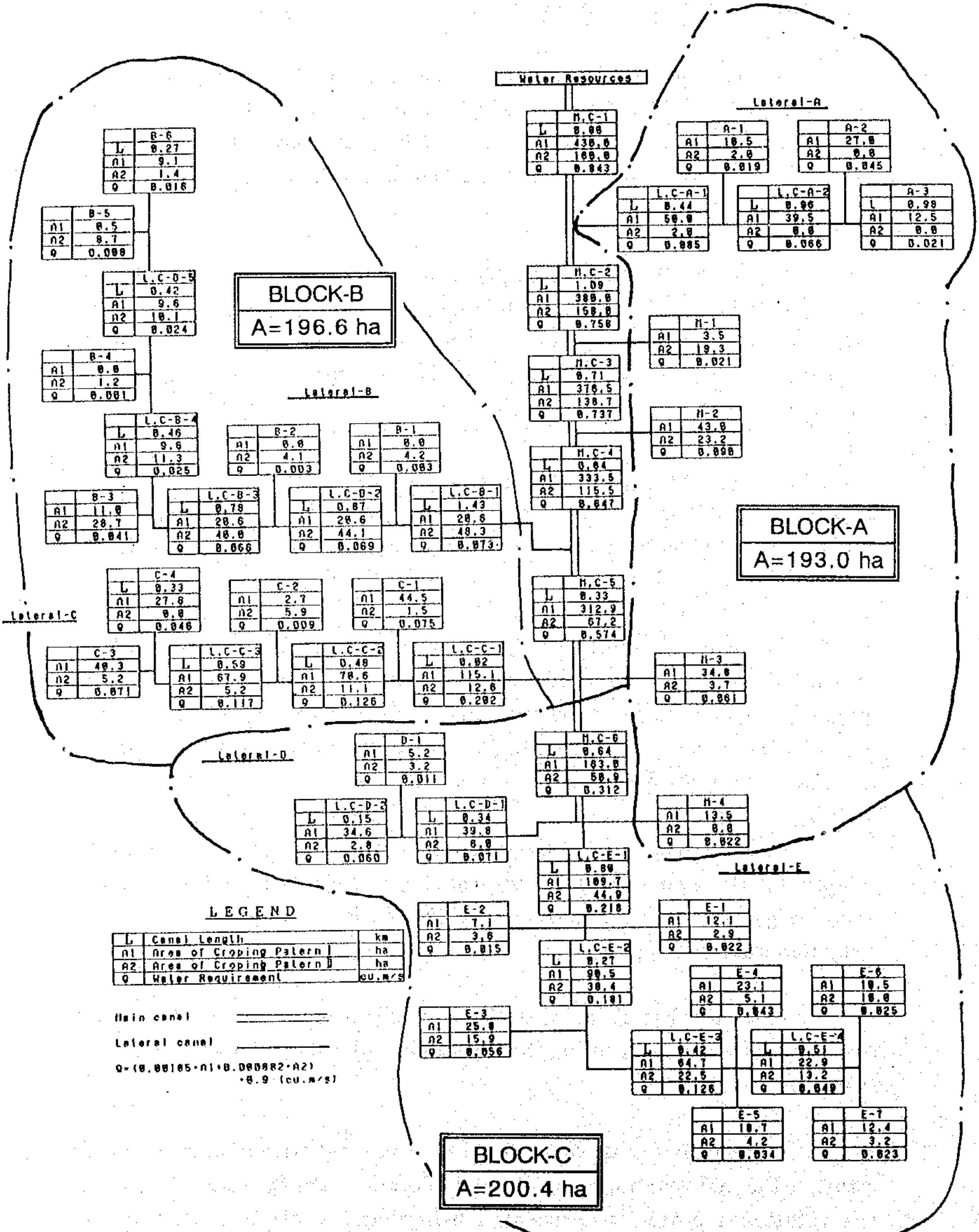
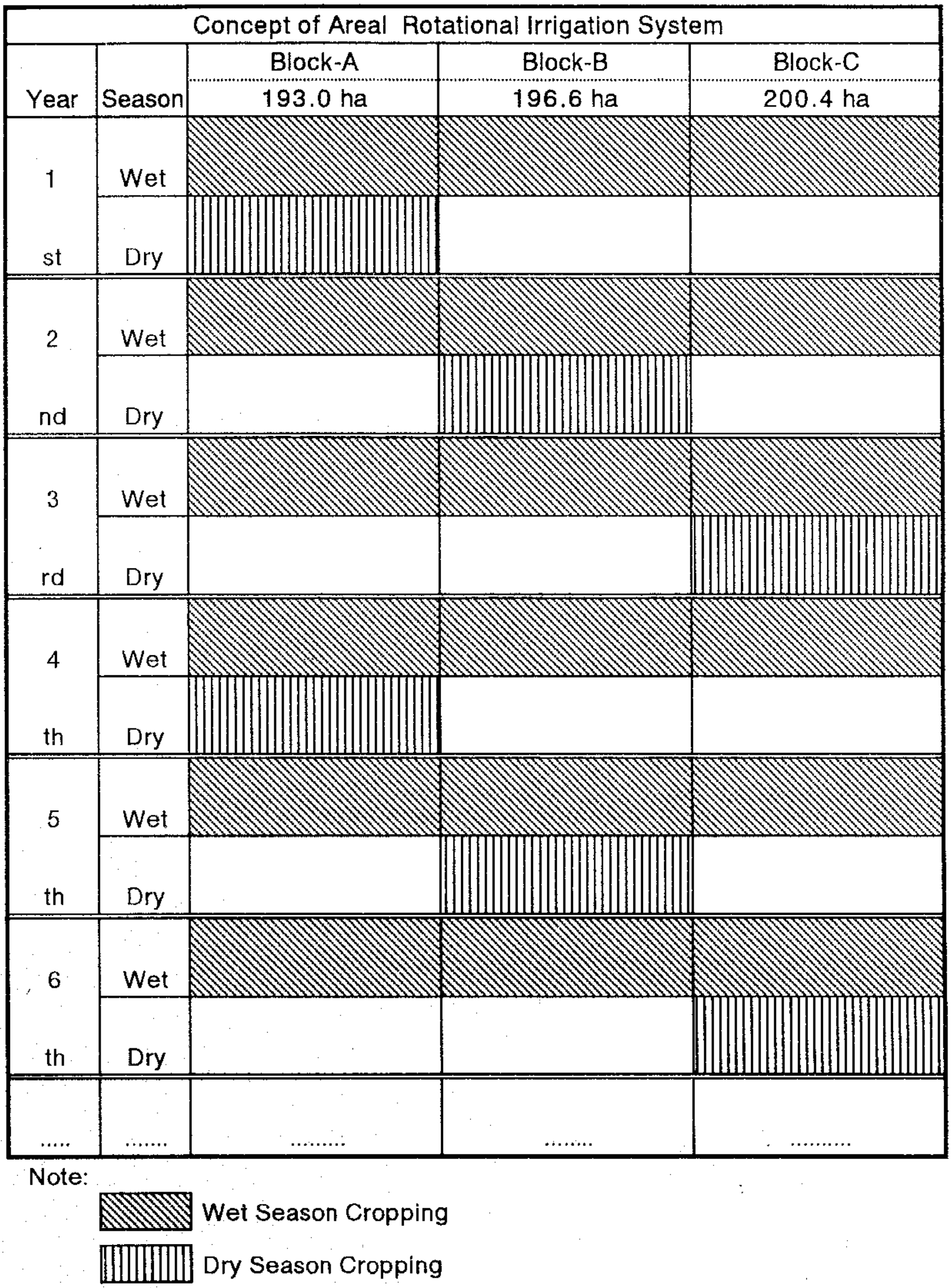


Figure G.3.2 Procedure of Areal Rotational Irrigation System
(in case of cropping intensity 130%, on Stage I Development)



APPENDIX H. AGRICULTURAL INFRASTRUCTURE DEVELOPMENT PLAN

H. 1 IRRIGATION FACILITY PLAN

H. 1 .1 Basic Concept

The basic concept for the planning of the irrigation facilities are summarized as follows:

1) The open canal type on the main and lateral canals are proposed in consideration of the operation and maintenance cost in the future. The terminal point of these canals shall be fixed depending on its irrigable area, which is estimated at about 30 ha to 50 ha.

2) In order to improve the irrigation efficiency at the terminal farm land, terminal irrigation and drainage canal networks are proposed in the Study Area, which will be constructed by DAR/beneficiaries.

3) The main canal will be constructed with concrete lining in consideration of leakage, sliding of the slope, and growing of weeds. On the other hand, the lateral canal will be constructed with earth lining in consideration of the maintenance works in the future, and because it is the most economical in terms of construction.

4) A part of the Study Area will need drainage crossing or some appurtenant structures, except irrigation areas, in consideration of economy.

H. 1. 2 Design Discharge

Unit design discharge and design discharge are calculated without consideration of effective rainfall, the results of which are shown below.

Season	Type of Cropping Pattern		Remarks
	Type-I (q1)	Type-II (q2)	
July (21)	1.850	0.882	Unit ; lit/sec/ha

The design discharge is calculated using the following equation.

$$Q=(q1 \times A1 + q2 \times A2) \times 0.9$$

where;

Q;	Design discharge	(lit/sec)
q1;	Unit design discharge (Type-I)	(lit/sec/ha)
q2;	Unit design discharge (Type-II)	(lit/sec/ha)
A1;	Irrigable area (Type-I)	(ha)
A2;	Irrigable area (Type-II)	(ha)

H. 1. 3 Design of Canals

1) Canal Alignment

The proposed irrigation canals are aligned depending on topographical condition based on the topomap with a scale of 1:4,000. Basically, the main canal follows the contour line, while the lateral canals are situated on higher elevations, parallel with the valley. The proposed canal length and gradient are shown as follows:

Canals		Length (km)	Canal Gradient	Major Appurtenant Structure
Main Canal		4.21	1/2000	3 Siphons, L=1.4 km
Lateral	A	2.38	1/600	2 Siphons, L=0.5 km
Lateral	B	4.23	1/1200	5 Siphons, L=0.3 km
Lateral	C	1.42	1/1000	
Lateral	D	0.49	1/1000	
Lateral	E	2.00	1/1500	

2) Typical Cross Section of Canal

The typical cross section of canals is proposed applying the trapezoidal section in consideration of economical merit and easier construction, particularly, dimension of canal width and water depth was decided based on the most effective cross-section. (refer to Dr-14)

H. 1. 4 Appurtenant Structures

Main appurtenant structures for canal networks being considered are as follows:

- Drainage crossing structures
- Road crossing structures
- Diversion structures/turnouts
- Check structures
- Drop structures

a) Drainage Crossing Structure

The drainage crossing structures are classified into two (2) types based on discharge of the stream or drainage canal as follows:

Crossing River Discharge	Type of Cross Structures
less than 4.0 cu.m/sec	Overchute Type
more than 4.0 cu.m/sec	Siphon Type

- Overchute

  In case of small discharge in crossing a stream or a drainage canal, concrete pipes with 600 to 1,000 mm in diameter are first installed on the stream bed, after which, the overchute are constructed above the concrete pipe structures. Furthermore, side slope of canals and river bed are protected with riprap structures. (refer to Dr-19).

- Siphon

  As mentioned above, siphon are applied in case of big discharge on a crossing stream or crossing of a wide valley. Concrete pipes are utilized as the main structure, because irrigation discharge is less and procurement of materials is easy. (refer to Dr-18).

b) Road Crossing Structures

Since irrigation water discharge is small, siphon or cross culvert is applicable. Construction materials are mainly reinforced concrete pipe (RCP).

The relationship between the discharge and size of concrete pipe are summarized below. (refer to Dr-20)

Design Discharge		Diameter of Concrete Pipe
0 ~0.2 (cu.m/sec)	....	300 (mm)
0.2 ~0.4 (cu.m/sec)	....	450 (mm)
0.4 ~0.6 (cu.m/sec)	....	600 (mm)
0.6 ~0.9 (cu.m/sec)	....	800 (mm)

Note: Velocity of discharge should be kept at more than 1.2 m/sec for protection of sedimentation in the pipes.

c) Diversion Structures/Turnouts

This structure is installed to distribute irrigation water from the main canal to the lateral and lateral to the main farm ditch. In this project, distribution discharge (Q) is less than 0.3 cu.m/sec, so that, turnout can be adopted as diversion facilities. As for water management, control of the discharge is carried out by slide gate.(refer to Dr-21)

d) Check Structure

The function of the check structure is to adjust the water level for stable distribution. It should be located at the downstream near the diversion structure and so that adjustment of the water level can be carried out with only a stop log because of small amount of discharge. The structure generally include a culvert box which is provided with operation bridge.

e) Drop Structure

The drop structure is provided for the adjustment of excess head caused by steep land gradient. The structure is made of reinforced concrete to resist erosion and landslide effects. These structures are constructed with operation bridge. (refer to Dr-22)

H. 2 DRAINAGE FACILITY PLAN

H. 2. 1 Basic Concept

Drainage system in the irrigable area is proposed to facilitate the removal of excess water in the agricultural areas, caused by rainfall and irrigation water. The basic concept of the drainage facilities are summarized as follows:

1) Gravity drainage system, which mainly utilized the existing rivers and small streams, would be proposed.

2) The canal proposed is an open earth canal type.

3) The facilities are designed based on the NIA's criteria. The design discharge module of 6.4 lit/sec/ha is calculated as follows:

   - Design rainfall : Daily rainfall in 5 years return period frequencies
   - Duration of drain : 2 days
   - Runoff Coefficient: 80%

H. 2. 2 Canal Structures

a) Canal Alignment

The canal alignment was carried out based mainly on topomap with a scale of 1:4,000 and field investigation.

The main drainage canal is aligned at the depressed area in a flat area which is located at the central part of the Study Area (refer to the attached Drawings). The proposed canal length are as follows:

- Main drainage canal A: L = 1.1 km
- Main drainage canal B: L = 0.7 km

c) Typical Cross Section

A trapezoidal section is proposed in consideration of economical merit and easier construction. The canal dimension is determined based on the most effective cross-section. The minimum velocity is at 0.4 m/sec for the prevention of sedimentation and growing weeds in the canal. (refer to Dr-15)

H. 2. 3 Appurtenant Structures (Road Crossing)

Since the drainage discharge is small, the structure mainly consists of reinforced concrete pipes. On the other hand, there are existing road crossings crossing the national highway. However, these structures are not very functional because of the absence of drainage. There is a need for these facilities to be rehabilitated in the future. The relationship between discharge and pipe's diameter are shown as follows:

Design Discharge	Diameter of Concrete Pipe
0 ~0.3 (cu.m/sec)	600 (mm)
0.3 ~0.6 (cu.m/sec)	800 (mm)
0.6 ~4.1 (cu.m/sec)	1,000 (mm)
1.1 ~2.2 (cu.m/sec)	1,000 (mm) ×2

H. 3 ROAD FACILITY PLAN

H. 3. 1 Basic Concept

The road network is one of the most important infrastructure for supporting farmer's life. In this project, the roads are classified into three (3) types, namely, main farm-to-market road, farm-to-market road and O/M road. Based on the said classification, the basic concept in the alignment of the roads are shown as follow:

1) Basic alignment is carried out in consideration of farm lot distribution which was carried out by DAR in the Tagumpay area. In the outlying area, the said topo-map is applied for road alignment.

2) Horizontal alignment was proposed to be undertaken to avoid much amount of cutting through and banking and crossing of structures, etc.

3) As to the vertical alignment, the longitudinal slope is fixed at 8.0 degree as the upper limit taking into consideration land sliding and smooth driving of vehicles.

The total length of the main farm-to-market road and the farm-to-market road are measured at 11.8 and 29.2 km, respectively.

H. 3. 2 Design Criteria for Structure

The road surface is paved by mixed gravel with 20 cm thickness in consideration of traffic volume. The height of the road surface is kept at a height higher than the surrounding ground surface. The side ditch is kept on both or one side to avoid road damage from rainfall.

The road width is classified as follows: (refer to Dr-17)

Type of Road	Total Width
Main Farm to Market Road	8.00 m
Farm to Market Road	6.00
O/M Road	4.00

APPENDIX I. RURAL INFRASTRUCTURE DEVELOPMENT

I. 1 VILLAGE PLAN

Prior to the development of the Study Area, the Tagumpay home lot area and farm lands were already distributed to the farmer beneficiaries by DAR. However, for the outlying areas, no lands have yet been distributed to the farmer beneficiaries. DAR will have to distribute the lands to the beneficiaries in 1995 after the completion of the development plan for the Tagumpay Area. At present, there are no more available space for additional home lot at the Tagumpay home lot area. Hence, the new beneficiaries who will be given farm lands at the outlying areas will also have to be provided with home lots. New villages therefore are necessary to be planned out/put up in the outlying area.

a) Proposed Number of Farmer's Beneficiaries

Based on the proposed land use in the outlying area of about 1,000 hectares, about 494 hectares of farm land (gross total) will be distributed to the new farmer beneficiaries. When a farmer is provided 3.0 ha of land (2.94 ha of farm land and 600 sq.m of home lot), the expected number of new beneficiaries will be as follows.

Net area : 452 x 0.9 = 407 ha
No. of farmers : 407 / 2.94 = 138 farmers

b) Proposed Minimum Acreage for the New Villages

Based on the distribution program at the Tagumpay area, about 600 sq.m of home lot was provided to a farmer. Based on this, the estimated minimum home lot area will be as follows:

Net area : 138 farmers x 600 sq.m = 8.28 ha

The minimum public space will be assumed at 30% of the home lot area, calculated as follows:

8.28 x 30% = 2.48 ha

The roads and other necessary facilities, about 10% of the home lot area will have to be provided. The total acreage therefore of the home lot area at the minimum will be as follows:

$$(8.28 + 2.48) / 0.9 = 10.76 \text{ ha}$$

c) Proposed Location of the New Villages

Based on the above estimations, two (2) new villages are proposed in the Study Area. The new village areas are proposed to be located at the northern edge of the Study Area which will temporarily be called as "Village (2)", and another at the east edge and will also be temporarily named as "Village (1)". (refer to Figures I.1.1 and I.1.2, and general Plan Map)

The outline of the two villages will be as follows:

	Village (1)	Village (2)	Total
Gross Area (ha)	5.98	9.98	15.96
No. of Home Lots	46	92	138
Home Lot Area (ha)	2.76	5.52	8.28
Public Space (ha)	2.22	2.74	4.96
Roads and others (ha)	1.00	1.72	2.72
Road Length (m)	987	1,715	2,702

Note: Based on the topo-map with a scale of 1/4,000 prepared by JICA Study Team.

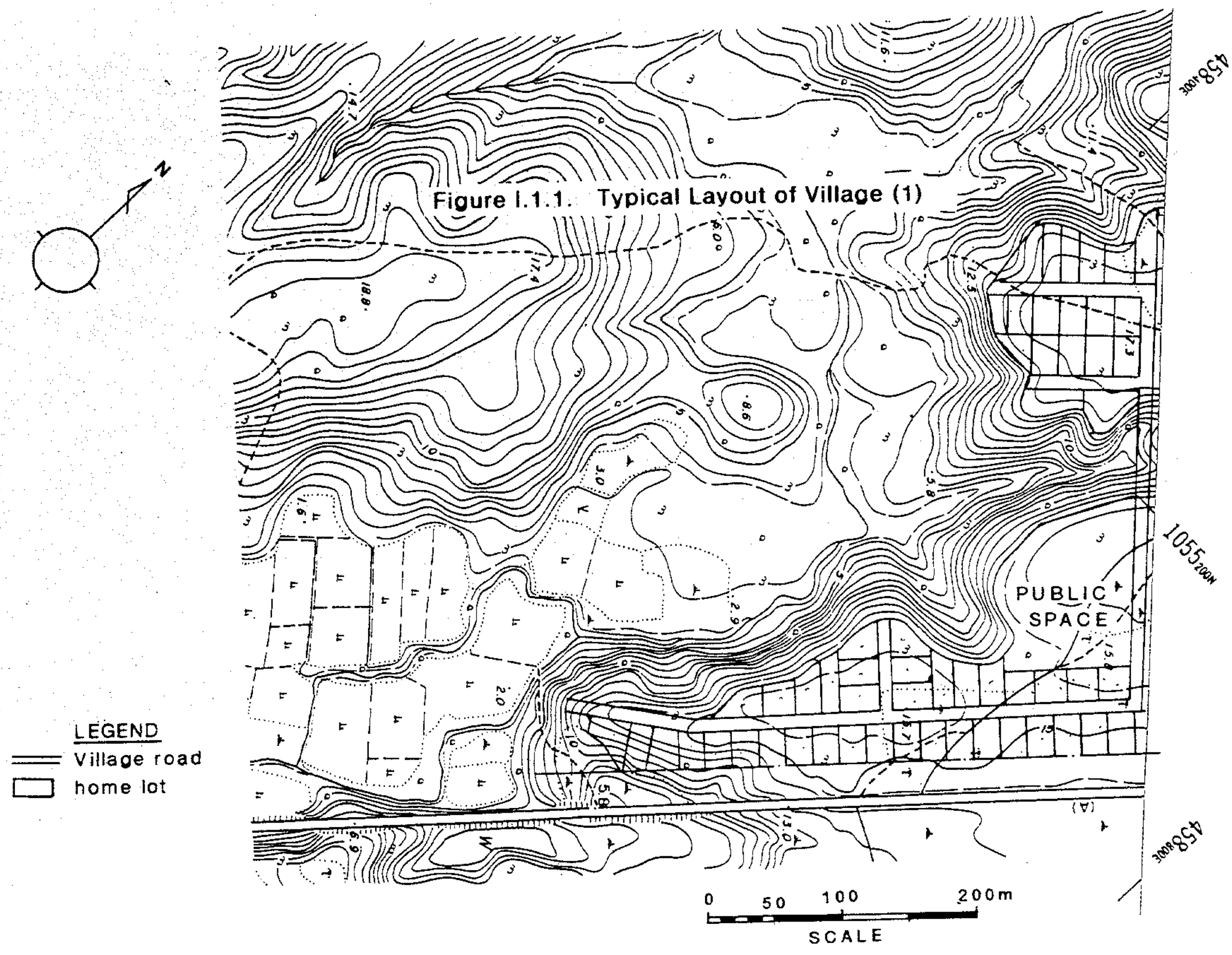
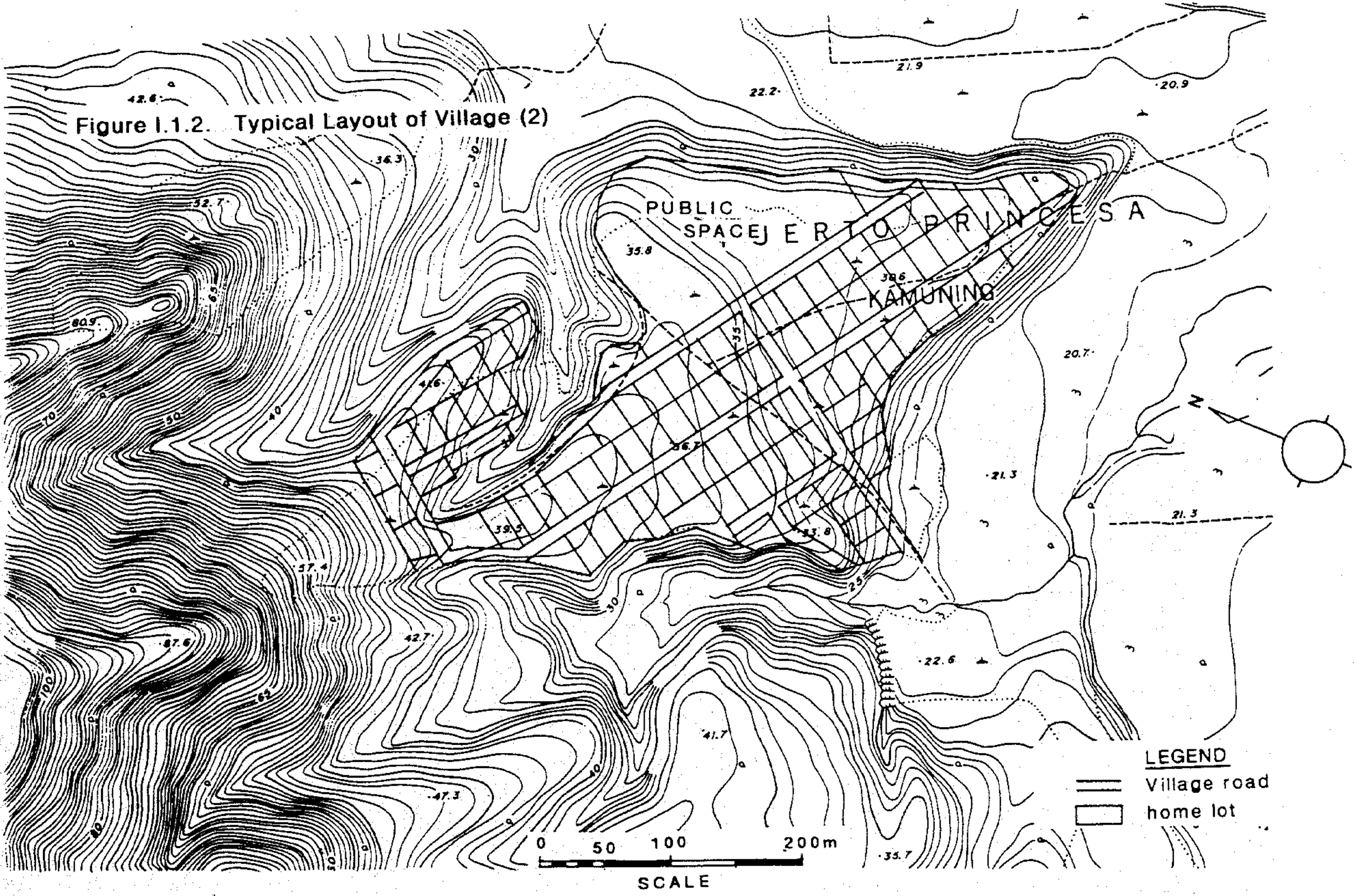


Figure I.1.1. Typical Layout of Village (1)

Figure I.1.2. Typical Layout of Village (2)



I. 2 ALTERNATIVE SOURCES OF WATER FOR DOMESTIC USE

I. 2. 1 Water Sources

Four (4) types of water sources can be applied in the Study Area, namely, deep well, shallow well, water which can be stored in the proposed reservoir, and surface runoff at the Inagawan River or its tributaries. The general characteristics of the water sources are described below.

a) Deep Well

Since water from the deep well is stable and clean throughout the year, it is good for drinking purposes and others. Also, there is no need to install water treatment facility when the deep well is applied.

However, the water quantity in the Study Area is limited up to 75 cu.m per day as mentioned in Appendix E.3. For the utilization of water using a deep well, a submerged pumping facility shall be required with necessary electric charges. However, due to the cleanliness of the water, a treatment facility will not be needed. Depending on the capacity of the delivery tank, the operation of the facility will be easy, using only the "ON-OFF" operation. (refer to Appendix E.3)

b) Shallow Well

Water from shallow wells is the most common water supply system in the Philippines. However, the use of shallow wells brings about some problems and constraints, such as water pollution, drying up of well during the dry season, etc. The Study Area is located at a higher land-like terraces hills with many eroded and deep valleys. Because of this topographical conditions, the shallow wells dries up during the dry season. Shallow wells can be proposed at the alluvial plains where the groundwater table is high. In these areas, shallow wells have water, even during the dry season. However, since the area is not located at the alluvial plain area, water is not stable during the dry season. Also, the labor required for hauling water will not be reduced due to the use of hand pump.

c) Water Stored in the Proposed Reservoir

Water can also be taken from the proposed reservoir for irrigation, by gravity system. This is the cheapest system among the other two (2) water sources already mentioned for the beneficiaries. After the construction of the feeder pipe line from the reservoir to the beneficiary area, only minor maintenance activities will be needed for the operation of the pipe line.

Depending on the reservoir capacity, however, water shortage will occur once in five (5) years. Also, without any treatment measures, water quality of the stored water will not very safe for drinking purposes. At least, a sedimentation tank and a chlorination facility should be installed, if this water system is proposed.

The treatment facility is usually operated by the water users association. The operation of the sedimentation tank is not so difficult. However, the operation of a chlorination facility is rather difficult, because a concentrate of more than 0.1 ppm of extricate chlorine should be maintained at the exit of the stop valve. For the storage of chlorine, careful attention is very necessary to prevent poison liquid. Hence, a storage facility should be proposed to be constructed to prevent the occurrence of poisonous materials. Hypochlorous acid can also be used for raw water treatment. This treatment is easier to operate the use of chlorine.

For the operation of the facility, a permanent person/staff should be employed. This system is applied at a bigger scale if the system used is like Level III. This kind of operation which will need higher technique will rather be difficult for the beneficiaries at present. Any miss operation may harm the beneficiaries' health because of high concentration of chlorine.

d) Surface Water at the Inagawan River or its Tributaries

The water at the Inagawan river and its tributaries is one of the possible water sources for the village water supply system in the Study Area.

However, as mentioned in Appendix C, the river discharge of Inagawan River fluctuates due to unstable rainfall. Also, water quality is not suitable for drinking. Specifically, during the dry season from January to

April, the discharge of the Inagawan River is limited and is not even enough to serve water for the CIS located at the downstream of the Study Area. During this period the surface discharge of the Inagawan tributaries also dries up.

Also, during the flooding period, the water becomes silted coming from the drainage area. This kind of water should be treated before delivery to the beneficiaries. The siltation of the river will eventually cause the blockage of the pipe system.

Another constraints is the elevation of the river bed where the water will be taken which is only about 20 m MSL. This is lower than that (about 40 m) of the surrounding area. A pump facility to lift up water is necessary to be constructed at the intake site. The operation and maintenance cost will therefore be higher than the other methods. Since the pump station is far from the beneficiary site, the operation and maintenance on optimum time is rather difficult without the presence of a permanent skilled personnel and communication system.

e) Conclusion

The water source system to be recommended in the beneficiary area should consider the following factors: easy operation and maintenance, lower cost, and less need for high technique/skill. Judging from characteristics of the above four (4) water sources, the deep well is the most suitable water source among the proposed water sources due to its cheaper and easier operation and maintenance, and less personnel and technique required. The system proposed will not require a treatment facility, hence, cost of facility is lower.

1. 2. 2 Water Quality

During the field survey period, water quality tests were carried out by the JICA Study Team. The test were executed twice, during the dry season and wet season. The water, taken from nine (9) shallow wells and four (4) springs, were analyzed by a portable field kit. The analysis conducted were water temperature, pH, Cu (copper), Fe (Iron), Zn (Zinc), NO2 (Nitrogen dioxide), GB (general bacteria), CGB (coliform group bacteria).

a) Shallow Wells

The beneficiaries who were residing at the home lot area of the Tagumpay settlement area, get their drinking and potable water from nine (9) shallow wells (six (6) wells within the home lot area and three (3) wells in the farm lot area) and from three (3) springs located near the home lot area. These shallow wells with depths ranging from 20 to 60 ft were privately constructed by the landowner. Some wells, however, dries up during the dry season. Also, the water quality of these wells are not very suitable for drinking. The farmer's wife, therefore, has to get water once or twice a day by hauling water from a well located far from her house.(refer to Figure I.2.1)

Based on the results of the water quality test conducted during the dry season in February, some water contain zinc and iron. However, its density is within the allowable limit under the water quality standard of the Philippines. No copper has been found in the water from the wells. (refer to Table I.2.1)

During the wet season in August, water quality tests were also conducted by the JICA Study Team at the field. The quality of water were found to be not suitable for drinking water because of the detected CGB in the water. (refer to Table I.1.2)

b) Springs

There are six (6) springs located at the depressions at the Tagumpay area. Two (2) of these springs are located near the home lot area while the other one at a center of a farm lot area (Farm lot No. 73). The maximum yield of water among springs is only 0.2 lit/sec. The water quality of the springs are usually not good during the day time, because some people wash and bath by the spring water, including animals. Therefore, people get water for drinking only early in the morning when there are fewer human activities (refer Figure to I.2.1).

According to the results of the water quality test conducted in February, CGB were found in all springs where water were sampled from stagnant water in the spring. Although zinc and iron were also detected, the density of the element is within allowable limit under the quality standard of drinking water in the Philippines (refer to Table I.2.1).

During the wet season, the results of the water quality test shows no CGB. The water is suitable for drinking. Other elements found did not exceed the allowable limits of the drinking water standards of the Philippines (refer to Table I.2.2).

I. 2. 3 Water Supply System and Plan

a) Existing Village Water Supply System

Near the location of the Study Area, at Barangay Inagawan, a Level III water system was constructed (1986) using a 100 ft artesian well with a capacity of about 0.4 lit/sec. To lift water up, a submersible pump was installed which goes up to the delivery tank then eventually distributed to the households. The pump is powered by electricity. About 48 households are served by the water system. The system was rehabilitated in 1989 by PIADP. The JICA Study Team conducted water quality test on the system and findings show that the quality of water is suitable for drinking and for other domestic purposes because it does not contain any bacteria or other harmful elements. A minimum water charge of 45 pesos (or 10 cu.m)per household connection per month and a subsequent water charge of 4.5 pesos/cu.m over ten (10) cu.m. is collected for the operation and maintenance.

b) Proposed Village Water Supply System

1) Water Distribution System

In order to reduce labor requirement for the hauling water for domestic purposes, the Level-II water system, where a public faucet for every six (6) house lots will be proposed, to distribute water to the farmer's house. The Level II water system will help contribute in the reduction of work load of the farmer's family members in hauling water, especially the women and children. Level III system will be developed in the future when the beneficiaries will have gained enough income to be able to shoulder the higher O&M cost of the facilities.

2) Water demand per capita

For Level-II water system, the water demand per capita of 60 lit/day/person is applied in the Philippines. For other purposes such as animal drinking water, washing of agricultural products, etc., the 10 lit/day/person will be assumed in estimating the future demand. A distribution loss of 25% (5% loss within the pipe line system and 20% operation loss) will be applied in the design of the pipe line system, thus bringing the water consumption rate at 87.5 lit/day/person.

3) Proposed number of beneficiaries

- Tagumpay Area

The projected population, 20 years after the implementation of the system will be applied to design the water supply system. Based on the present population of 1,733 (= 321 houses x 5.4 person/house) at Tagumpay Area, the projected population of 3,500 is calculated (using the annual growth rate of 3.58%). Other persons residing within the area under off-farming, is assumed to be about 20% of the estimated population. The proposed number of beneficiaries, therefore, will reach 4,200, twenty (20) years after the implementation of the project.

- Other Villages

The proposed number of beneficiaries in the other villages is estimated to be about 140 families. Using the same assumption as the above, the projected population of the other villages will be about 1,900 (1,833 person) in the same year.

4) Total demand

- Tagumpay Area

Based on the above figures the proposed total water demand is calculated at 368 cu.m/day.

- Other Villages

According to the above figure, the total demand of 166 cu.m/day is calculated.

5) Necessary number of water supply system (block)

- Tagumpay Area

  Five (5) water supply blocks (system) will be proposed for the beneficiaries due to water sources constraints having a yield of only 75 cu.m/day.

- Other Villages

  Based on the same reason, three (3) systems are proposed to be constructed.

6) Water demand for each system

- Tagumpay Area

  For the beneficiaries of about 840 persons, using the recommended system, the total water demand for each water system was estimated as follows:

  Average daily demand (Da): 74 cu.m/day
  Maximum daily demand (Dm): 96 cu.m/day (=1.3 x Da)
  Peak hour demand (Dp) : 185 cu.m/day (=2.5 x Da)

- Other Villages

  For the beneficiaries of about 630 persons using the same system, the total water demand for each system was estimated as follows:

  Average daily demand (Da): 55 cu.m/day
  Maximum daily demand (Dm): 72 cu.m/day (=1.3 x Da)
  Peak hour demand (Dp) : 138 cu.m/day (=2.5 x Da)

I. 2. 4 Proposed Facilities

a) Typical Block Alignment at Tagumpay Area

Based on the results of the test well in the Study Area, there is a problem in the quantity of water from the deep well. The expected amount of water from the deep wells is only about 75 cu.m/day per well. Therefore, the recommended number of deep well must be at least be five (5). The wells will separately be aligned at about 300 to 400 m far to avoid interference with each other. (refer to Appendix E.3)

The beneficial area is divided into five (5) blocks, considering the same acreage of block. (refer to Figure I.2.2)

b) Pipe Line Alignment

Based on the block alignment, the pipe lines will be aligned as the following three (3) cases to avoid interference of wells.

Case-I	:	The well shall be located within the home lot area
Case-II	:	The distribution pipe line should be installed at the shortest route possible across the housing lot.
Case-III	:	The well should at least be 300 m away each others.

1) Case-I

The distance of some wells is less than 300 m. The pipe line will be aligned under the village roads for easy maintenance and operation. The total pipe line length is 3,750 m. (refer to Figure I.2.3)

2) Case-II

The pipe line length is 3,426 m. This system has the shortest pipe length among the three (3) cases. Since the pipe line will cross the housing area, there is a possibility of encountering right-of-way problems in the installation of the pipe lines. Moreover, the operation and maintenance is more difficult as compared with the other cases. (refer to Figure I.2.4)

3) Case-III

The interference of the wells can be avoided, because the distance of each well can be kept at a distance of at least 300 m. Since the location of wells is outside the home lot area, right-of-way will not be much of a problem. However,the operation and maintenance of the wells under this case is rather difficult because the well is located far away from the home lot area. The total pipe length required is 3,926 m.(refer to Figure I.2.5)

Based on above mentioned features of the above three (3) alternative alignments, Case-I is recommended to be applied for the Study Area because it will not encounter any right-of-way problem and because it is easier to operate and maintain. (refer to Figure I.2.3)

c) Proposed Facilities

Deep wells with submerged pump, elevated delivery tank, pipe lines and communal faucets are proposed in the beneficiary area.

1) Deep well with submerged pump

(Tagumpay Area)

A deep well with a diameter of 100 mm will be needed. The diameter of the submerged pump with a capacity of 0.067 cu.m/min (=96 cu.m/day) and a 2.2 kw motor, will be 32 mm with a total head of 70 m. The proposed well depth is 50m. A head loss of ten (10) m from the well to the delivery tank is assumed. The tank will be located 10 m higher than the ground surface. Consequently, a total head of 70 m will be necessary

(Other Village)

A deep well with a diameter of 100 mm will be needed. The diameter of the submerged pump with a capacity of 0.050 cu.m/min (=72 cu.m/day) and a 1.5 kw motor, will be 32 mm with a total head of 70

m. The proposed well depth is 50 m. Due to the same reasons, the total lifting head of 70 m of pump is proposed.

2) Elevated delivery tank

(Tagumpay Area)

The delivery tank will have a capacity of two (2) hours volume between the peak hour demand and the average day demand, which is 7.4 cu.m (= (185-96)/24 x 2). The tank will have the same width and length of 2.0 m, and an effective depth of 1.85 m for a smooth delivery of water. The total depth of the tank will be 2.4 m including 30% of the effective depth (h) of water storage, equivalent to 55 cm, for clearance. The actual height of the tank would be planned later based on the exact location and ground elevation. The bottom of the tank will be about 10 m above the ground surface level.

(Other Villages)

Using the same procedure as that of the Tagumpay Area, the tank capacity will be 5.5 cu.m, long while the width will be 2.0m with an effective depth of 1.38m. The total depth of the tank will be 1.8 m including the 42 cm clearance.

3) Feeder Pipe Line

In Blocks 2 and 3, the delivery tank and the deep well will separately be located to avoid interference with each other. The feeder pipe will have to be designed between the well and the delivery tank. The proposed length are 120 m in Block 2 and 90 m in Block 3. The Steel Gas Pipe (SGP) of 40 mm in diameter will be proposed considering the economic velocity of discharge in the pipe, which is about 1.0 m/sec, and higher water pressure for lifting up water by pump.

$V = Q / A = 0.001117 / 0.00126 = 0.88$ m/sec

Where: V - mean velocity (m/sec)
Q - design discharge (cu.m/sec) = 0.001117 cu.m/sec

A - flow area of pipe (sq.m) = 0.00126 sq.m in 40 mm pipe diameter

4) Distribution Pipe line

(Tagumpay Area)

The capacity of the pipe line is designed to meet the peak hour demand of 185 cu.m/day, equivalent to 2.14 lit/sec at the maximum. The proposed pipe diameter will be determined based on the design discharge. The pipe lines will be buried at about 1.0 m below the village road surface to avoid some damages brought about by passing vehicles and others. The other necessary structures such as air valve, stop valve, drains, etc. will also be proposed.The pipe diameter is determined by the William-Hezen formula with C (roughness coefficient factor) of 150 (Vinyl Chloride pipe). The formula is as follows:

$$HL = 10.666 \times C^{-1.85} \times D^{-4.87} \times Q^{-1.85}$$

Where: HL - Friction loss per pipe liner meter (m)
C - Roughness coefficient factor
D - Diameter of pipe (m)
Q - Discharge (cu.m/sec)

$$THL = HL \times L \times 1.1$$

Where: THL- Total head loss (m)
L - Pipe liner length (m)
1.1- fraction for other head loss caused by the bend, structures, etc.

The actual effective head of the distribution pipe line will be kept at more than 10 m above the ground surface. The friction loss of 5 m in the delivery pipe, branching off from the distribution pipe line to the communal faucet is assumed. An effective water head of 5 m (0.5 kg./sq.cm) at the exit of the stop valve will be expected during the hourly peak water demand time. During normal times, the effective head at the exit of the stop valve is more than that of the peak water demand time. The proposed pipe diameters are from 25 to 100 mm. (refer to Table I.2.3)

(Other Village)

The capacity and diameter of the pipe lines will be determined in the same way as that of Tagumpay area. The design discharge of 143 cu.m/day (equivalent to 1.66 lit/sec) will be calculated. The necessary structures and the formula in the design of the pipe line will also be the same as that of the Tagumpay area.

5) Communal faucet

(Tagumpay area)

Three (3) stop valves will be designed and the proposed pressure at the exit of a faucet will be 0.5 kg/sq.cm.

The proposed discharge is calculated as:

Total water demand = 87.5 lit/day x 4,200 = 368 cu.m/day
Water demand for each household: 368/321 = 1.15 cu.m/day
Average daily demand by faucet : 1.15 x 6 = 6.9 cu.m/day
(equivalent to 0.080 lit/sec)
Daily maximum demand by faucet : 0.080 x 1.3 = 0.104 lit/sec
Daily peak demand by faucet : 0.080 x 2.5 = 0.200 lit/sec

(Other Village)

Also the same as that of the Tagumpay area.

Table I.2.1 Results of Water Quality Test at Wells and Springs in Dry Season (in the Tagumpay Settlement Area)

Well No.	Location	Depth (ft)	Construction Fund	Water Quality: 1st Test: pH	Cu (mg/lit)	Fe (mg/lit)	Zn (mg/lit)	NO2 (mg/lit)	RE (mg/lit)	GB	CGB	2nd Test: CGB
1	HL- 20	20	Private	7.0	-	5.0	-	-	-	3	dct	
2	HL- 31	20	Private	7.8	-	3.0	3.0	-	-	91	dct	
3	HL- 39	40	Private	7.0	-	0.5	0.5	-	-	81	dct	
4	HL-157	na	Private	7.0	-	-	0.7	-	-	-	-	dct
5	HL-321	20	Private	7.5	-	0.3	0.3	-	-	21	-	-
6	HL-299	30	Private	7.5	-	0.8	5.0	-	-	2	-	dct
7	FL- 1	25	Private	6.5	-	0.2	0.5	-	-	11	dct	
8	FL- 25	60	Private	6.5	-	8.0	6.0	0.1	-	22	dct	
9			Private	7.0	-	0.2	0.8	-	-	2	-	dct
Spring No.	**Location**		**Yeild (lit/sec)**	**Water Quality: 1st Test: pH**	**Cu (mg/lit)**	**Fe (mg/lit)**	**Zn (mg/lit)**	**NO2 (mg/lit)**	**RE (mg/lit)**	**GB**	**CGB**	**2nd Test: CGB**
1	FL- 45		0.22	7.5	-	-	-	-	-	75	dct	
2	FL- 66		0.07	7.0	-	0.5	0.4	-	-	9	dct	
3	FL- 73		0.02	5.8	-	-	-	-	-	20	dct	

Note: 1st in Jan. 28 and 2nd in Feb 10, 1994

Table I.2.2 Results of Water Quality Test at Wells and Springs in Wet Season (in the Tagumpay Settlement Area)

Well No.	Location	Depth (ft)	Aug/18/1994: Water Temperature (℃)	pH	Cu (mg/lit)	Fe (mg/lit)	Zn (mg/lit)	NO2 (mg/lit)	GB	CGB	Aug/30/1994: Water Temperature (℃)	pH	Cu (mg/lit)	Fe (mg/lit)	Zn (mg/lit)	NO2 (mg/lit)	GB	CGB
1	HL- 20	20	27.5	6.3	-	3.0	1.0	0.05	6	-	28.0	6.5	-	3.0	1.5	-	4	-
2	HL- 31	20	28.0	7.0	-	2.0	1.0	-	100	-	28.0	7.0	-	2.0	7.0	-	5	dct
3	HL- 39	40	29.0	7.0	-	3.0	1.0	-	100	-	27.5	7.0	-	0.5	0.7	-	100	dct
4	HL-157	20	30.0	7.5	-	-	0.5	-	100	dct	29.0	7.5	-	-	2.0	-	100	dct
5	HL-321	20	32.0	7.0	-	0.5	1.5	-	3	dct	30.0	7.0	-	0.5	1.0	-	9	-
6	HL-299	30	27.5	7.0	-	0.2	3.0	-	100	-	28.0	7.0	-	0.1	5.0	-	100	-
7	FL- 1	25	29.0	6.0	-	1.0	1.5	-	2	-	29.5	6.5				-	8	-
8	FL- 25	60	29.0	6.0	-	8.0	7.0	0.7	100	-	28.5	6.0	-	8.0	4.0	1.0	9	dct
9	HL-322	60									30.0	6.5	-	0.7	10.0	-	4	-
Spring No.	**Location**	**Yeild (lit/sec)**	**Aug/18/1994: Water Temperature (℃)**	**pH**	**Cu (mg/lit)**	**Fe (mg/lit)**	**Zn (mg/lit)**	**NO2 (mg/lit)**	**GB**	**CGB**	**Aug/30/1994: Water Temperature (℃)**	**pH**	**Cu (mg/lit)**	**Fe (mg/lit)**	**Zn (mg/lit)**	**NO2 (mg/lit)**	**GB**	**CGB**
1	FL- 45	0.32	29.0	7.0	-	0.5	0.5	0.05	100	-	28.0	7.0						
2	FL- 66		28.0	7.0	-	2.0	0.5	-	100	-	27.0	7.0	-	2.0	0.5	-		
3	FL- 73		30.0	5.7	-	0.2	0.3	-	100	-	27.0	5.7	-	-	-	-		
4	FL- 70		32.0	6.2	-	0.2	-	-	100	-	27.0	6.5	-	0.2	-	-		

note: 'dct' is detect

Table I.2.3 Hydraulic Calculation of Distribution Pipe Line on Village Water Supply System (Case-1)

(1/3)

1 Block

STA	L	Q	D	Head Loss /100m	Head Loss	WL	GL	Actual Head
	(m)	(lit/) sec)	(mm)	(m)	(m)	(m)	(m)	(m)
No.0	0					29.60	20.5	9.10
+2	2	2.0	75	0.307	0.01	29.59	20.5	9.09
PF-4	10	2.0	75	0.307	0.03	29.56	18.5	11.06
+42	32	2.0	75	0.307	0.11	29.45	18.9	10.55
+2						29.59		
PF-3	50	0.3	25	1.936	1.06	28.53	13.0	15.53
PF-2	60	0.2	25	0.914	0.60	27.93	16.2	11.73
PF-1	60	0.1	25	0.254	0.17	27.76	15.0	12.76
+42						29.45		
+112	70	1.6	75	0.203	0.16	29.29	17.1	12.19
PF-8	32	0.8	40	1.205	0.42	28.87	17.0	11.87
PF-7	60	0.6	40	0.708	0.47	28.40	16.3	12.10
PF-6	60	0.4	25	3.297	2.18	26.22	10.0	16.22
PF-5	60	0.2	25	0.914	0.60	25.62	10.0	15.62
+112						29.29		
+182	70	0.8	50	0.406	0.31	28.98	16.0	12.98
PF-12	32	0.8	50	0.406	0.14	28.84	17.1	11.74
PF-11	60	0.6	50	0.239	0.16	28.68	17.0	11.68
PF-10	60	0.4	25	3.297	2.18	26.50	16.5	10.00
PF-9	60	0.2	25	0.914	0.60	25.90	14.0	11.90
Total	778							

2 Block

STA	L	Q	D	Head Loss /100m	Head Loss	WL	GL	Actual Head
	(m)	(lit/) sec)	(mm)	(m)	(m)	(m)	(m)	(m)
No.0	0					27.20	15.0	12.20
+2	2	2.4	75	0.431	0.01	27.19	7.0	20.19
PF-14	30	0.4	25	3.297	1.09	26.10	8.5	17.60
PF-13	60	0.2	25	0.914	0.60	25.50	7.0	18.50
+2						27.19		
PF-15	30	2.0	75	0.307	0.10	27.09	17.0	10.09
PF-16	60	1.8	75	0.253	0.17	26.92	15.9	11.02
+124	32	1.6	75	0.203	0.07	26.85	6.0	20.85
+194	70	1.6	50	1.465	1.13	25.72	15.0	10.72
PF-20	32	0.8	50	0.406	0.14	25.58	15.8	9.78
PF-19	60	0.6	40	0.708	0.47	25.11	15.0	10.11
PF-18	60	0.4	25	3.297	2.18	22.93	13.0	9.93
PF-17	60	0.2	25	0.914	0.60	22.33	6.0	16.33
+194						25.72		
+264	70	0.8	50	0.406	0.31	25.41	5.0	20.41
PF-24	32	0.8	40	1.205	0.42	24.99	13.7	11.29
PF-23	60	0.6	40	0.708	0.47	24.52	12.5	12.02
PF-22	60	0.4	25	3.297	2.18	22.34	12.1	10.24
PF-21	60	0.2	25	0.914	0.60	21.74	8.5	13.24
Total	778							

(2/3)

3 Block

STA	L	Q	D	Head Loss /100m	Head Loss	WL	GL	Actual Head
	(m)	(lit/) sec)	(mm)	(m)	(m)	(m)	(m)	(m)
No.0	0					24.50	13.2	11.30
+2	2	2.2	75	0.367	0.01	24.49	10.9	13.59
PF-30	10	1.0	50	0.614	0.07	24.42	14.1	10.32
PF-29	50	0.9	50	0.505	0.28	24.14	11.9	12.24
PF-28	70	0.8	50	0.406	0.31	23.83	11.4	12.43
PF-27	60	0.6	40	0.708	0.47	23.36	11.6	11.76
PF-26	60	0.4	25	3.297	2.18	21.18	11.1	10.08
PF-25	60	0.2	25	0.914	0.60	20.58	9.0	11.58
+2						24.49		
PF-31	45	0.9	40	1.498	0.74	23.75	12.9	10.85
PF-56	75	0.8	40	1.205	0.99	22.76	12.0	10.76
PF-57	60	0.6	40	0.708	0.47	22.29	11.0	11.29
PF-58	60	0.4	25	3.297	2.18	20.11	9.2	10.91
PF-59	60	0.2	25	0.914	0.60	19.51	8.0	11.51
Total	612							

4 Block

STA	L	Q	D	Head Loss /100m	Head Loss	WL	GL	Actual Head
	(m)	(lit/) sec)	(mm)	(m)	(m)	(m)	(m)	(m)
No.0	0					26.50	16.0	10.50
+20	20	2.4	100	0.106	0.02	26.48	16.4	10.08
PF-44	32	0.8	50	0.406	0.14	26.34	15.2	11.14
PF-45	60	0.6	50	0.239	0.16	26.18	13.0	13.18
PF-46	60	0.4	40	0.334	0.22	25.96	12.2	13.76
PF-47	60	0.2	40	0.093	0.06	25.90	16.0	9.90
+20						26.48		
+90	70	1.6	75	0.203	0.16	26.32	15.1	11.22
PF-48	32	0.8	50	0.406	0.14	26.18	14.6	11.58
PF-49	60	0.6	40	0.708	0.47	25.71	12.2	13.51
PF-50	60	0.4	40	0.334	0.22	25.49	12.0	13.49
PF-51	60	0.2	25	0.914	0.60	24.89	14.7	10.19
+90						26.32		
+160	70	0.8	50	0.406	0.31	26.01	14.2	11.81
PF-52	32	0.8	40	1.205	0.42	25.59	13.8	11.79
PF-53	60	0.6	40	0.708	0.47	25.12	12.6	12.52
PF-54	60	0.4	25	3.297	2.18	22.94	10.0	12.94
PF-55	60	0.2	25	0.914	0.60	22.34	12.0	10.34
Total	796							

Table I.2.3 (Cont'd)

(3/3)

5 Block								
STA	L	Q	D	Head Loss /100m	Head Loss	WL	GL	Actual Head
	(m)	(lit/sec)	(mm)	(m)	(m)	(m)	(m)	(m)
No.0	0					33.80	22.9	10.90
+10	10	2.0	50	2.214	0.24	33.56	23.1	10.46
PF-35	32	0.4	40	0.334	0.12	33.44	23.0	10.44
PF-34	60	0.3	40	0.196	0.13	33.31	22.0	11.31
PF-33	60	0.2	25	0.914	0.60	32.71	19.9	12.81
PF-32	60	0.1	25	0.254	0.17	32.54	16.0	16.54
+10						33.56		
+80	70	1.6	50	1.465	1.13	32.43	22.2	10.23
PF-39	32	0.8	40	1.205	0.42	32.01	22.0	10.01
PF-38	60	0.6	40	0.708	0.47	31.54	20.0	11.54
PF-37	60	0.4	25	3.297	2.18	29.36	16.0	13.36
PF-36	60	0.2	25	0.914	0.60	28.76	14.0	14.76
+80						32.43		
+150	70	0.8	40	1.205	0.93	31.50	18.5	13.00
PF-43	32	0.8	40	1.205	0.42	31.08	20.0	11.08
PF-42	60	0.6	40	0.708	0.47	30.61	16.5	14.11
PF-41	60	0.4	25	3.297	2.18	28.43	12.0	16.43
PF-40	60	0.2	25	0.914	0.60	27.83	15.8	12.03
Total	786							

Pipe Length by Diameter (m)

	Block No.					
Dia	1	2	3	4	5	Total
(mm)						
125	-	-	-	-	-	0
100	-		-	20	-	20
75	114	124	2	70	-	310
50	162	172	130	194	80	738
40	92	152	240	332	346	1,162
25	410	330	240	180	360	1,520
T	778	778	612	796	786	3,750

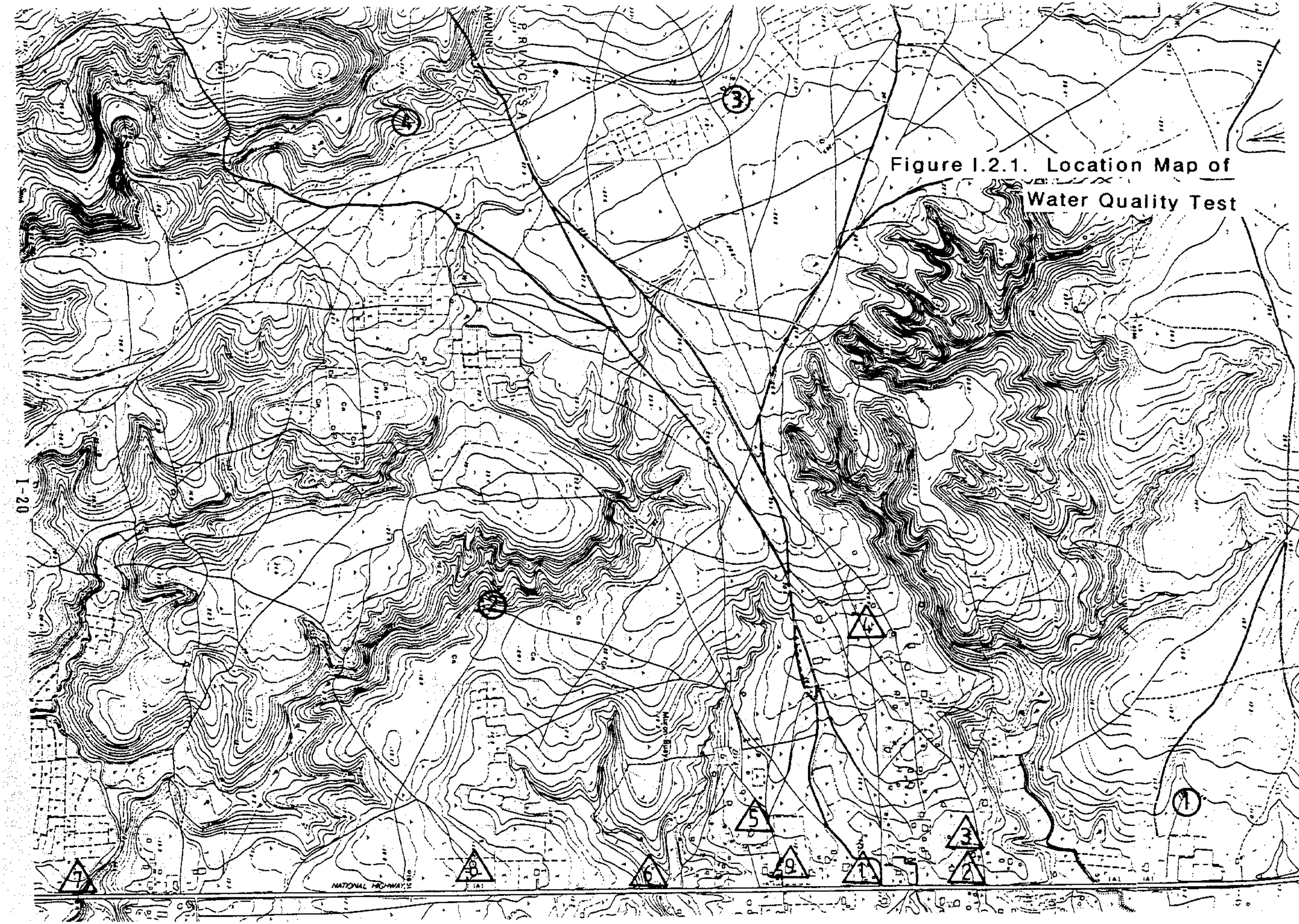
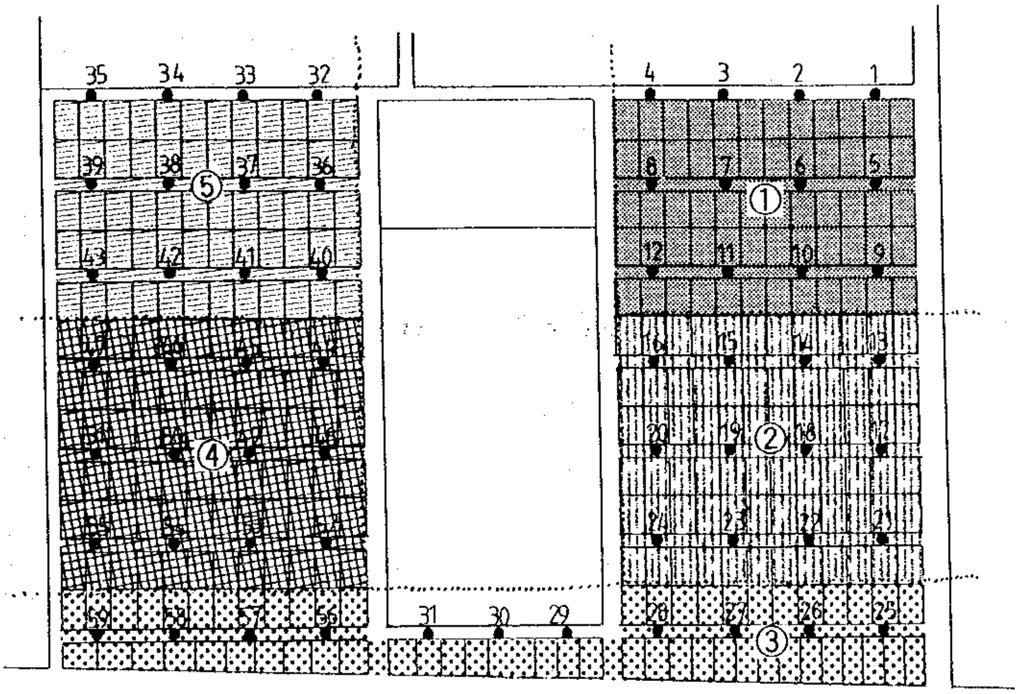


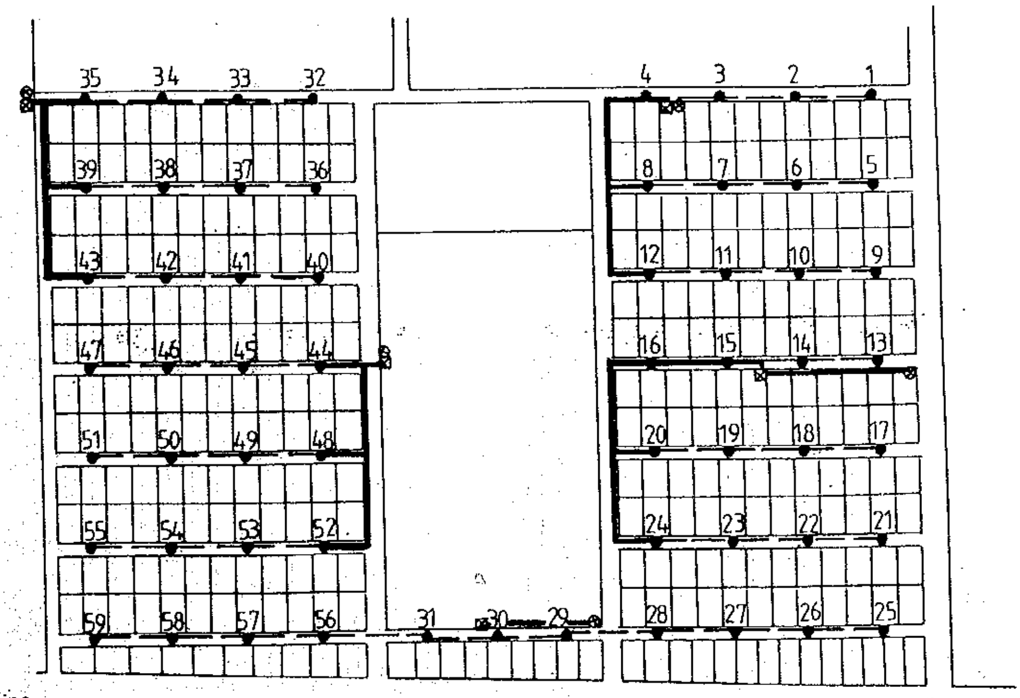
Figure I.2.1. Location Map of Water Quality Test

Figure I.2.2 Village Water Service Block



LEGEND
Boundary of Block
① Block No.

Figure I.2.3 Alternative Plan of Pipe Line Alignment (Case-I)



LEGEND
Deep Well
Delivery Tank
Distribution Line
Public Faucet

Figure I.2.4 Alternative Plan of Pipe Line Alignment (Case-II)

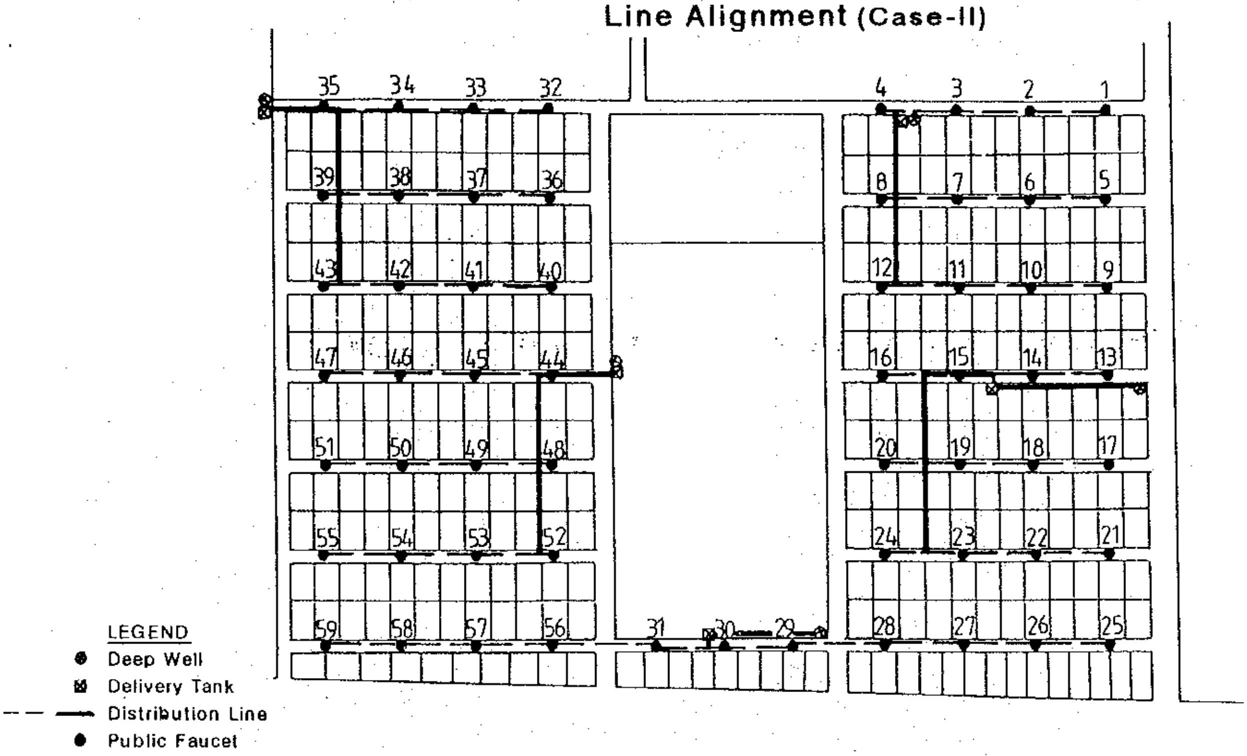
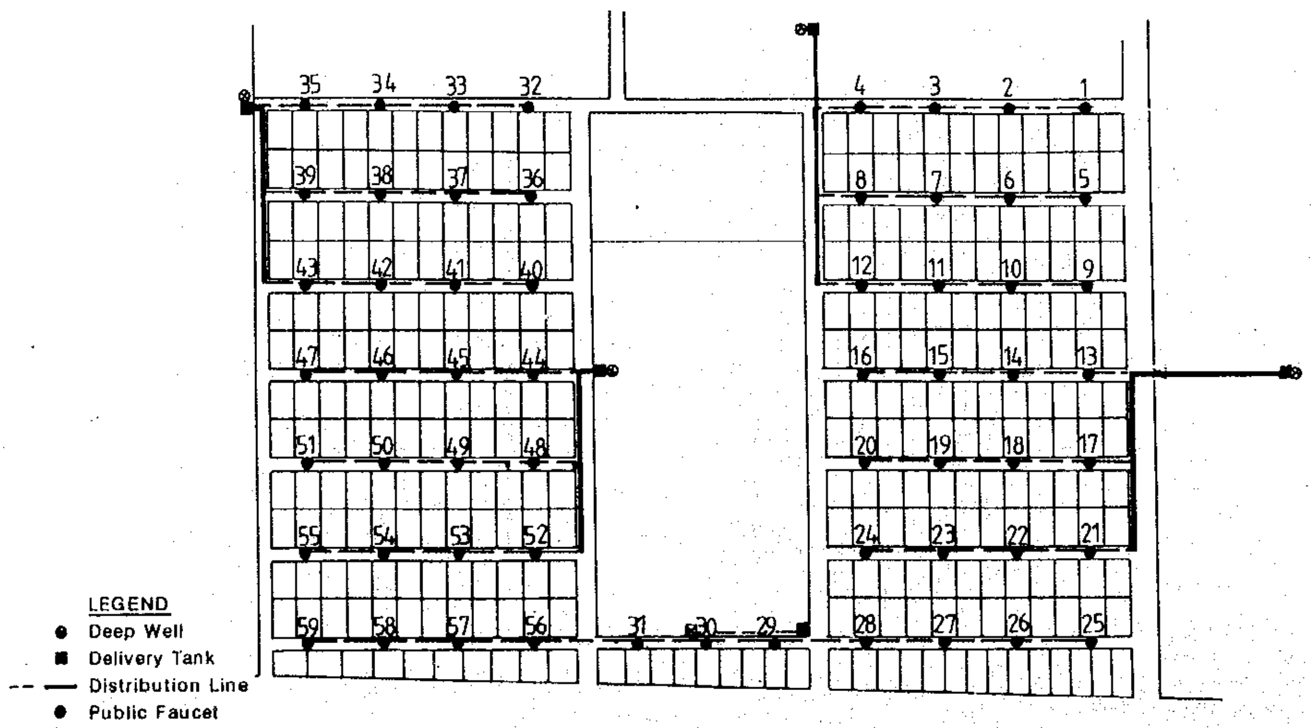


Figure I.2.5 Alternative Plan of Pipe Line Alignment (Case-III)



I. 3 VILLAGE ROAD AND DRAIN

a) Village Road

1) Proposed Road Length

At the Tagumpay Area, the city government constructed main village roads along the public space as requested by the beneficiaries in the Study Area. However, only the stripping works were undertaken by the city government at present. The remaining proposed roads have not been implemented yet. The city government has no definite plans to improve and construct the village roads at the Tagumpay home lot area. The total proposed length of the village road is 5,306 m for the home lot area of 24.7 ha (equivalent to 214.8 m/ha). The roads at the Tagumpay Home Lot Area are classified into two (2), the main village roads with a total length of 967 m and the 18 village roads with a total length of 4,339 m.(refer to Table I.3.1 and Figure I.3.1)

2) Scale of Road and Pavement

Road width of six (6) and four (4) meters will be proposed for the main village road (MVR) and the village road (VR), respectively. The road way will be paved with gravel, 20 cm thick in order to make the road passable even during the rainy season. The design limit of longitudinal slope of road surface is 12%. The road way with the longitudinal slope of more than 8% will be paved with concrete to prevent the road surface from erosion caused by heavy rainfall. The length of the paved concrete is 84 meters at the main village road and 445 meters at the village roads at the Tagumpay Home Lot Area. (refer to Table I.3.1 and Figures I.3.2 and I.3.3)

b) Village Drain

To drain smoothly excess water from the housing area, the village drains will be proposed.

1) Alignment of the Village Drain

Based on the topographic condition, the village drains shall be aligned within the village area. The drainage discharge from the drainage area shall be drained out of the village area. (refer to Figure I.3.4)

2) Drainage Module

The drainage module applied is the same as the drainage plan for the farm land. The drainage module of 6.4 lit/sec/ha is calculated based on the run-off coefficient of 80% and two (2) days drainage period for the probable daily rainfall of 138.4 mm on a five (5) year return period.

3) Scale and Length of the Proposed Drain

About 1.4 km of six (6) village drains will be proposed in the village area. The drain shall have a side slope of 1:1, a minimum bottom width of 30 cm and a minimum depth of 60 cm. The maximum bed slope shall be limited to 1/130 due to the limited velocity of 1.0 m/sec of drain water to resist canal scouring. (refer to Tables I.3.2 to I.3.4. and Figure I.3.4)

4) Appurtenant Structures

There will be 17 road crossing structures and 14 drop structures. The 14 drops are proposed to avoid canal scouring by keeping the velocity to less than 1.0 m/sec. Reinforced Concrete (RC) pipe culverts with a minimum diameter of 600 mm will be utilized for the structures under the village road.(refer to Table I.3.2 and Figure I.3.4)

Table I.3.1 List of Proposed Village Road (in Tagumpay Area)

Name of Road	Road Width (m)	Road Length (m)	Pavement Length Gravel (m)	Pavement Length Concrete (m)	Total (m)
MVR-1	6.0	482.0	398.0	84.0	482.0
MVR-2	6.0	485.0	485.0	0.0	485.0
ST		967.0	883.0	84.0	967.0
VR-1	4.0	240.0	192.0	48.0	240.0
VR-2	4.0	240.0	199.0	41.0	240.0
VR-3	4.0	240.0	190.0	50.0	240.0
VR-4	4.0	240.0	204.0	36.0	240.0
VR-5	4.0	240.0	240.0	0.0	240.0
VR-6	4.0	240.0	240.0	0.0	240.0
VR-7	4.0	240.0	164.0	76.0	240.0
VR-8	4.0	168.0	168.0	0.0	168.0
VR-9	4.0	168.0	128.0	40.0	168.0
VR-10	4.0	240.0	184.0	56.0	240.0
VR-11	4.0	240.0	240.0	0.0	240.0
VR-12	4.0	240.0	240.0	0.0	240.0
VR-13	4.0	240.0	240.0	0.0	240.0
VR-14	4.0	240.0	240.0	0.0	240.0
VR-15	4.0	240.0	193.0	47.0	240.0
VR-16	4.0	240.0	240.0	0.0	240.0
VR-17	4.0	475.0	424.0	51.0	475.0
VR-18	4.0	168.0	168.0	0.0	168.0
ST		4,339.0	3,894.0	445.0	4,339.0
Total		5,306.0	4,777.0	529.0	5,306.0
per ha		214.8	193.4	21.4	214.8

Note:Total are of 24.7 ha
Based on topo map (scale of 1/4,000)

Table I.3.2 List of Main Village Drain

Table I.3.2. List of Main Village Drain

Name of V.Drain	Length (m)	Drainage Area (ha)	Design Discharge (lit/sec)	Longt. Slope (1/)	--- Dimesion --- Width (m)	Depeth (m)	Proposed Slope (1/)	No.of Drops
VD-1	210	14.5	74.0	70	0.3	0.6	130	2
VD-2	175	2.6	13.3	220	0.3	0.6	220	0
VD-3	280	13.4	68.3	50	0.3	0.6	130	4
VD-4	590	15.3	78.0	60	0.3	0.6	130	5
VD-5	90	15.1	77.0	50	0.3	0.6	130	2
VD-6	60	30.4	155.0	80	0.3	0.6	130	1
Total	1,405							14

Table I.3.3 Calculation of Slope

Name of V.Drain	Length (m)	--- Elevation --- High (m)	Low (m)	Differ.	Existing Slope (1/)	Proposed Slope (1/)	Proposed Differ.	No.of Drops
VD-1	210	10	6.9	3.1	68	130	1.6	2
VD-2	175	7	6.2	0.8	219	220	0.8	0
VD-3	280	11.5	6	5.5	51	130	2.2	4
VD-4	590	17	7.5	9.5	62	130	4.5	5
VD-5	90	9.5	7.5	2	45	130	0.7	2
VD-6	60	6.8	6	0.8	75	130	0.5	1
Total	1405							14

Table I.3.4 Calculation of Drainage Capacity

ss= 1:1 n= 0.03

Bottom (m)	Depth (m)	Water Depth (m)	Area (m2)	P (m)	R (m)	I (1/)	V (m/sec)	Q (m3/sec)
0.3	0.6	0.4	0.280	1.431	0.196	100	1.125	0.315
.	.	.	.	.	.	110	1.072	0.3
.	.	.	.	.	.	120	1.027	0.288
.	.	.	.	.	.	130	0.986	0.276
.	.	.	.	.	.	140	0.951	0.266
.	.	.	.	.	.	150	0.918	0.257
.	.	.	.	.	.	160	0.889	0.249
.	.	.	.	.	.	220	0.758	0.212

Note: by Maning Equation

Figure I.3.1 Location Map of Village Road

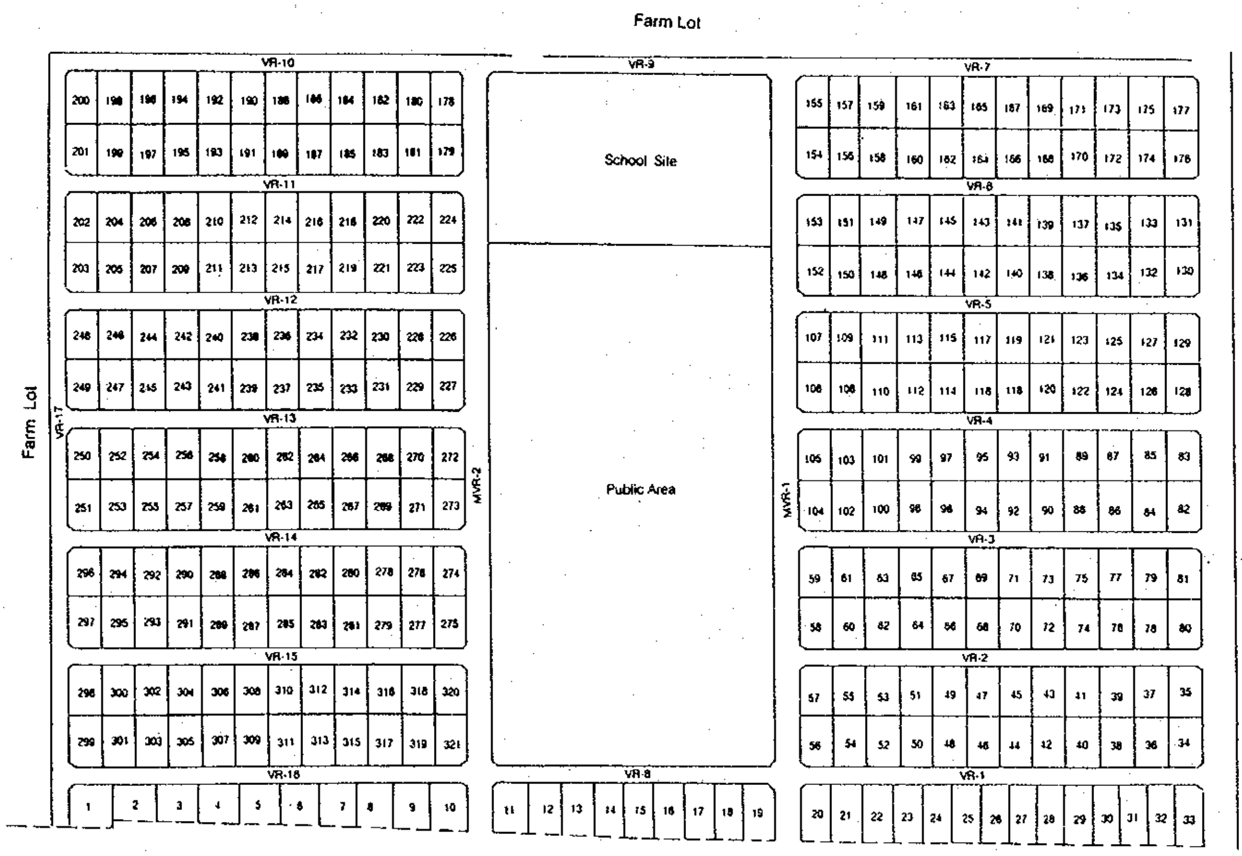
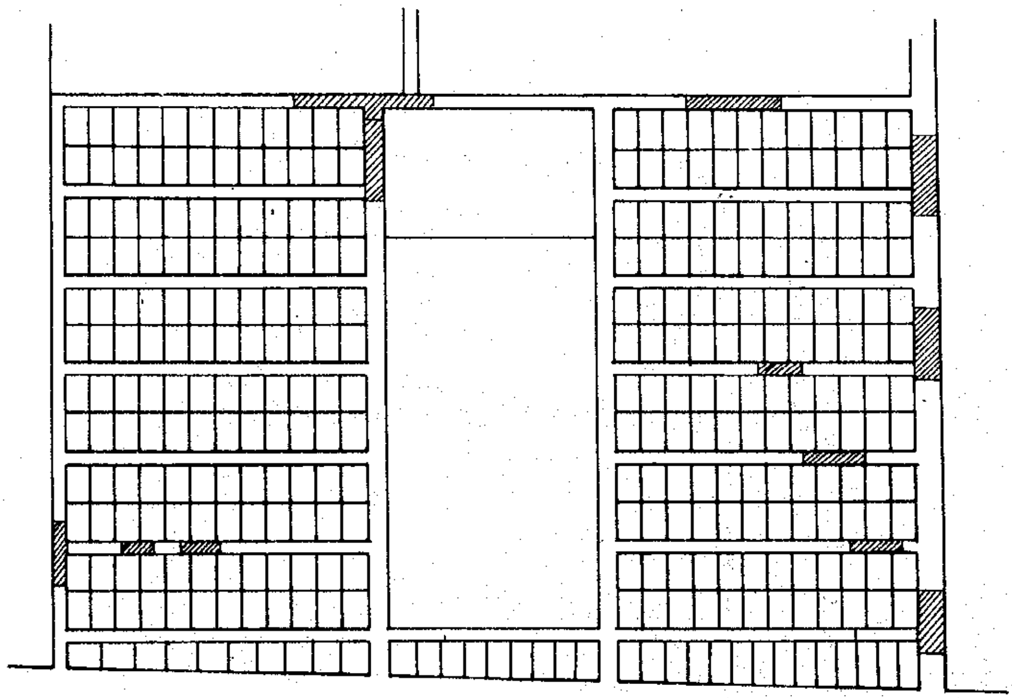


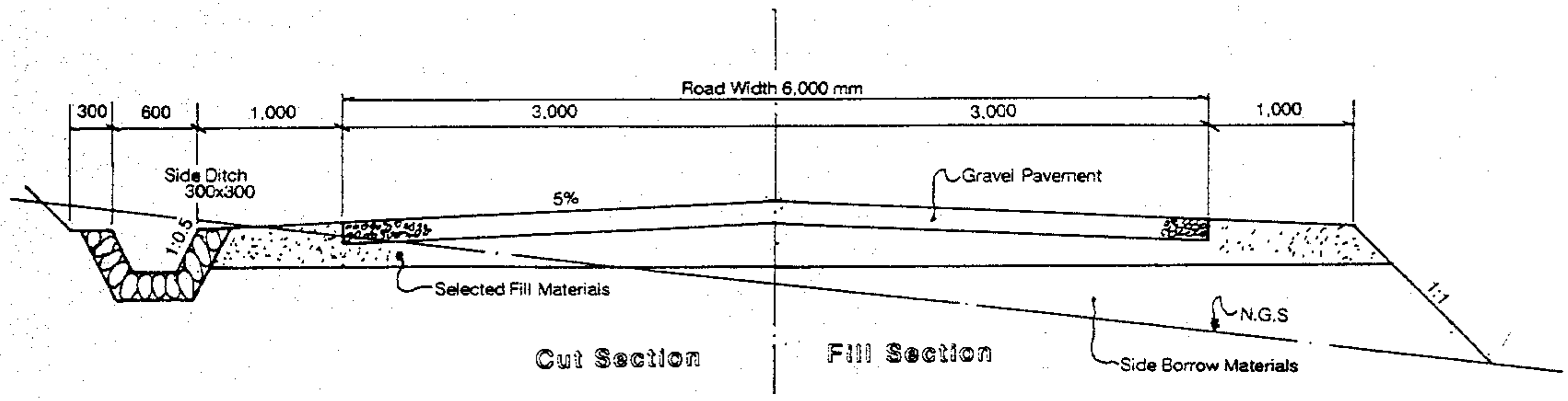
Figure I.3.3 Location of Concrete Pavement Portion



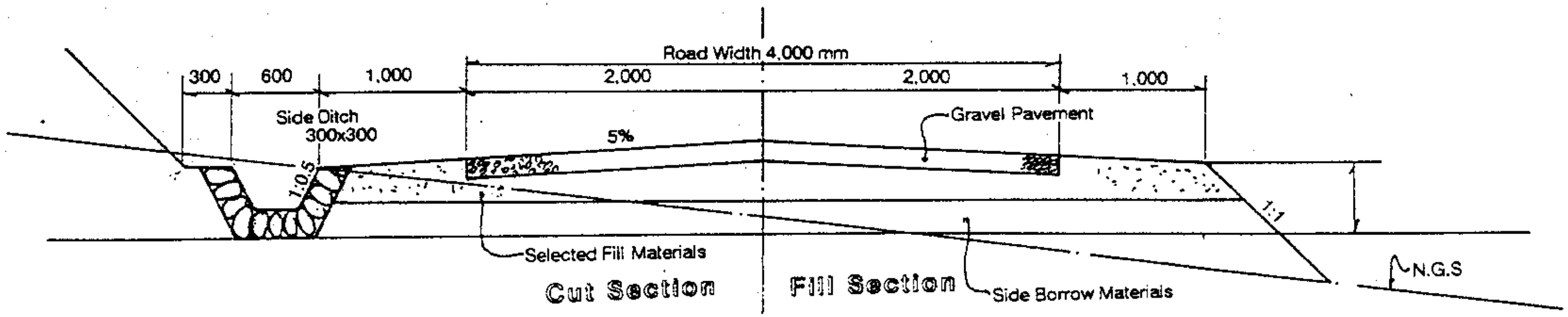
LEGEND

Concrete Pavement Portion

Figure I.3.2. Typical Cross Section of Village Road

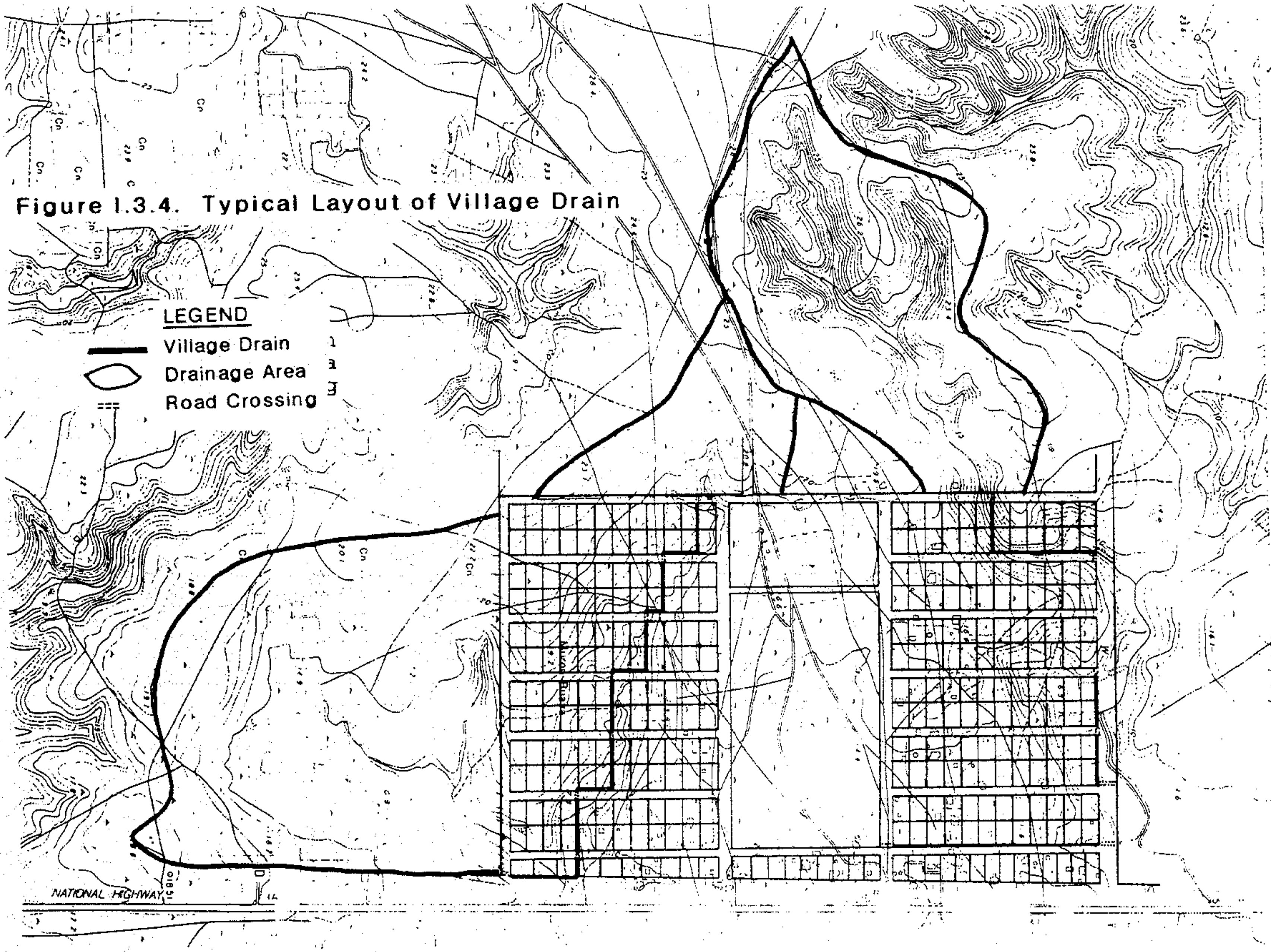


MAIN VILLAGE ROAD (MVR)



VILLAGE ROAD (VR)

Figure I.3.4. Typical Layout of Village Drain



I. 4 RURAL ELECTRIFICATION

I. 4. 1 General

Along the National road, there is a transmission line with 13.2 KV running through the Inagawan sub-colony from the Narra thermal power plant. The electricity cost per kwh increased to 3.75 pesos (with minimum charge of 37.5 pesos) from the former charge of 3.6461 pesos (with minimum charge of 36.461 pesos) in April, 1994. (refer to Figure I.4.1)

Some houses along the national road availed of electricity connected to the transmission line. The electricity with a common voltage of 220 to 240 volts and a frequency of 60 Hz is supplied after dropping down from 7,620 volts.

I. 4 .2 Proposed Facilities

The proposed power supply would be tapped from the transmission line. A 1-phase primary line of 1.8 km and a secondary line of 6.3 km would be installed along the village roads in Tagumpay Area. Four (4) transformers which will drop down the voltage from 7,620 to 220 volts will be proposed to be provided. About 70 wooden poles will be constructed to distribute electricity to the houses. (refer to Figure I.4.2)

For village 1 which is located along the national road, the existing system will also be applied. After dropping down the voltage from 7620 to 220 to 240, a distribution line will be aligned within the village area.

Since Village 2 is located farther from the power transmission line, a primary line of about 4 km will be necessary. A transformer will be installed to drop down the power from 7,620 volts to 220 to 240 volts within the village area. Distribution lines will be proposed to energize the farmers houses.

I. 4. 3 Other Necessary Costs

For the installation of the above facilities, the charges for internal wiring of 1,500 pesos, electric meter of 800 pesos and electrification fee of 1,200 pesos will be shouldered by the farmers.

Figure I.4.1 Present Power Supply System in Palawan

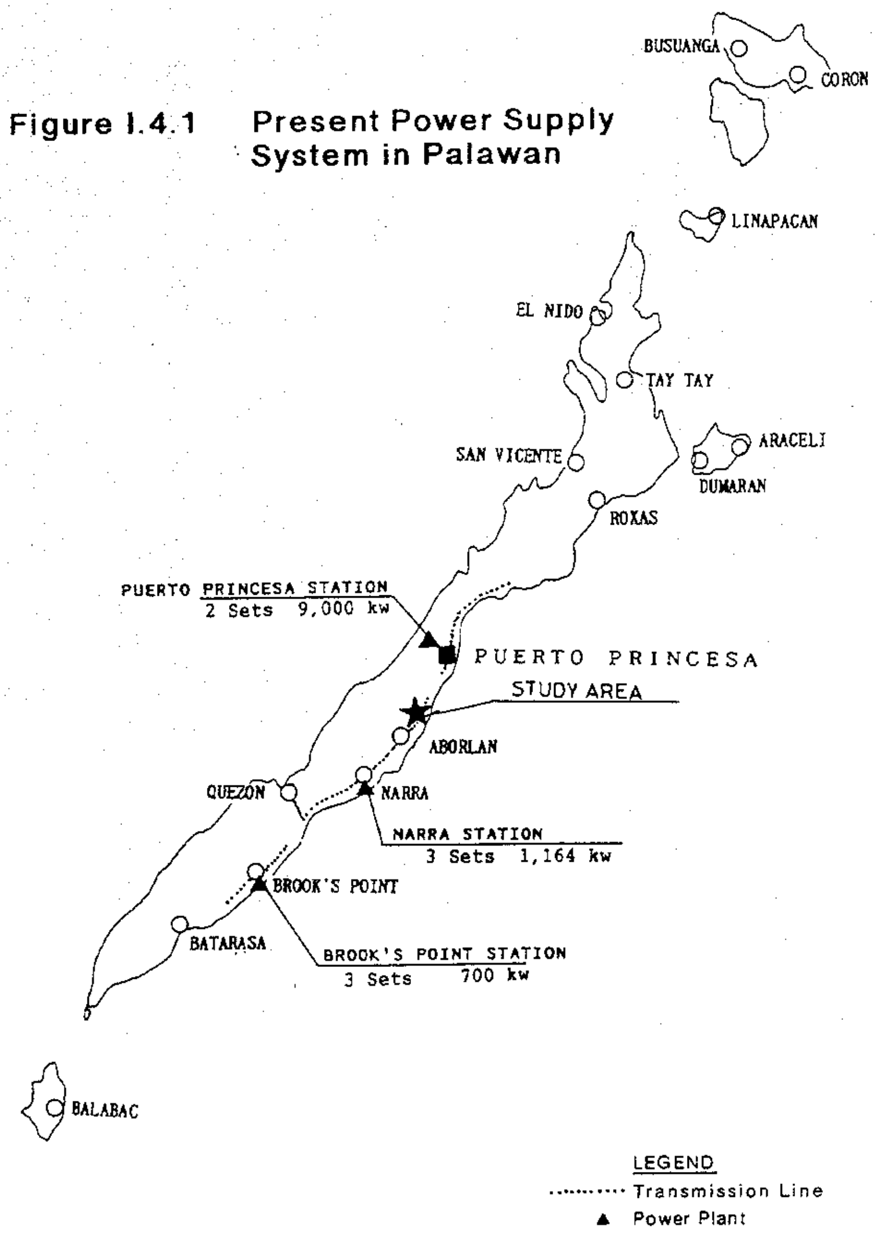


Figure I.4.2 Typical Layout of Rural Electrification