FIG. VII-3

MAXIMUM INUNDATION CONDITIONS ON THE PADDY FIELD DURING TYPHOON UNDANG

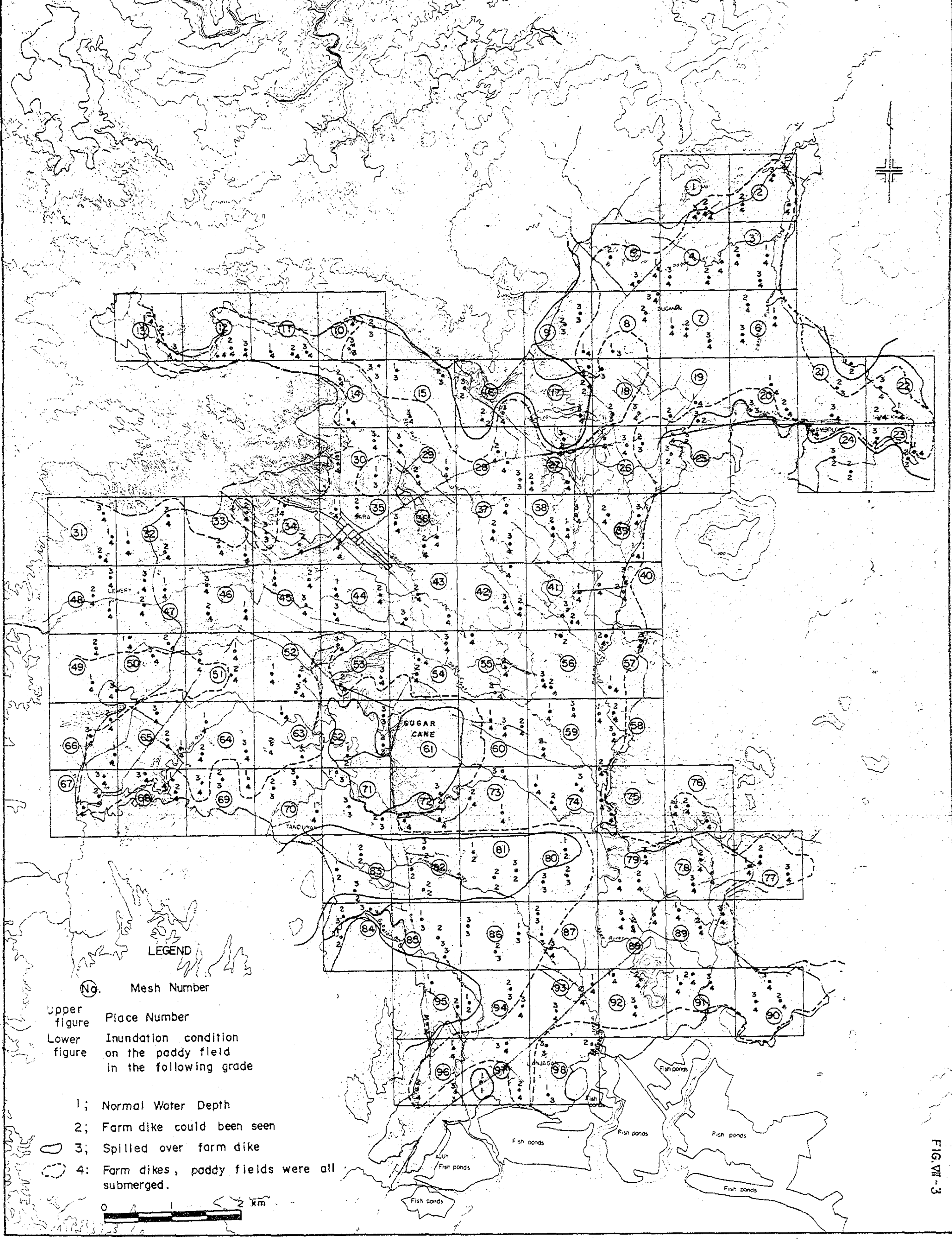
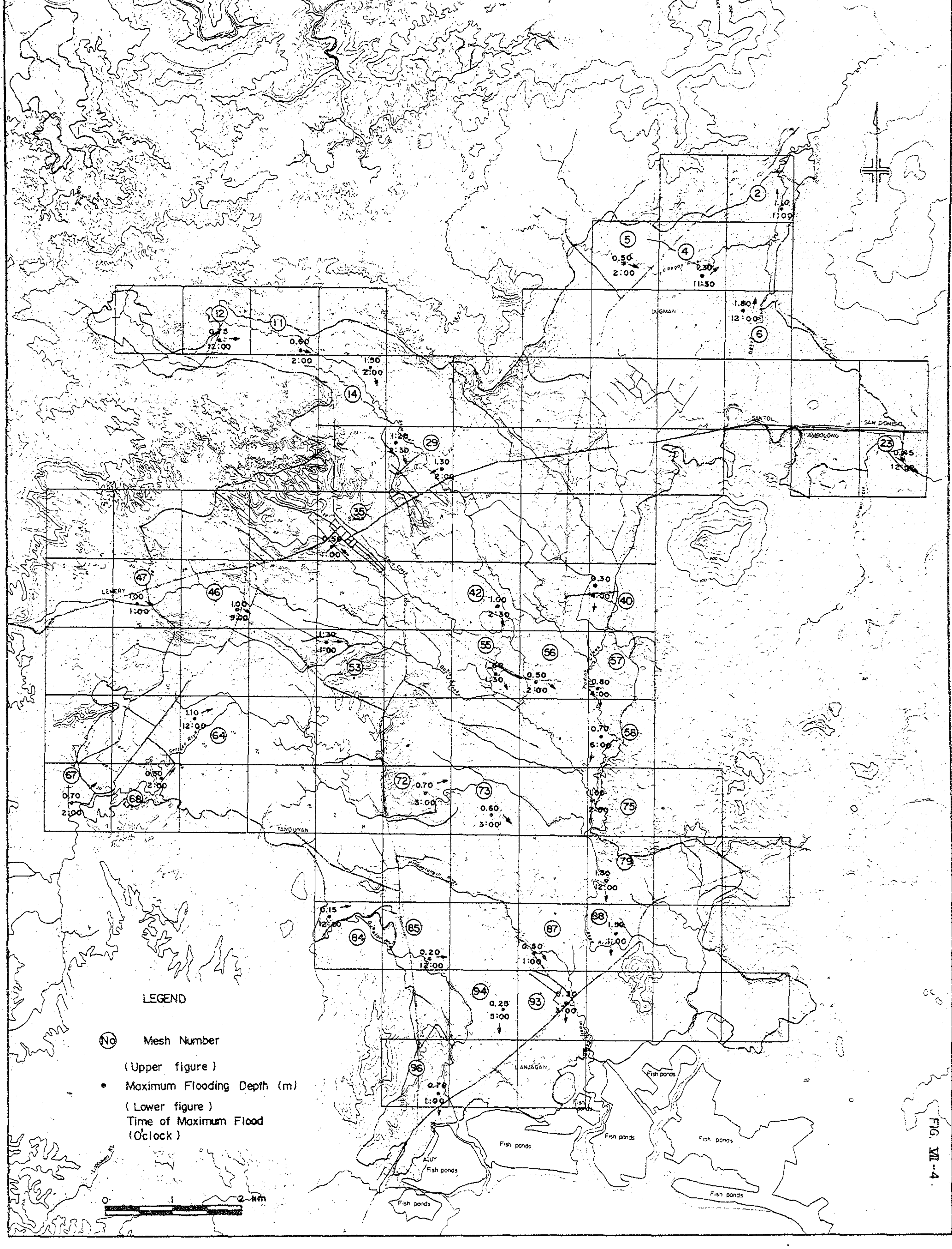


FIG. VII-4

MAXIMUM FLOODING DEPTH AND TIME DURING TYPHOON UNDANG



FIG·VII-5

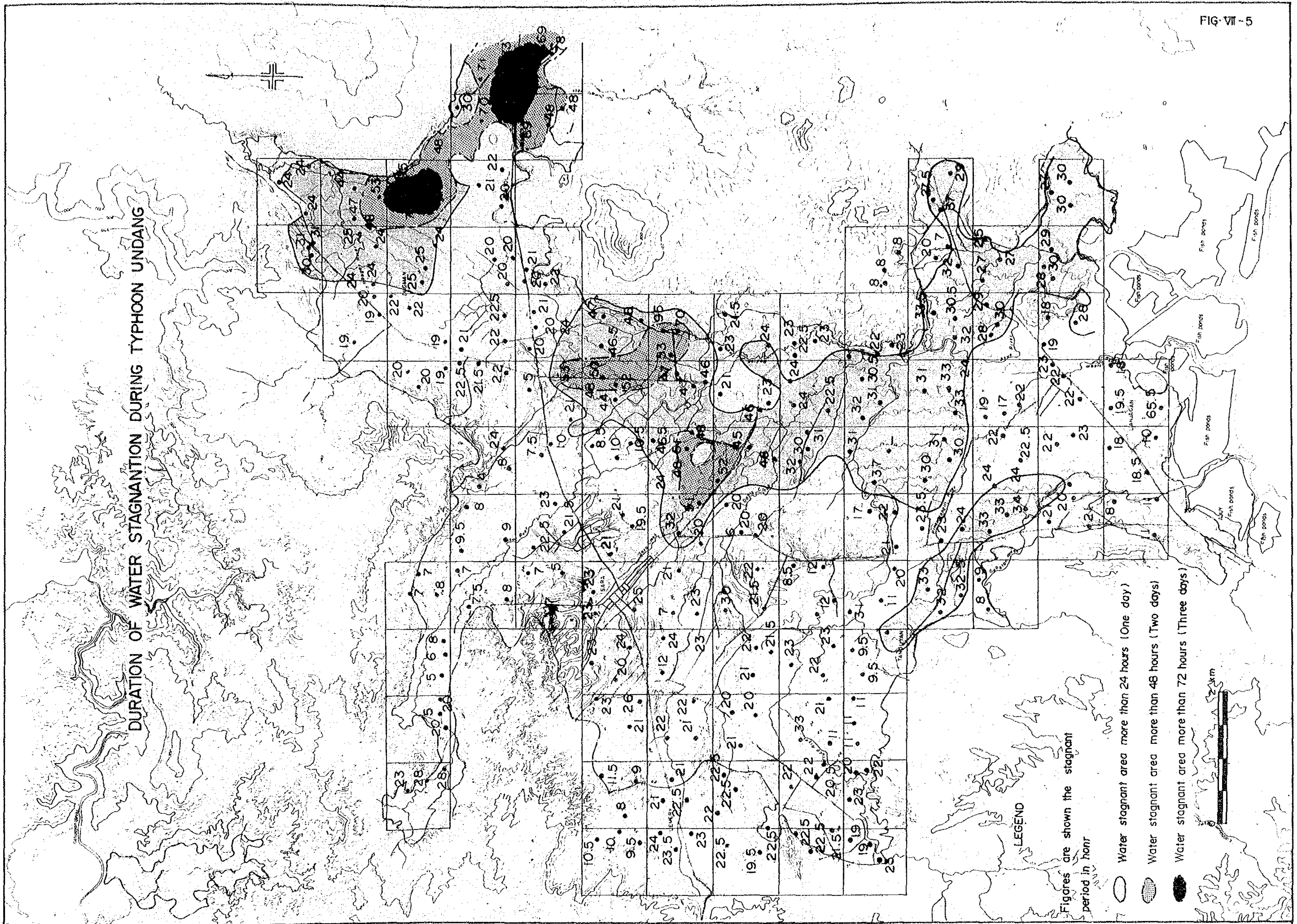
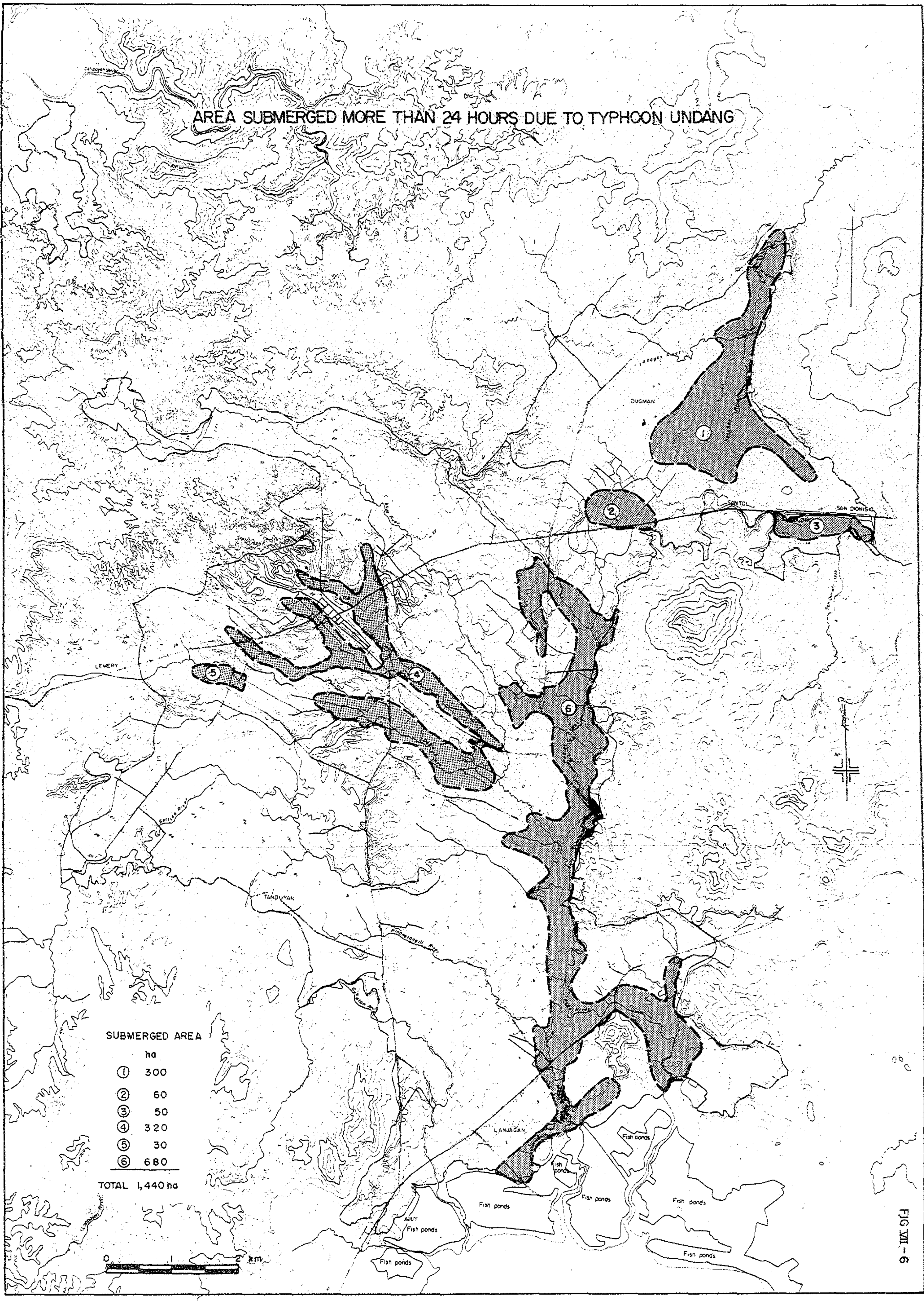

DURATION OF WATER STAGNANTION DURING TYPHOON UNDANG
LEGEND
Figures are shown the stagnant period in hour
Water stagnant area more than 24 hours (One day)
Water stagnant area more than 48 hours (Two days)
Water stagnant area more than 72 hours (Three days)
Fish ponds

FIG VII - 6


AREA SUBMERGED MORE THAN 24 HOURS DUE TO TYPHOON UNDANG
DUGMAN
SANTOL
SAN DIONISIO
LEMERY
TANDUYAN
LANJAGAN
AJUY
Fish ponds
SUBMERGED AREA
ha
① 300
② 60
③ 50
④ 320
⑤ 30
⑥ 680
TOTAL 1,440 ha
0 1 2 km

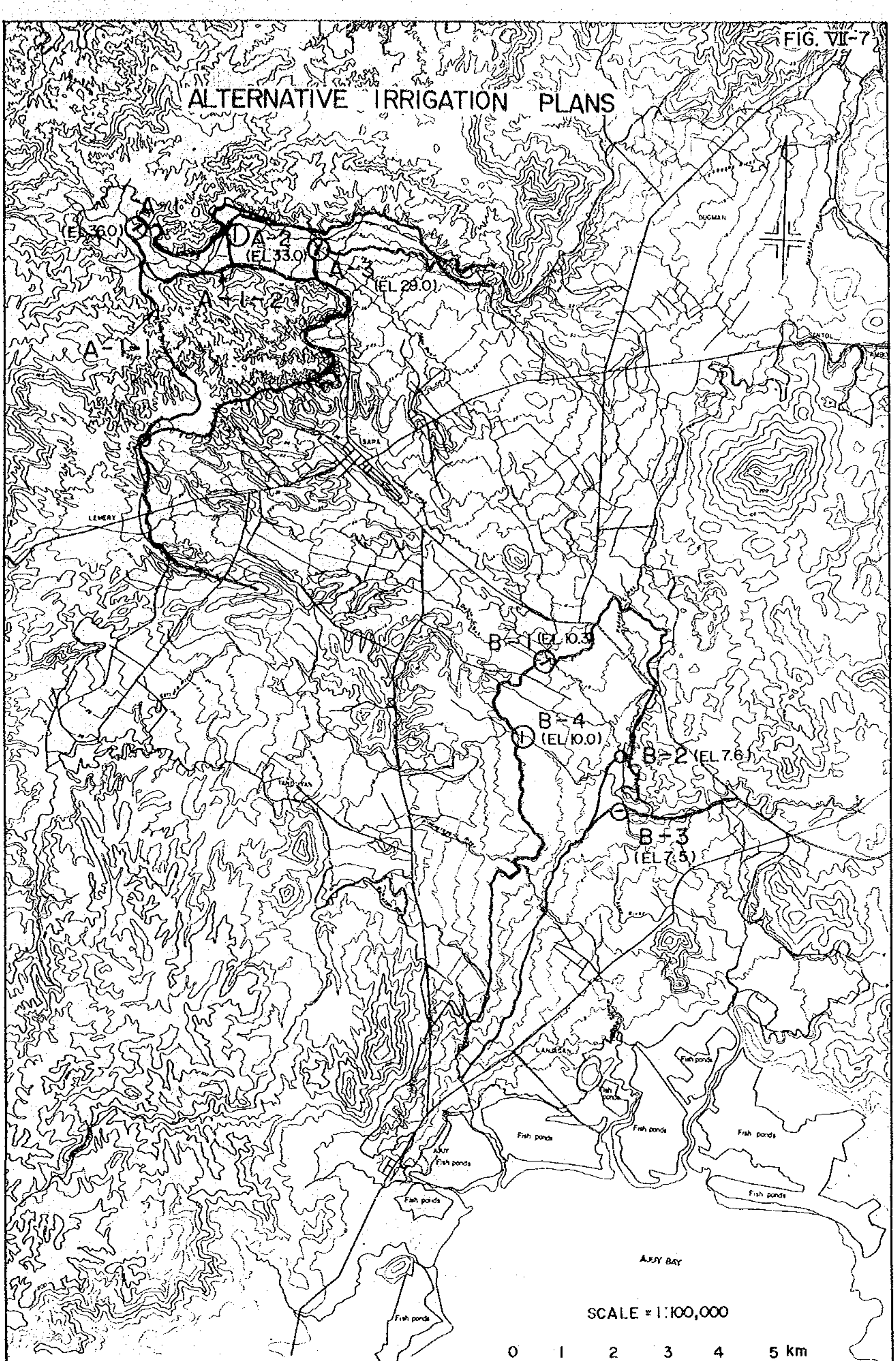


FIG. VII-7
ALTERNATIVE IRRIGATION PLANS
A-1
(EL 36.0)
A-2
(EL 33.0)
A-3
(EL 29.0)
A-1-2
A-1-1
B-1
(EL 10.3)
B-4
(EL 10.0)
B-2
(EL 7.6)
B-3
(EL 7.5)
Fish ponds
SCALE = 1:100,000
0 1 2 3 4 5 km

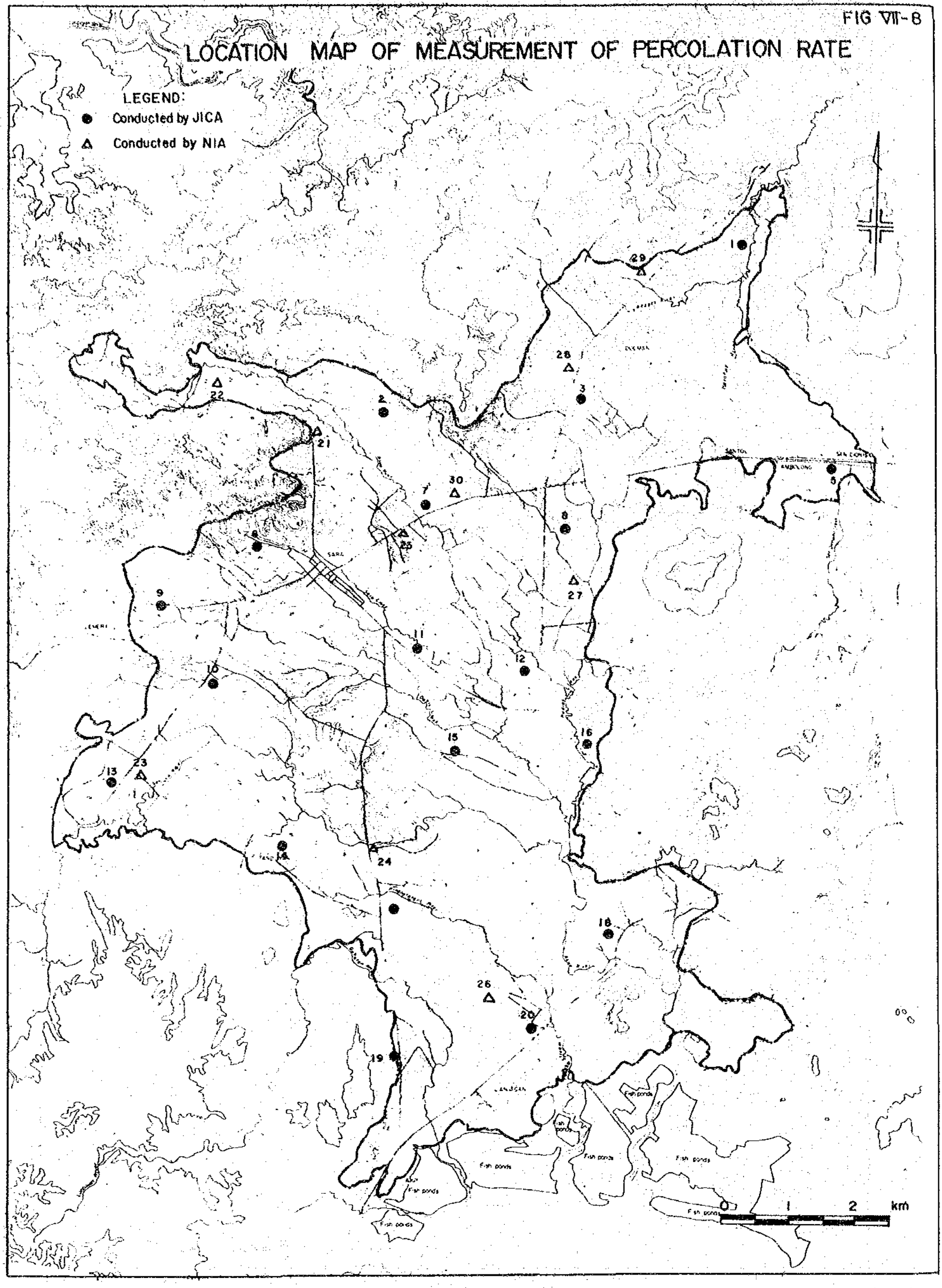


FIG VII-8
LOCATION MAP OF MEASUREMENT OF PERCOLATION RATE
LEGEND:
Conducted by JICA
Conducted by NIA
0 1 2 km
Fish ponds

FIG. VII - 9

MAXIMUM CONSUMPTIVE USE FOR PADDY

(Design Water Requirements)

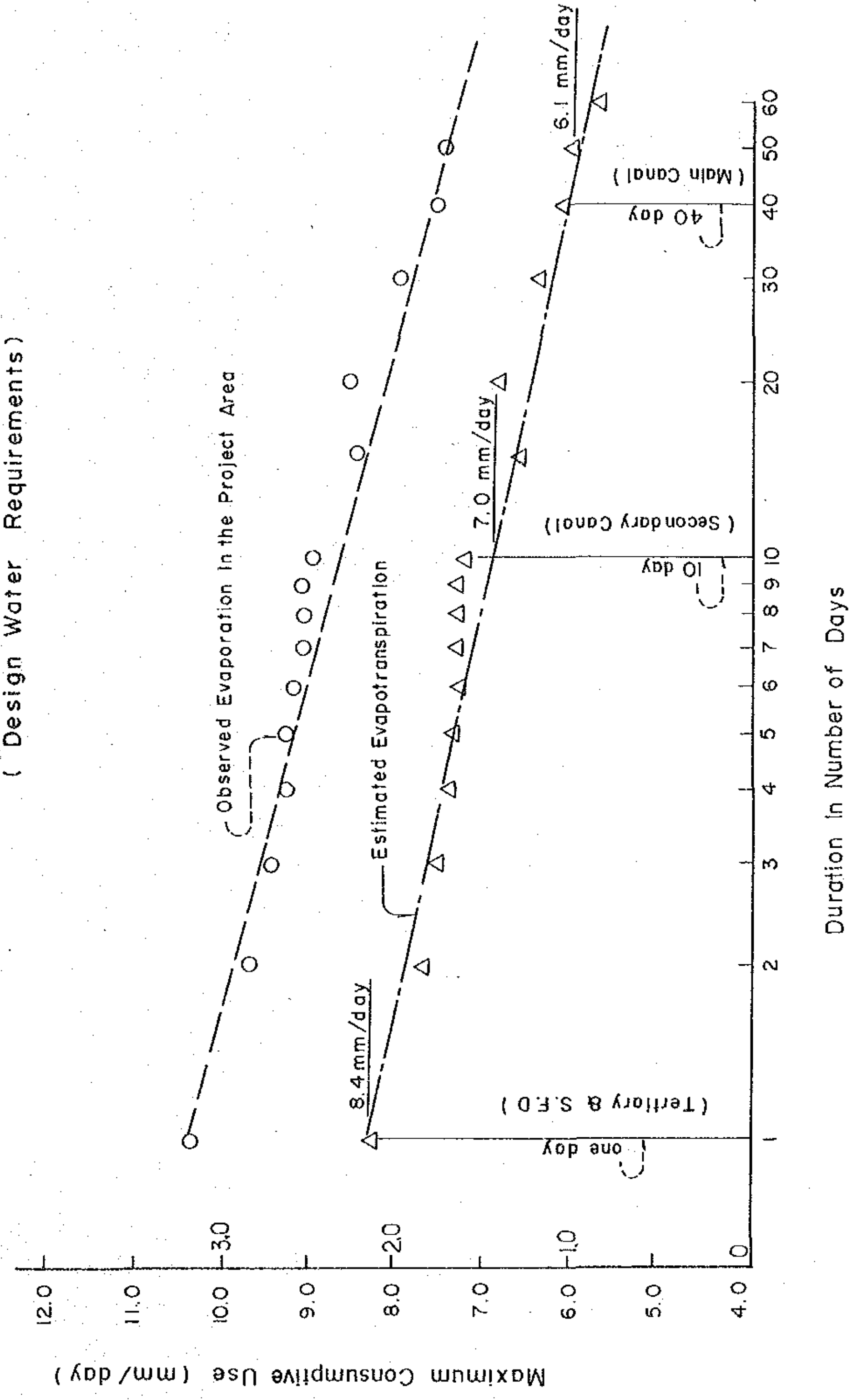


FIG. VII - 10

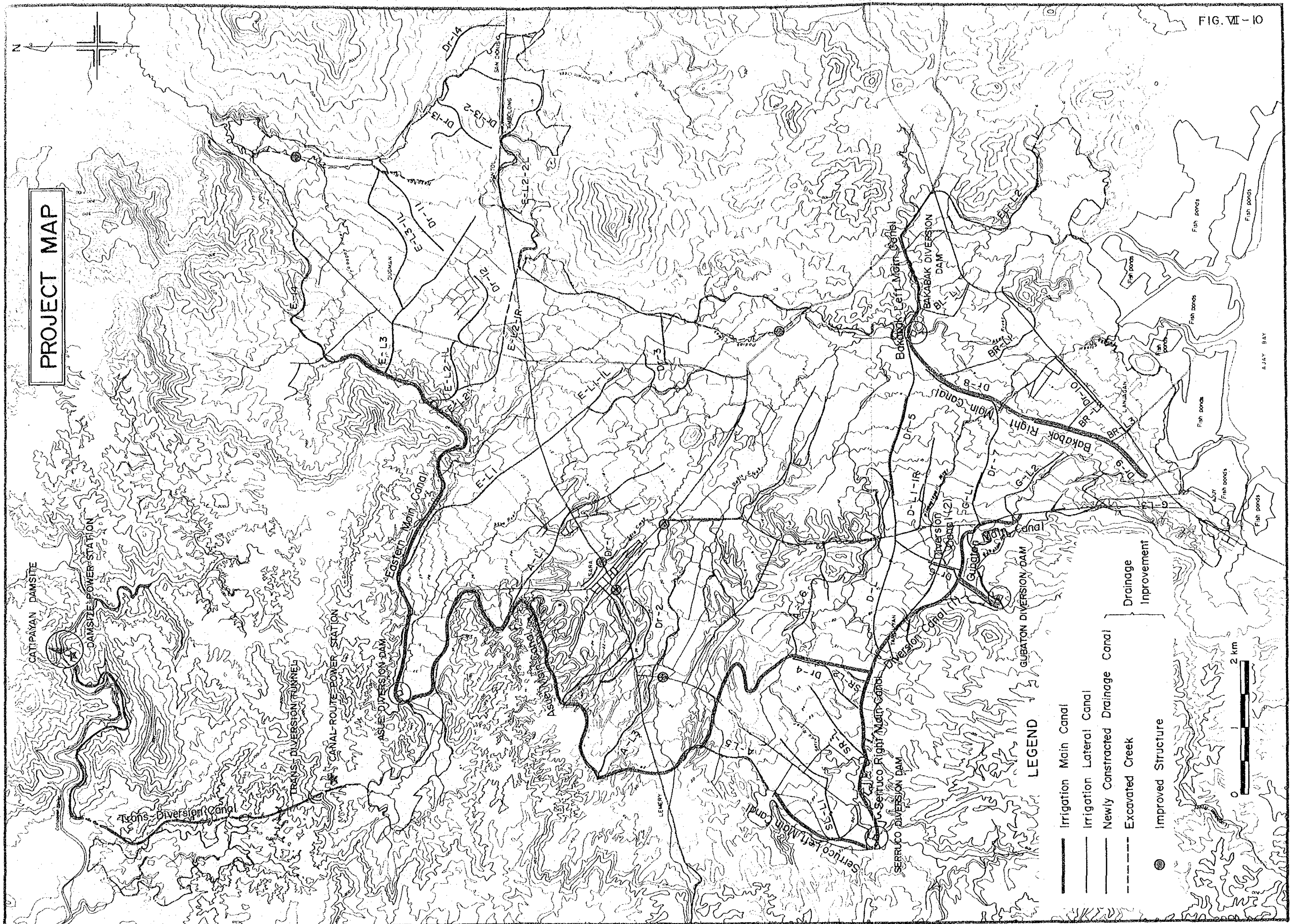

PROJECT MAP
N
CATIPAYAN DAMSITE
DAMSITE POWER STATION
TRANS-DIVERSION TUNNEL
CANAL-ROUTE POWER STATION
ASUE DIVERSION DAM
Trans-Diversion Canal
Eastern Main Canal
Asue Main Canal
SERRUCO DIVERSION DAM
Serruco Left Main Canal
Serruco Right Main Canal
Diversion Canal (1)
Diversion Canal (2)
GUBATON DIVERSION DAM
Gubaton Main Canal
BAKABAK DIVERSION DAM
Bakabak Left Main Canal
Bakabak Right Main Canal
Fish ponds
AJAY BAY
LEGEND
Irrigation Main Canal
Irrigation Lateral Canal
Newly Constracted Drainage Canal
Excavated Creek
Drainage Inprovement
Improved Structure
0 1 2 km

FIG. VII - 11

SCHEMATIC DIAGRAM OF PROPOSED IRRIGATION NETWORK

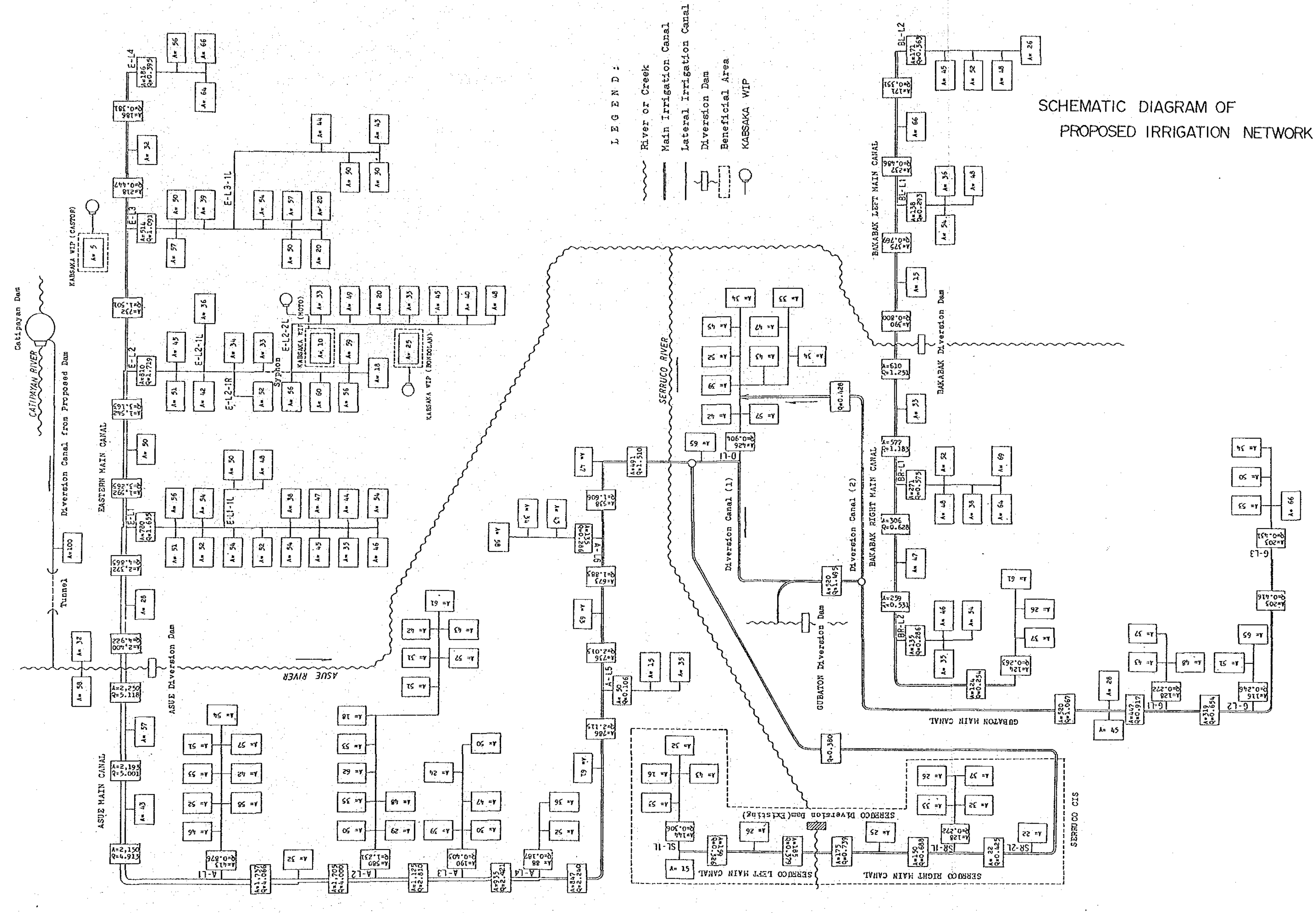


FIG. VII - 12

SCHEMATIC DIAGRAM OF PROPOSED IRRIGATION SYSTEM

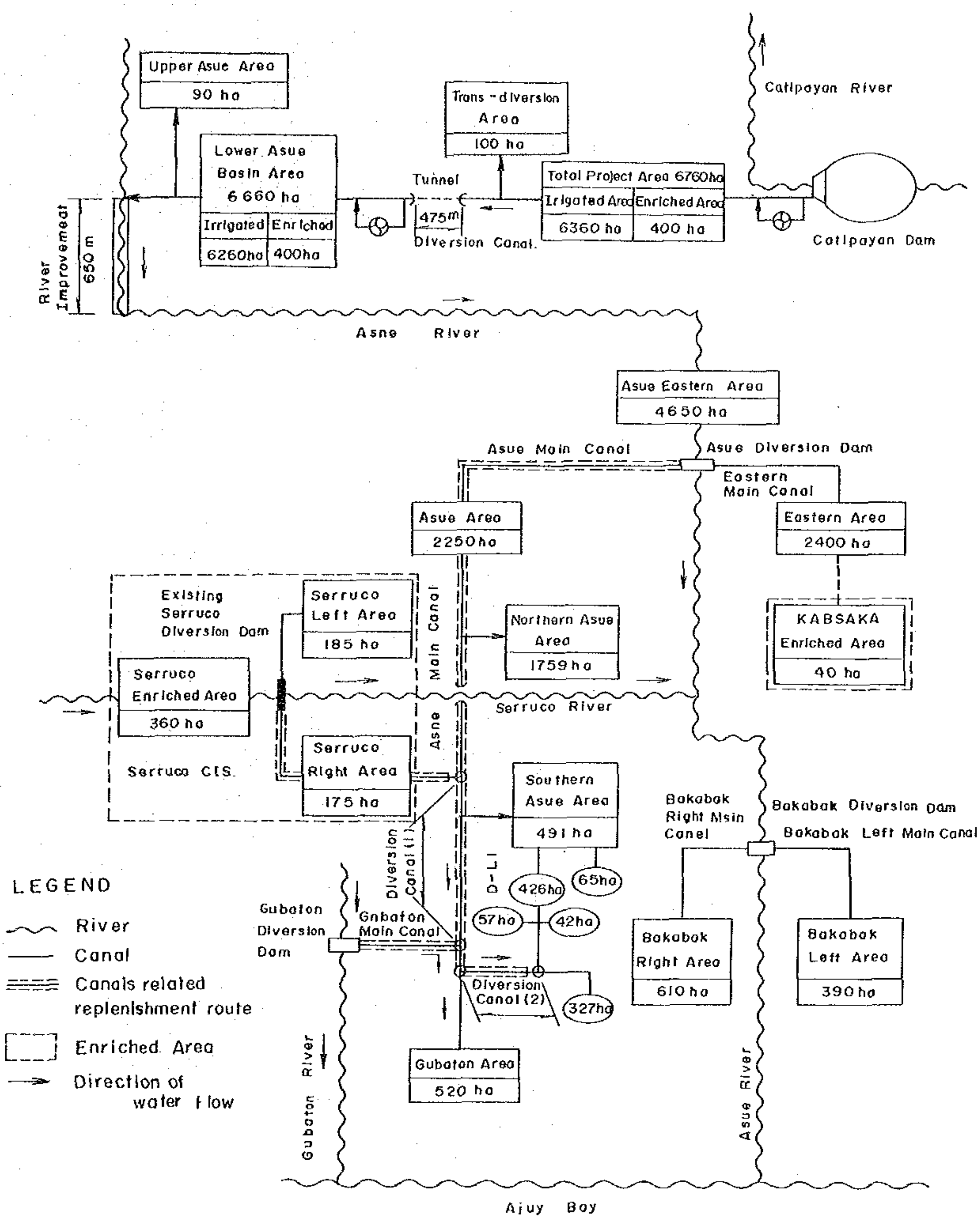
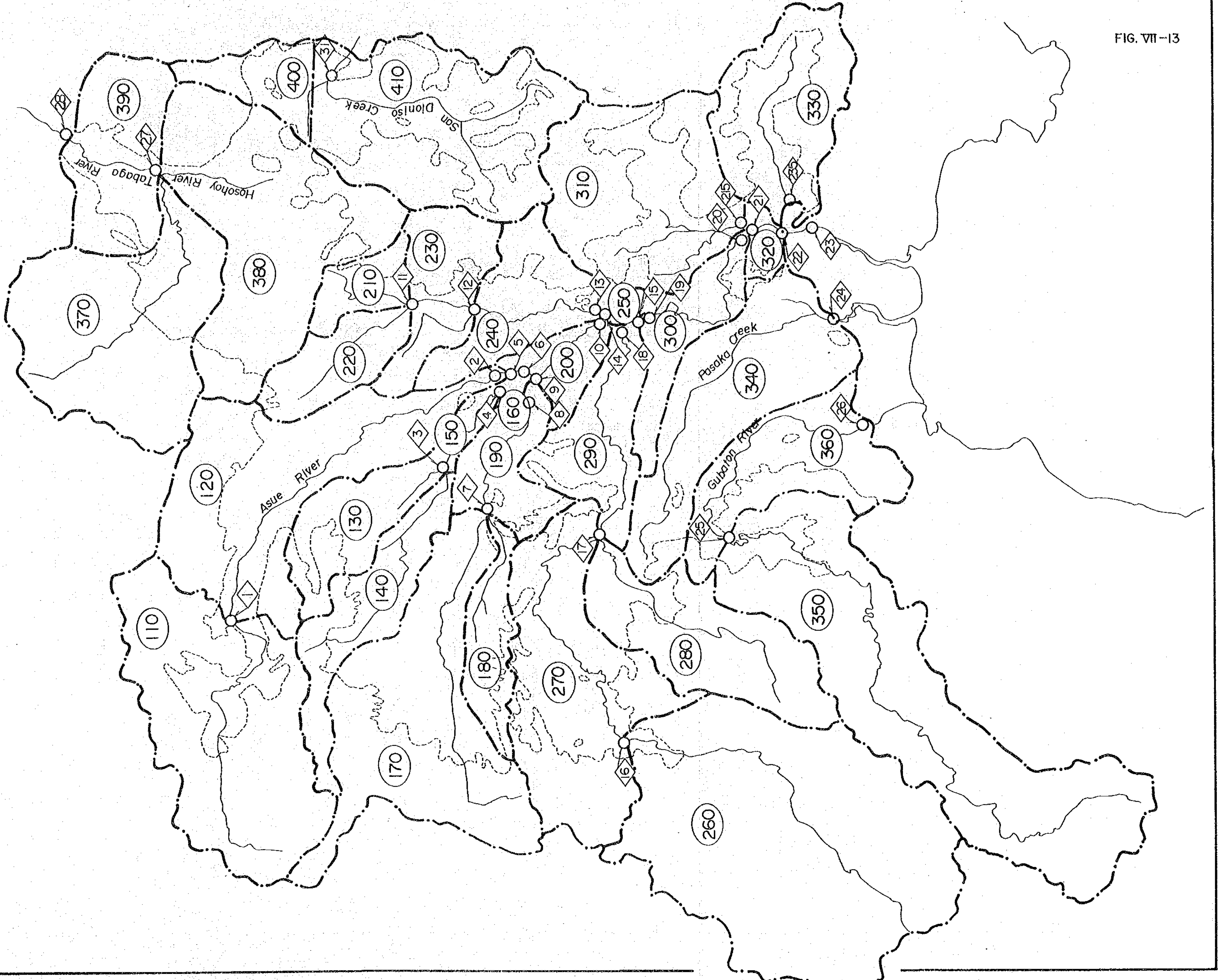


FIG. VII-13



SUB-CATCHMENT AREA IN THE ASUE BASIN

SCHEMATIC DIAGRAM OF THE DRAINAGE SYSTEM IN THE ASUE BASIN

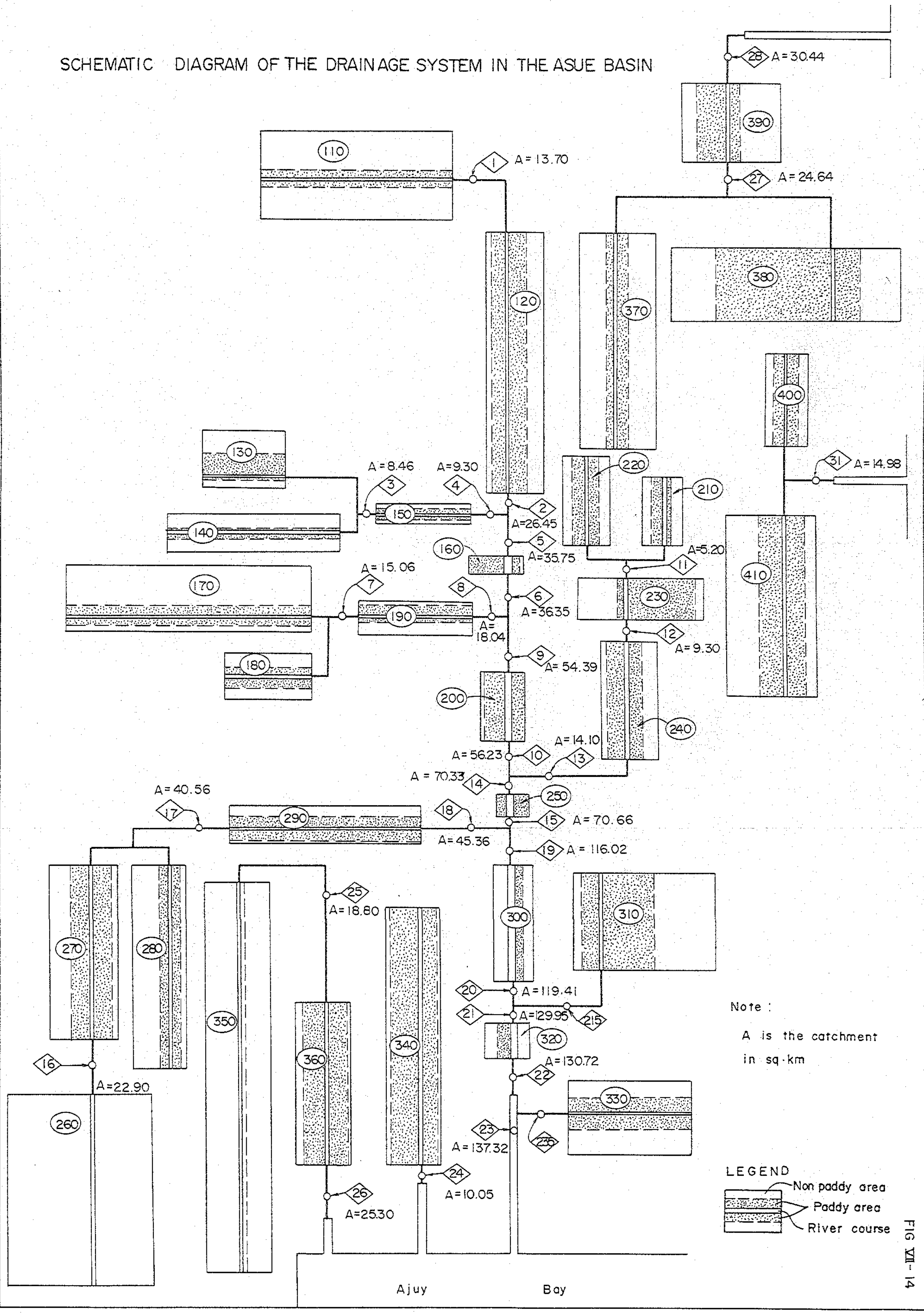


FIG VII-14

Fig. VII-15

DESIGN RAINFALL PATTERN FOR DRAINAGE

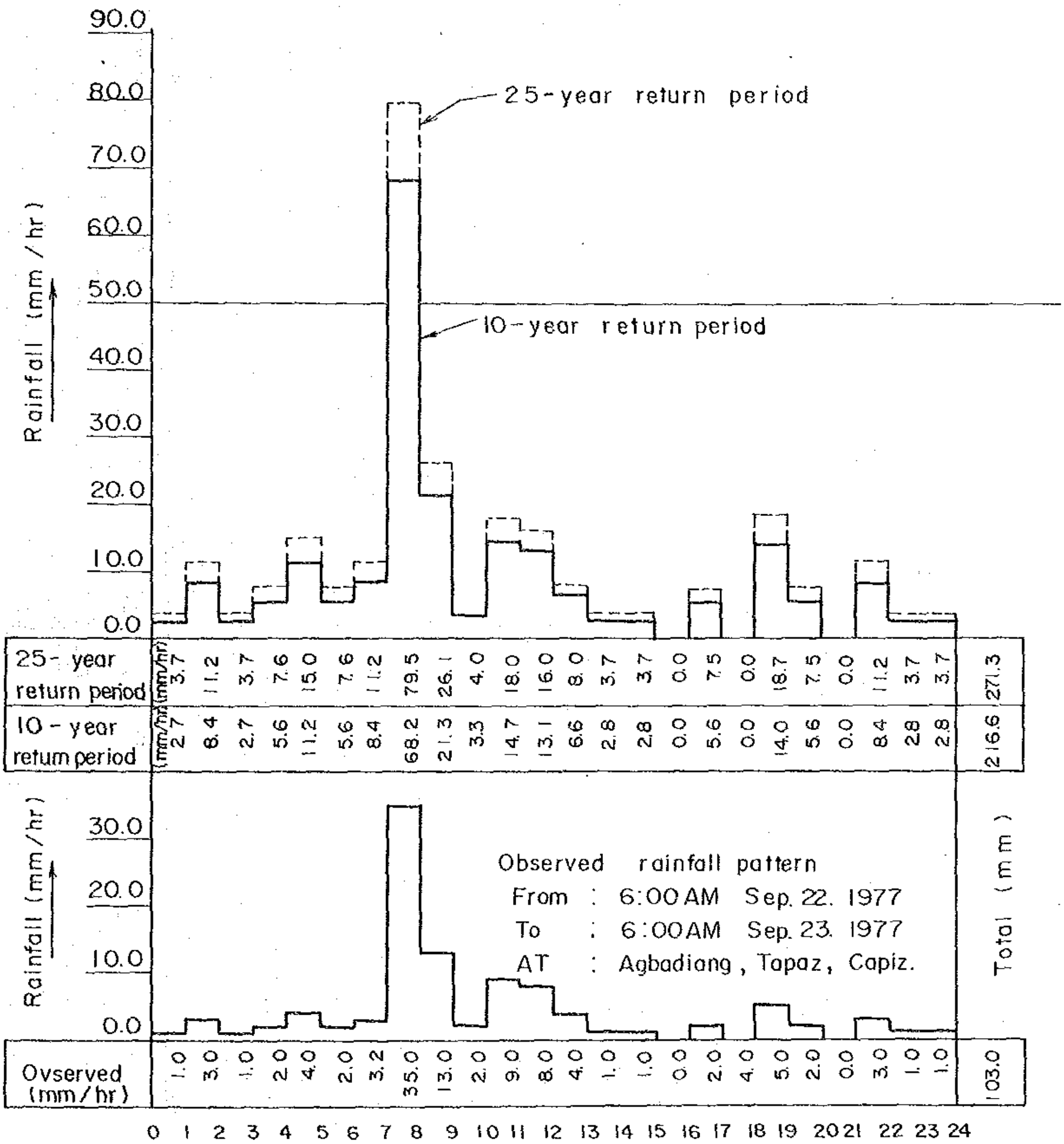


FIG. VII-16

CUMULATIVE RAINFALL / CUMMULATIVE LOSS CURVE

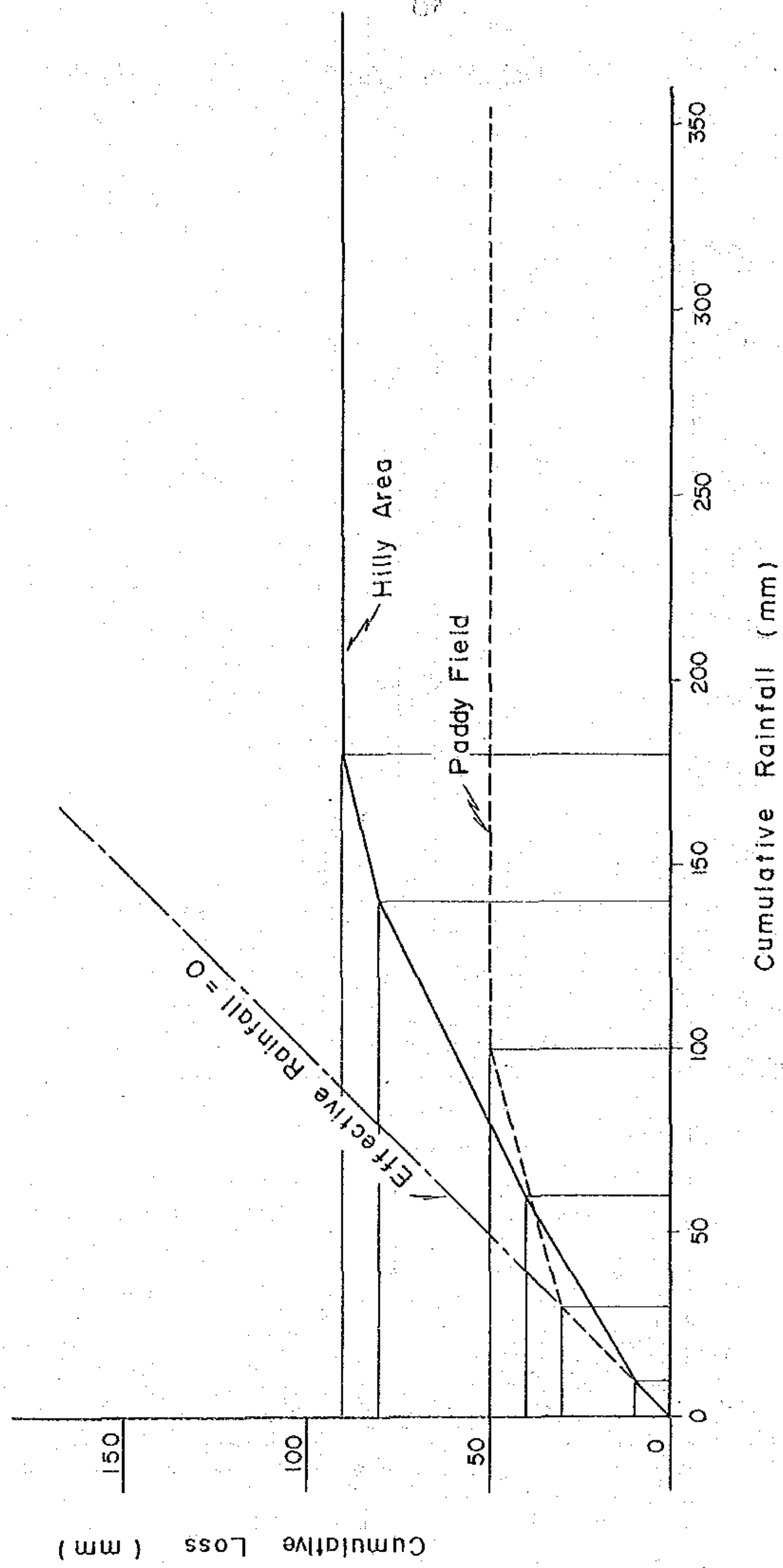
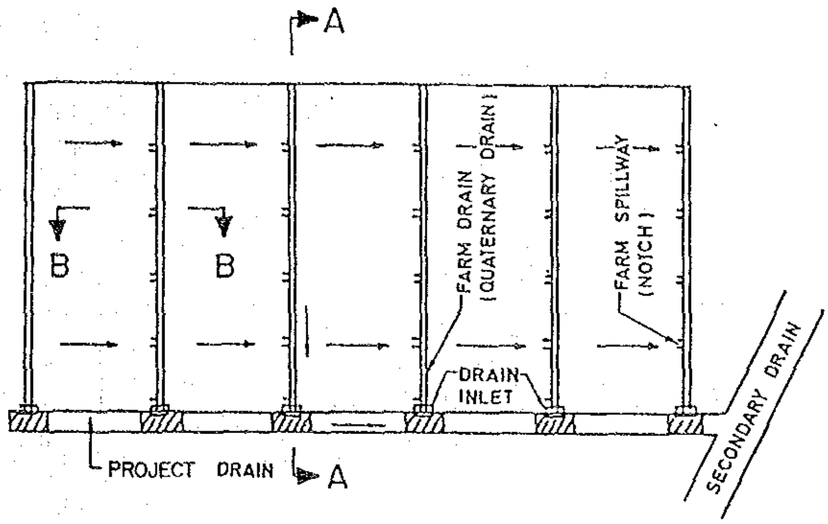
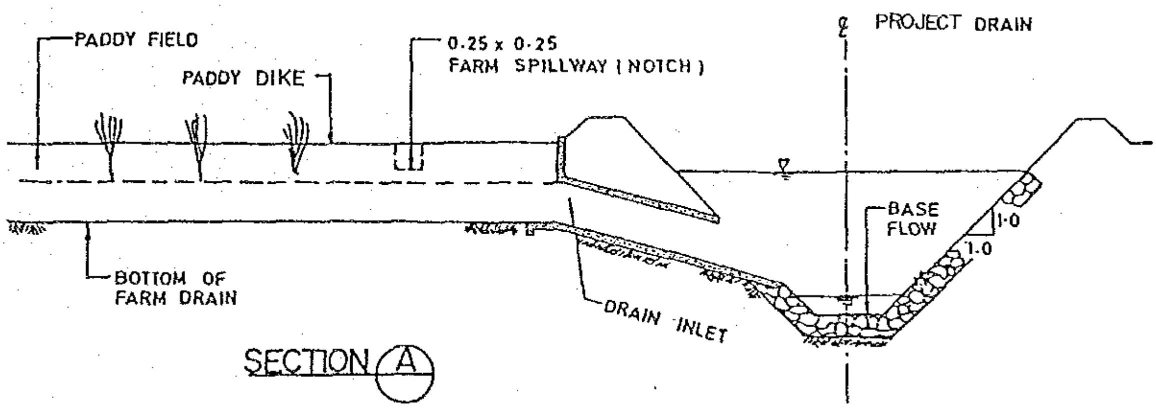


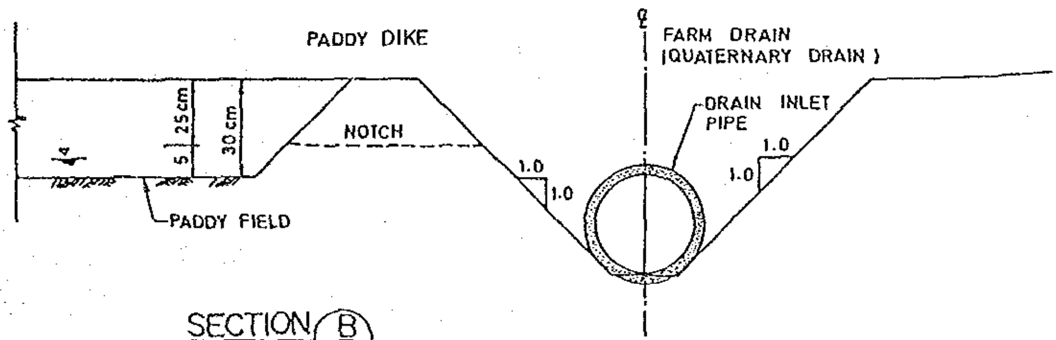
FIG.VII-17

DRAINAGE SYSTEM IN THE PADDY FIELD





SECTION (A)



SECTION (B)

Fig. VII-18

CHARACTERISTICS OF RUNOFF FROM PADDY FIELD

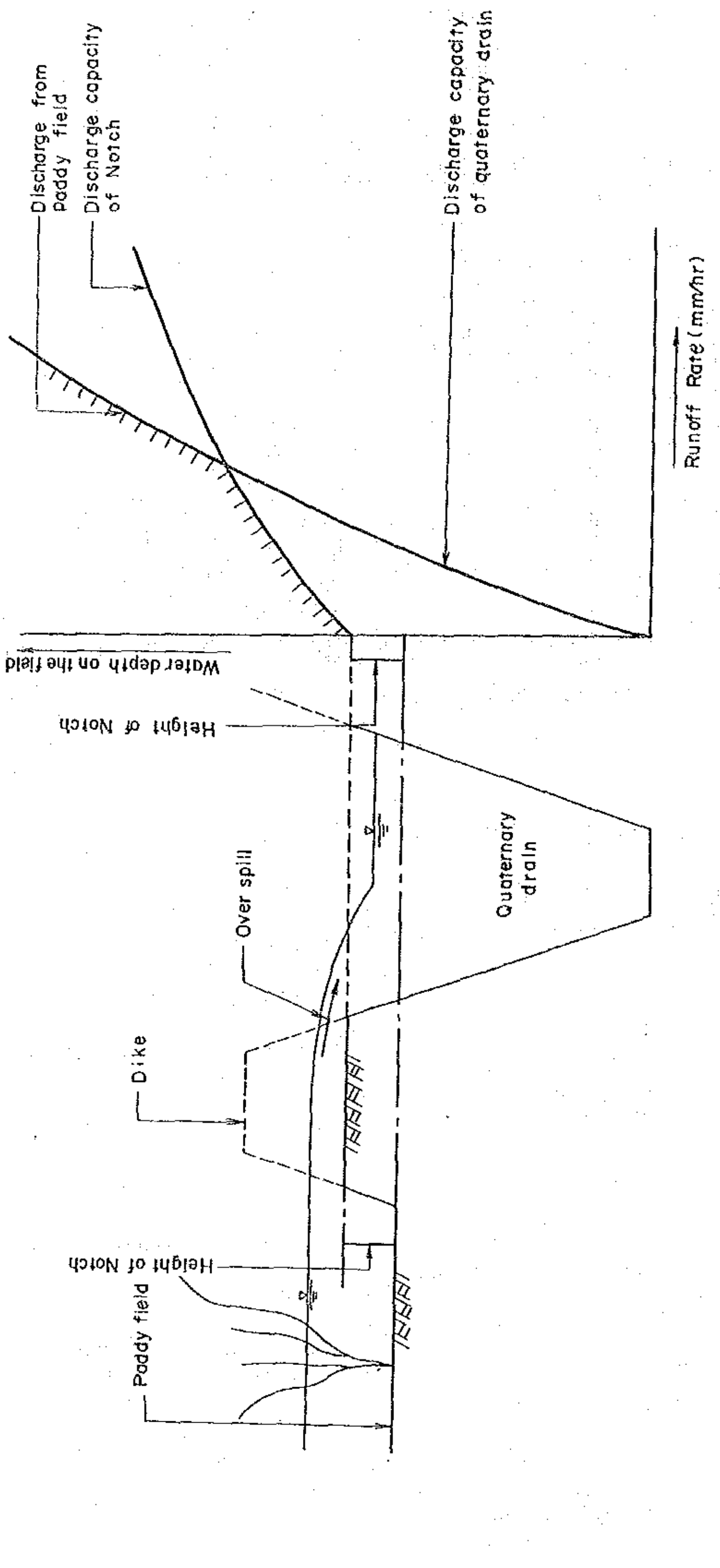
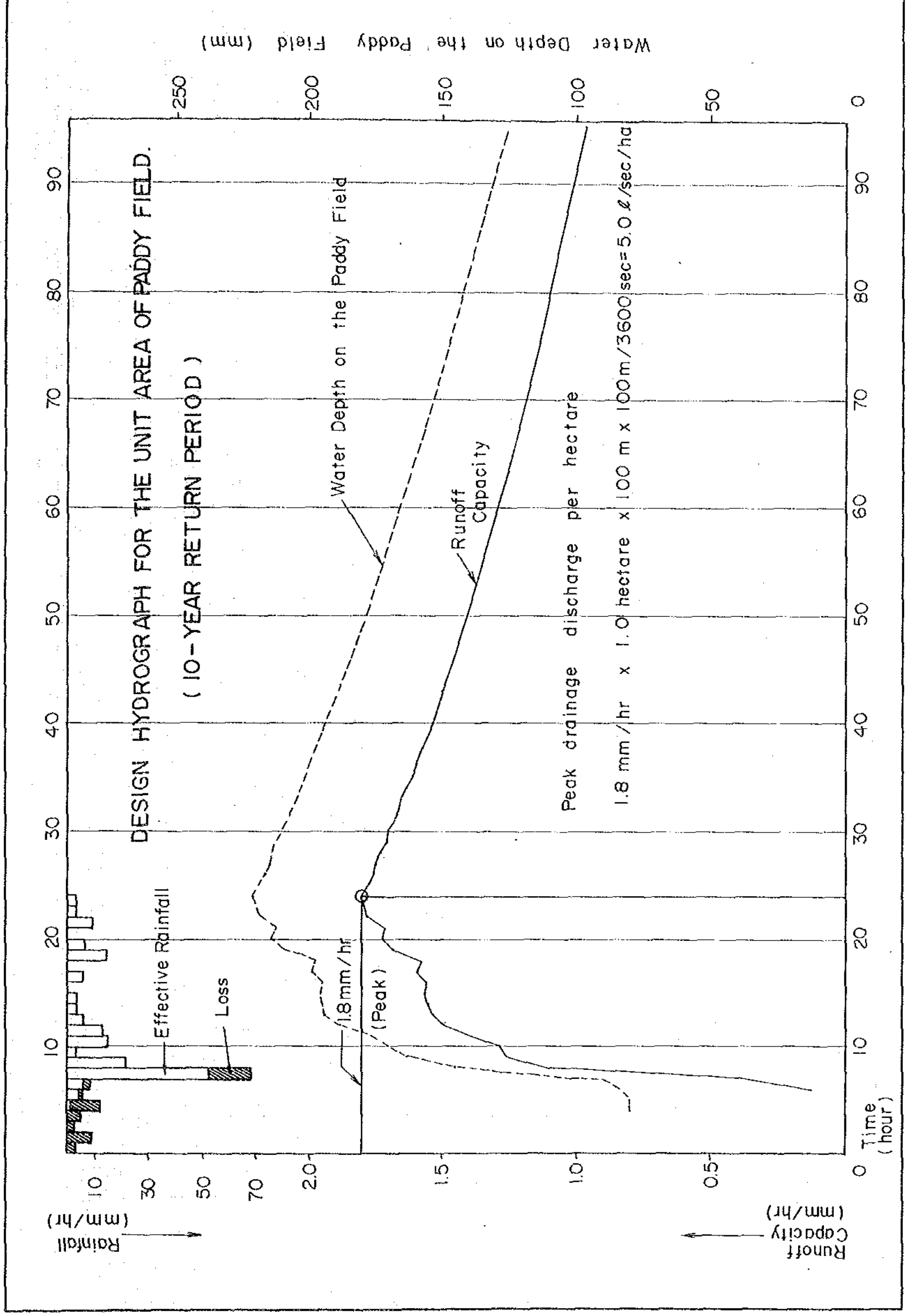
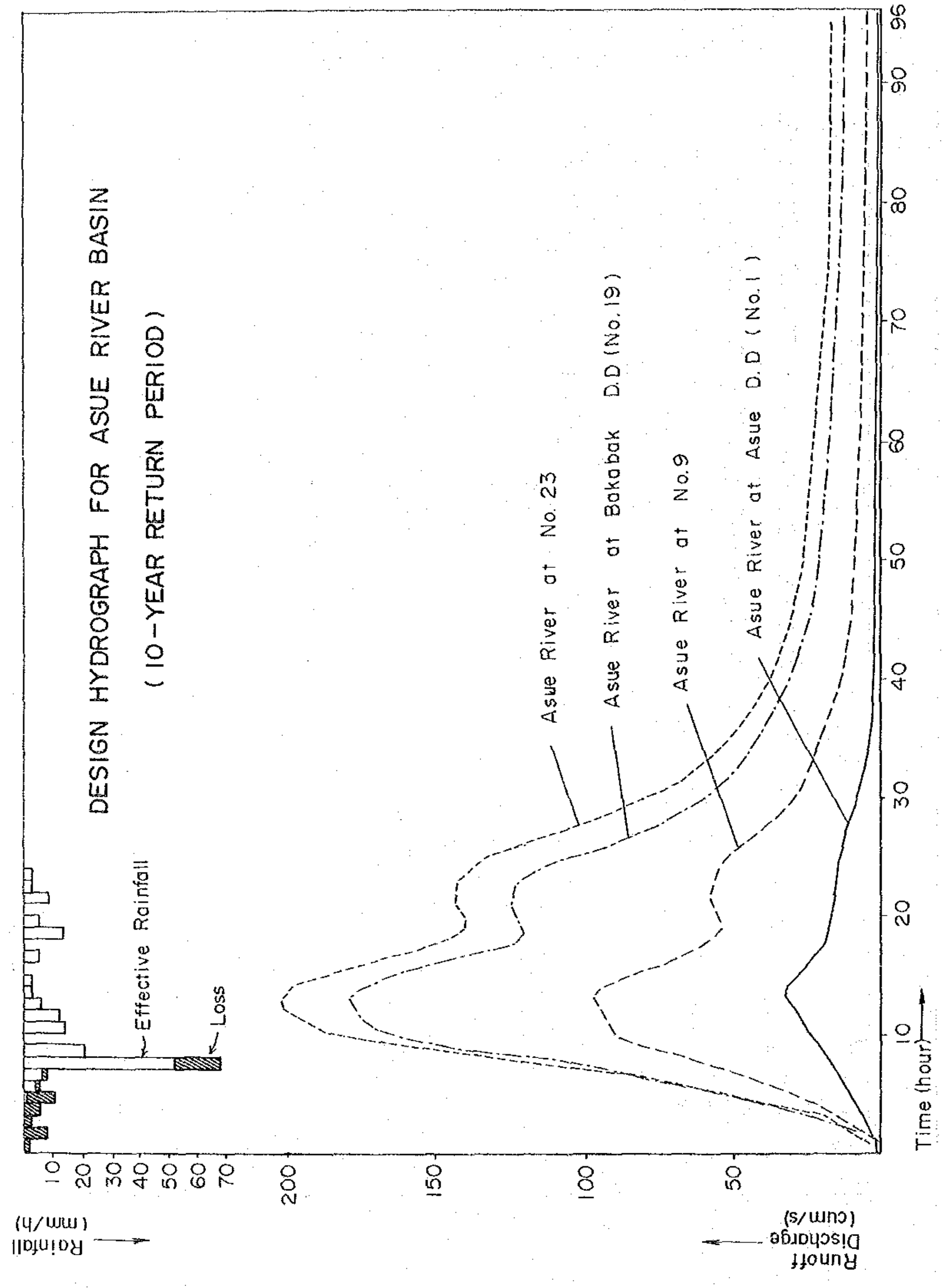


Fig. VII - 19



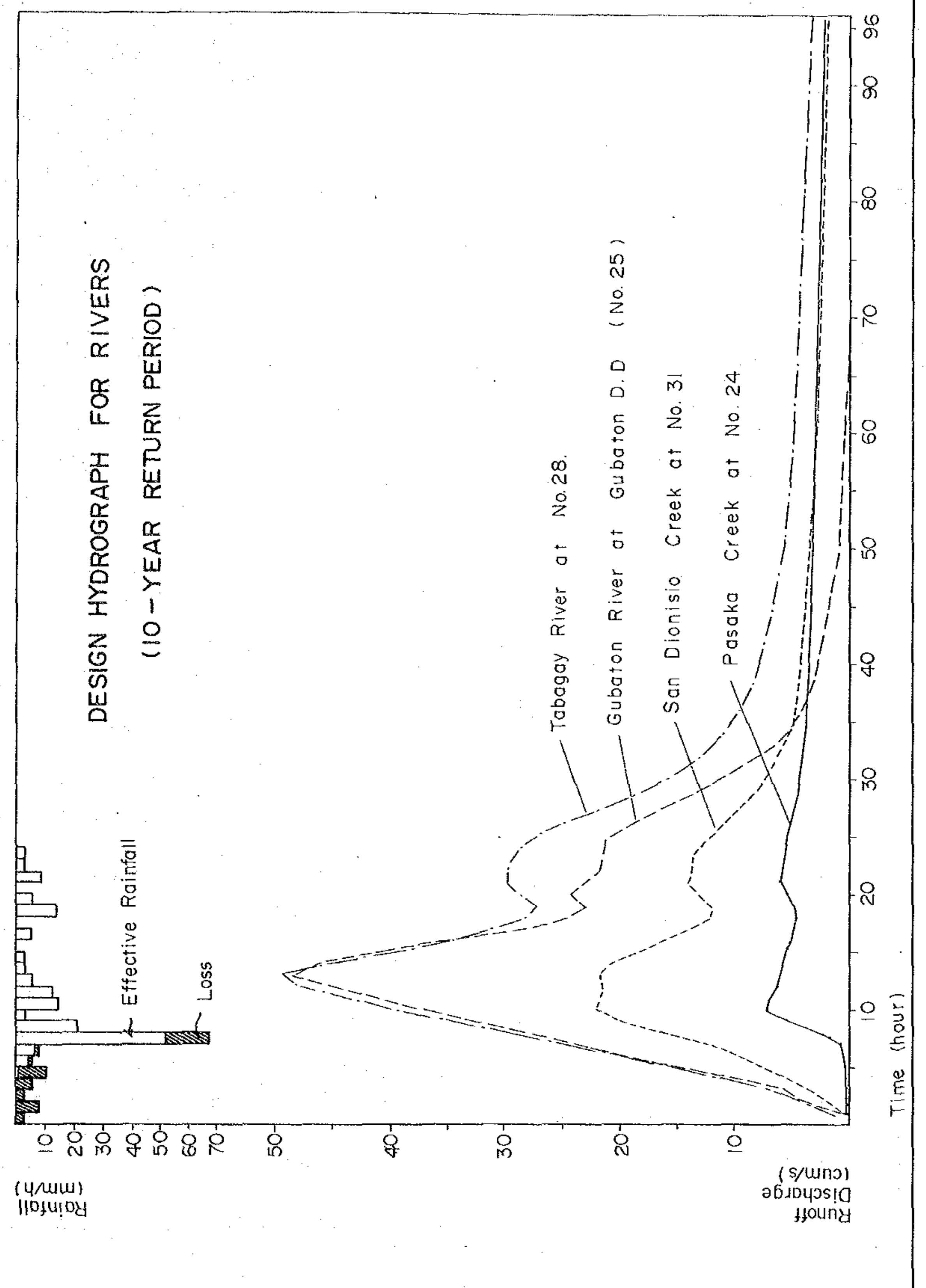
DESIGN HYDROGRAPH FOR THE UNIT AREA OF PADDY FIELD.
(10-YEAR RETURN PERIOD)
Water Depth on the Paddy Field (mm)
Water Depth on the Paddy Field
Runoff Capacity
Effective Rainfall
Loss
1.8 mm/hr
(Peak)
Peak drainage discharge per hectare
1.8 mm/hr x 1.0 hectare x 100 m x 100 m/3600 sec = 5.0 ℓ/sec/ha
Time (hour)
Rainfall (mm/hr)
Runoff Capacity (mm/hr)
0
10
20
30
40
50
60
70
80
90
250
200
150
100
50
2.0
1.5
1.0
0.5

Fig. VII-20



DESIGN HYDROGRAPH FOR ASUE RIVER BASIN
(10-YEAR RETURN PERIOD)
Effective Rainfall
Loss
Asue River at No.23
Asue River at Bakabak D.D (No.19)
Asue River at No.9
Asue River at Asue D.D (No.1)
Rainfall (mm/h)
Runoff Discharge (cum/s)
Time (hour)
0
10
20
30
40
50
60
70
90
96
100
150
200

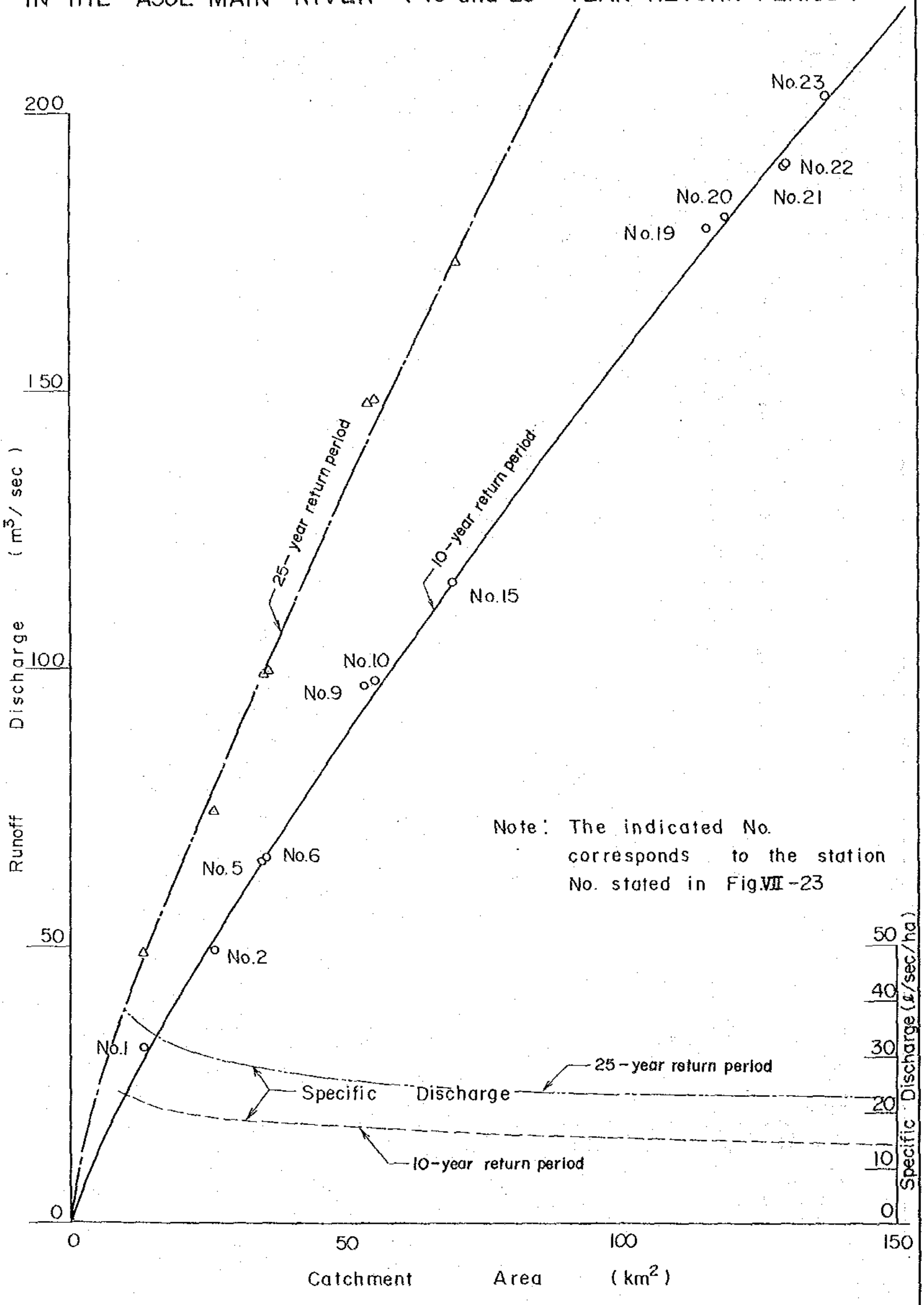
Fig. VII-21



DESIGN HYDROGRAPH FOR RIVERS
(10 - YEAR RETURN PERIOD)
Tabagay River at No.28.
Gubaton River at Gubaton D. D (No. 25)
San Dionisio Creek at No. 31
Pasaka Creek at No. 24
Effective Rainfall
Loss
Rainfall (mm/h)
Runoff Discharge (cum/s)
Time (hour)
10
20
30
40
50
60
70
50
40
30
20
10
10
20
30
40
50
60
70
80
90
96

Fig. VII-22

RELATIONSHIP BETWEEN PEAK DISCHARGE VS CATCHMENT AREA IN THE ASUE MAIN RIVER (10 and 25 - YEAR RETURN PERIOD)



DRAINAGE DISCHARGE FOR A 10-YEAR RETURN PERIOD

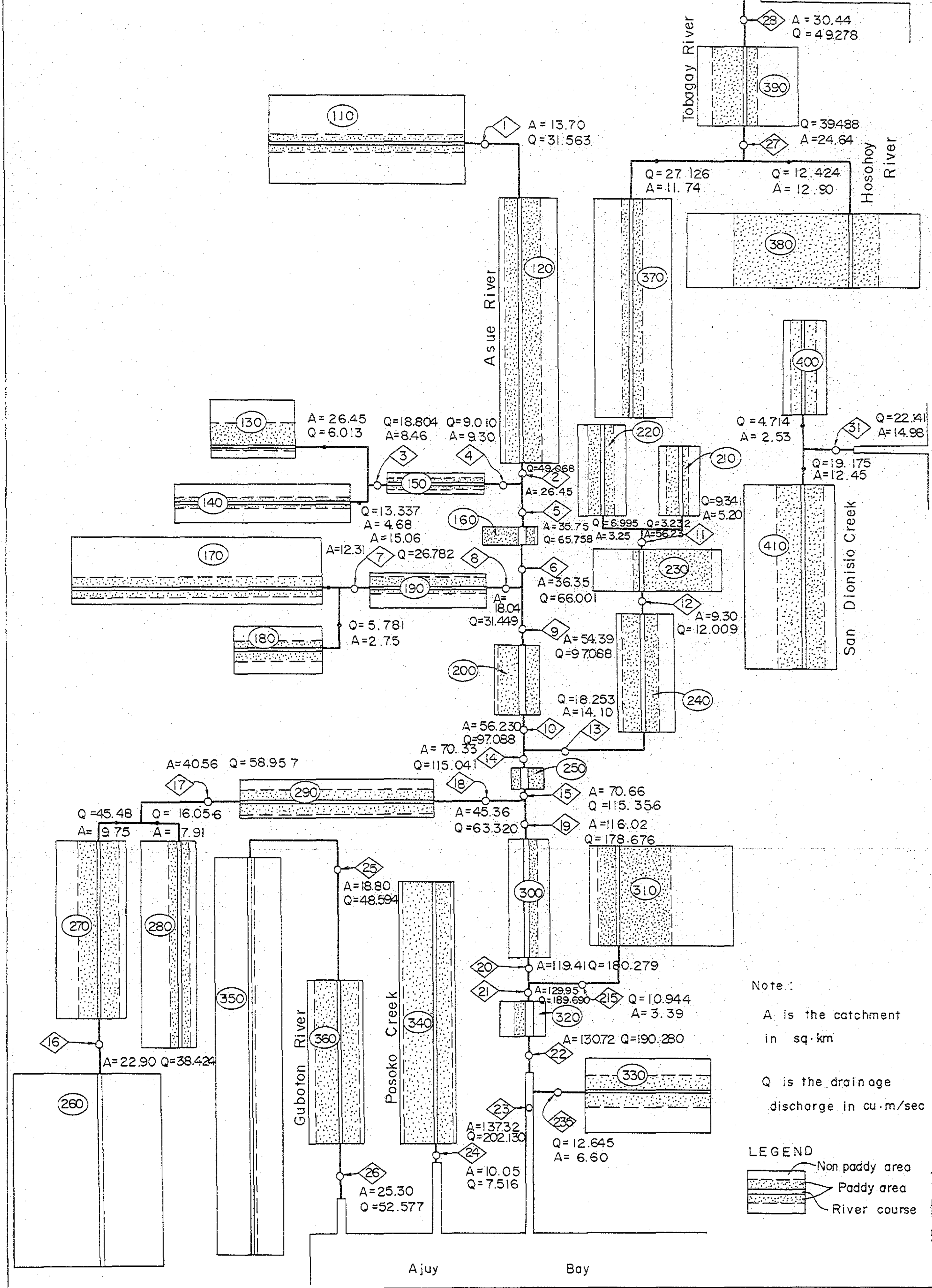


FIG VII-23

DRAINAGE DISCHARGE FOR A 25-YEAR RETURN PERIOD

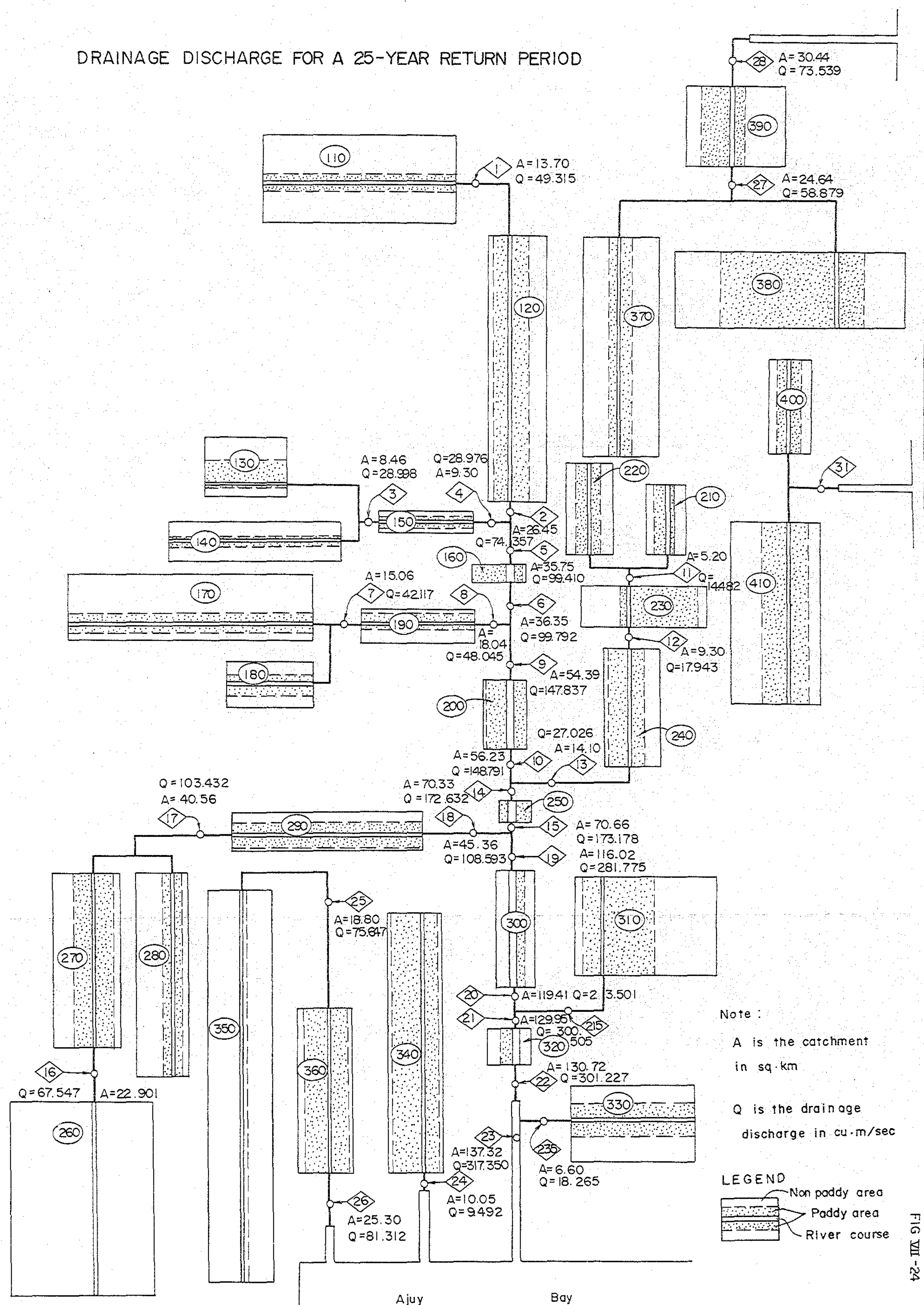
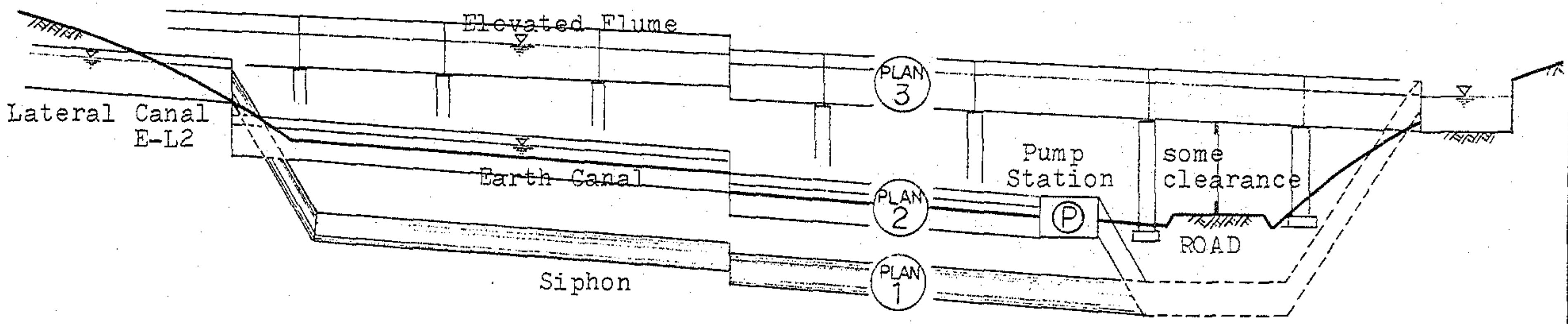


FIG VII-24

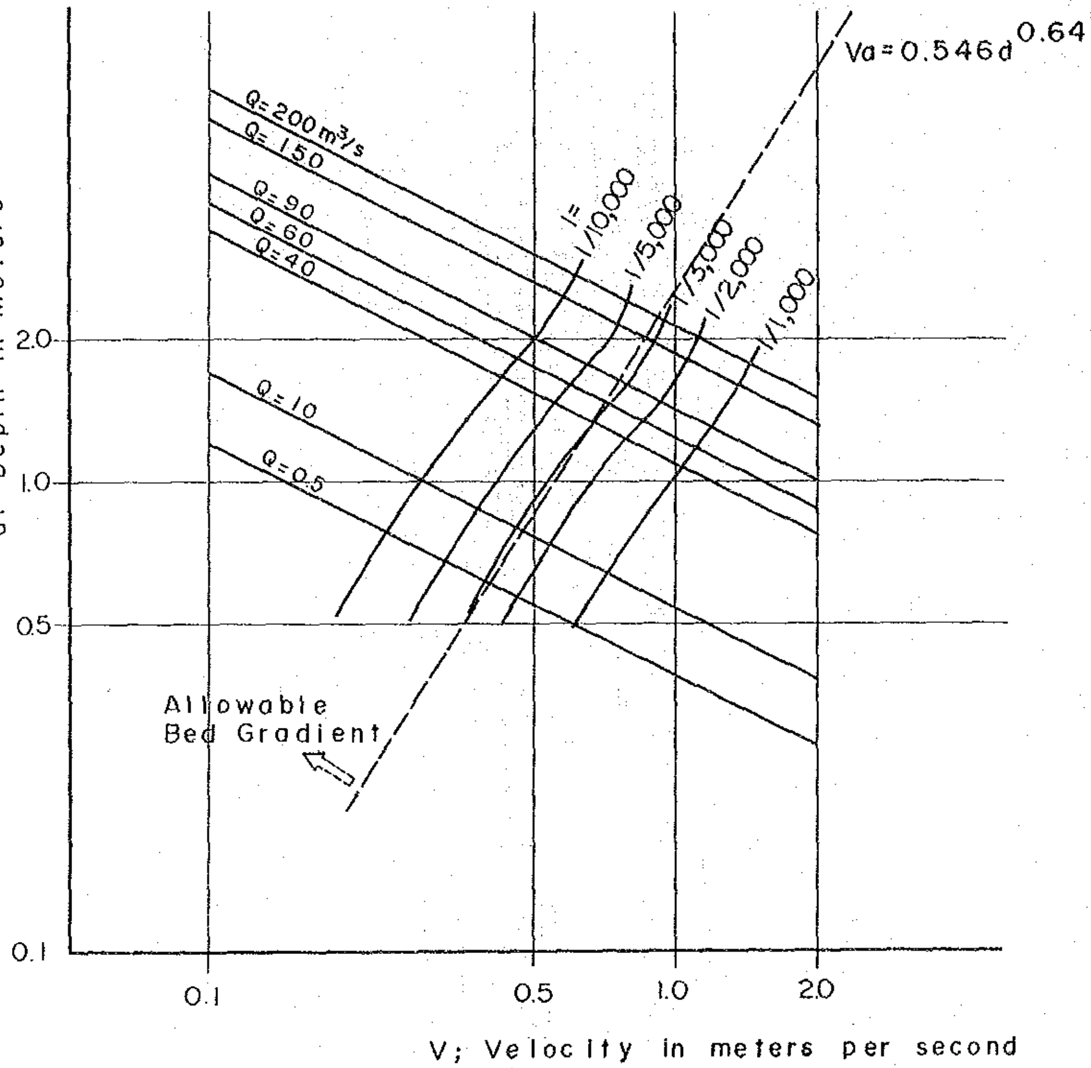


ALTERNATIVE PLAN FOR STRUCTURES TO IRRIGATE EASTERN AREA

FIG VII-25

FIG VII-26

CANAL BED SLOPE / FLOW VELOCITY RELATION



This relation has been analysed under conditions mentioned below;

1) Roughness Coefficient	0.025
2) B/d Ratio	
for canal with capacity of less than $4.0m^3/s$	2.0
for canal with capacity ranging from $4.01m^3/s$ to $9.0m^3/s$.	2.5
for canal with capacity greater than $9.01m^3/s$.	3.0
3) Inside slope	1:1.5

FIG. VII -27

TYPICAL LAYOUT OF ON-FARM FACILITIES

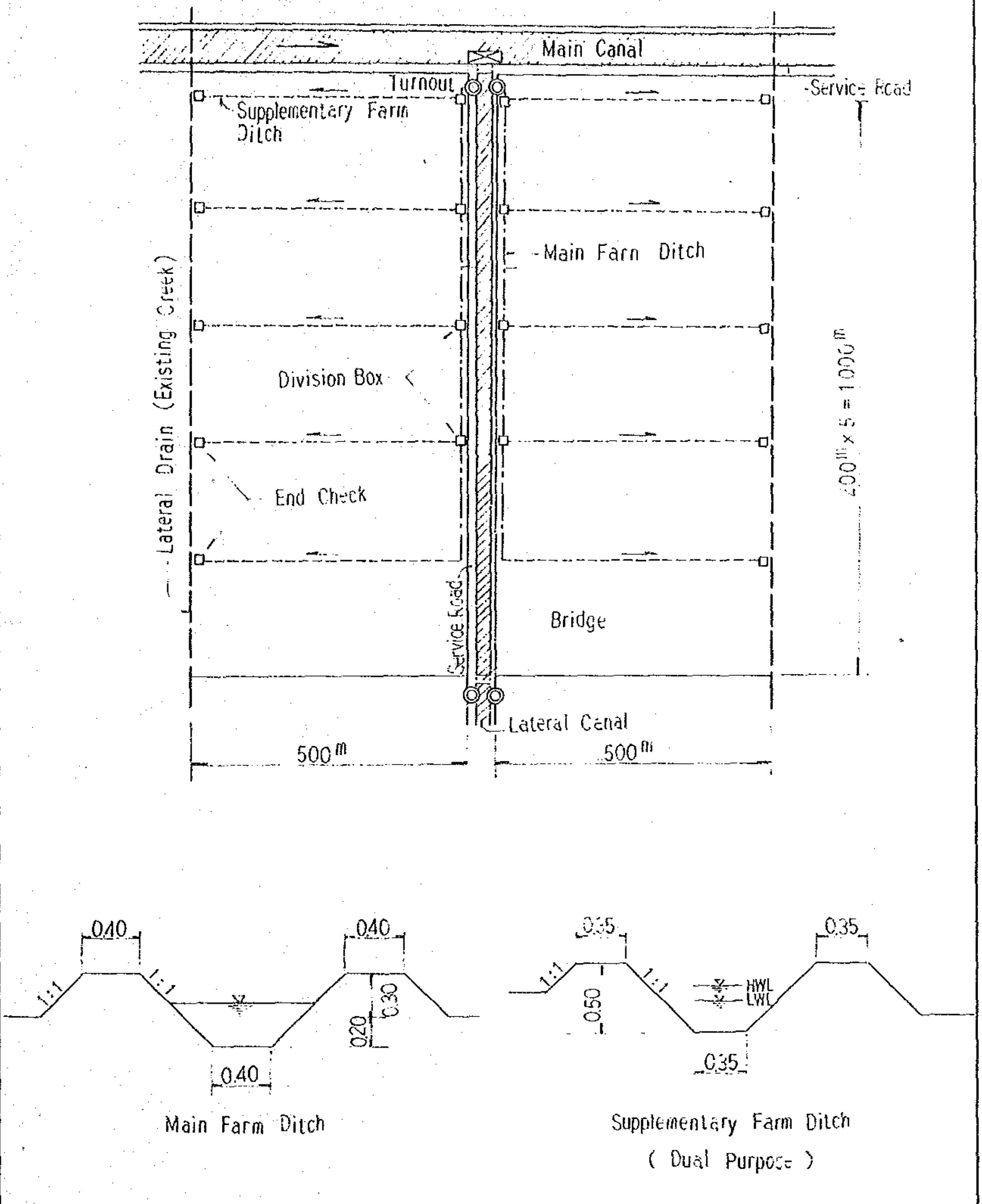
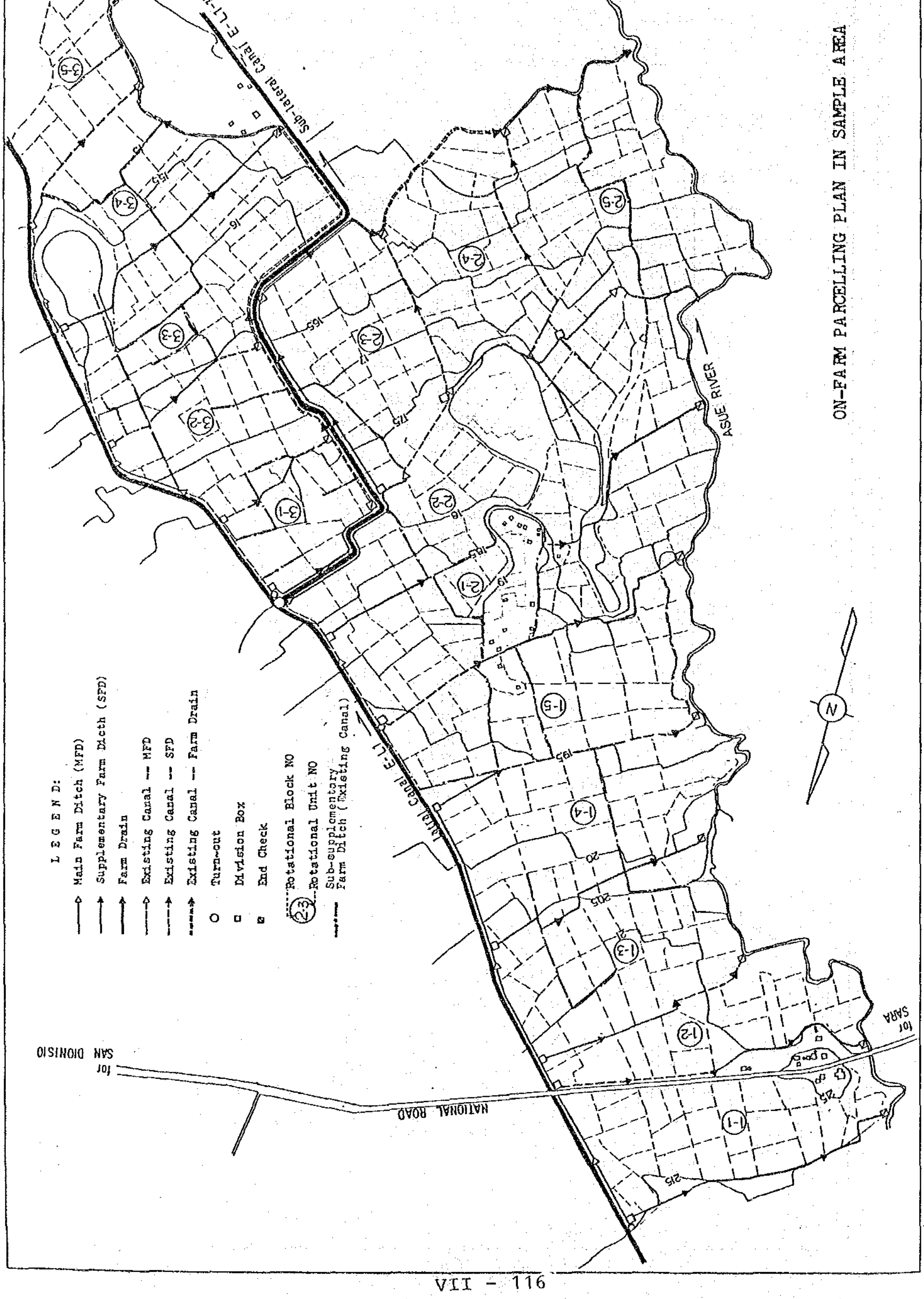
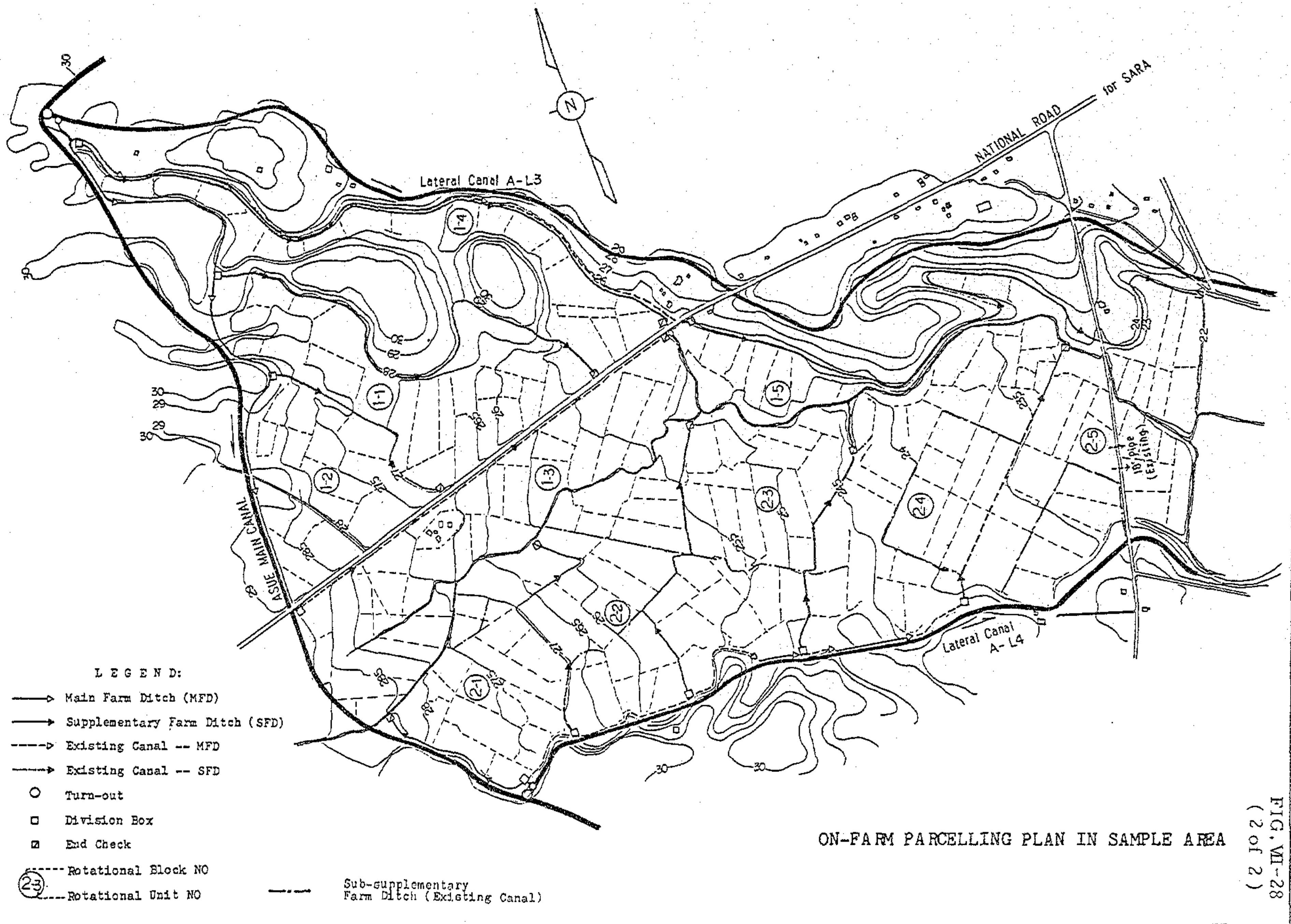


FIG. VII-28
(1 of 2)



ON-FARM PARCELLING PLAN IN SAMPLE AREA

FIG. VII-28
(2 of 2)



ON-FARM PARCELLING PLAN IN SAMPLE AREA

APPENDIX VIII

DAM AND TRANS-DIVERSION

APPENDIX VIII

DAM AND TRANS-DIVERSION

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LIST OF FIGURES

APPENDIX VIII

DAM AND TRANS-DIVERSION

1. DAM ENGINEERING

1.1 Selection of Dam Site

In order to supplement irrigation water shortages in the benefit area of the Asue River basin, a trans-basin scheme is planned which will divert water from the Catipayan River to the Asue Basin by construction of a dam on the former. Under the proposed scheme, a maximum flow of $6m^3/s$ will be supplied to the Asue River basin by trans-diversion from a reservoir on the Catipayan River. An effective reservoir capacity of 21.5 million m^3 was established as the appropriate dam scale as described in APPENDIX VI WATER RESOURCES DEVELOPMENT.

The two basic considerations governing selection of the dam site are as follows:

a) The main objective of dam construction is to provide sufficient irrigation water to the Asue River Basin via a trans-diversion canal and tunnel. The length of the latter is comparatively long, though the lower site needs a relatively short canal.

b) The Catipayan River is joined by the Catang River about 6km upstream from the boundary between the flat and mountainous regions in the vicinity of Barangay Aldemil. Consequently, in order to secure required water supply, selection of a dam site downstream from the above confluence is preferable.

In consideration of the above conditions, three alternative dam sites namely site A, B and C from downstream were selected between the confluence point and Barangay Aldemil on the basis of topographical maps and field investigations (FIG. VIII-1). Comparative study including comparison of construction costs was conducted for all three sites. The required reservoir capacity was estimated from water requirement as follows:

- Gross Storage Capacity	28.2MCM
- Effective Storage Capacity	21.5MCM
- Design Sediment Volume	6.7MCM

The dam features for each of the alternative sites are presented briefly in the following table.

MAIN FEATURES OF THE DAM

Item	Site A	Site B	Site C
Catchment Area	50.8km^2	48.0km^2	44.2km^2
Normal High Water Level	E1.104.0m	EL.101.5m	EL.124.0m
Dam Crest Elevation	EL.109.5m	EL.117.0m	EL.129.5m
Dam Height from Riverbed	47.5m	52.0m	46.5m
Dam Volume	2,050,000m^3	1,420,000m^3	796,000m^3
Trans-diversion Canal Length	4.0km	5.5km	7.8km
Access Road Extension	2.0km	3.5km	6.0km
By-pass Tunnel Length	500m	550m	400m

The reservoir capacity and area curve for each site is presented in FIG. VIII-2 to VIII-4. Study results showed that, as site A is located farther downstream than sites B or C, the location of the same is particularly advantageous for a shorter trans-diversion canal and access road length.

River width at the said site however, is about 80m and dam volume of the same is the largest of the three sites. Geologically, weathering on both abutments is advanced in comparison with other sites while topographically, the site is disadvantaged by a low and narrow mountain ridge on the abutment.

On the basis of the above conditions, site A was judged to be unsuitable for construction of a 50m class high dam. The same was accordingly eliminated from detailed study, and a detailed comparative study of sites B and C was carried out from the viewpoint of topographical and geological conditions, as well as estimated construction cost.

Results of the comparative study are presented in TABLE VIII-1. As the said table shows, construction cost for site C is ₱74.8 million less than for site B. Although from the viewpoint of construction plan, site C is inferior to site B, the overall advantages of site C are greater than those for site B. Accordingly, site C was selected as the Catipayan dam site under the present Project.

1.2 Dam Scale and Type

1.2.1 Dam and Reservoir Scale

The required effective storage capacity is estimated at 21.5 MCM. The scale of the dam is accordingly as follows:

- Catchment Area	: $44.2km^2$
- Gross Storage Capacity	: 28.2 MCM
- Effective Storage Capacity	: 21.5 MCM
- Dead Storage Volume	: 6.7 MCM
- Normal High Water Level	: EL.124.0m
- Design Flood Level	: EL.127.0m
- Freeboard	: 2.5m
- Crest Elevation	: EL.129.5m
- Dam Height	: 48.5m (47.5m from riverbed)

Due to insufficient data on sediment at the proposed dam site, design sediment volume was determined on the basis of actual measurements and data from other dams in the Philippines as follows:

$$1,500m^3/km^2/year \times 44.2km^2 \times 100 \text{ years} = 6,630,000m^3 : 6,700,000m^3$$

1.2.2 Dam Type

As a result of study of data obtained from geological and field survey, a zone-type (center core) rock fill dam is considered the most technically suitable and economic dam type for the proposed dam site for the following reasons.

a) The topography of the dam axis has a comparatively large configuration factor of 5 and thus a fill-type dam is considered more suitable than a concrete type.

b) Sufficient gravel required for concrete aggregate is not available within the site vicinity. Rock and earth fill materials required for a fill type dam however, are abundant near the site.

c) From the abundance of rock, semi-pervious and impervious materials and the quality of the same, and in consideration of the comparatively large scale of the proposed dam, a center core type rock fill dam, which is least subject to settling, is considered most appropriate.

d) As hydropower facilities are planned directly downstream from the proposed dam, frequent changes in water level are expected. Accordingly, in consideration of the effect of water level variations on the dam, construction of a thick permeable zone on the outer bank will be required.

1.3 Foundation and Embankment Materials

1.3.1 Dam Foundation

The bedrock of the site is composed of andesite and basalt pyroclastic rocks and outcroppings of the same occur in the riverbed. Although pyroclastic rocks are divided into various rock types, for the purpose of dam construction the same may be considered as one uniform rock foundation. Although some weathered zones and small fractures occur in the upstream portion, the same are not expected to hinder dam construction in view of the overall soundness and quality of the bedrock.

Foundation treatment will consist of grout, with curtain grout applied to the center core and possibly consolidation grout on some portions above the dam crest depending on results of further survey. It is also necessary to allow for rim grout in the lower ridge section.

1.3.2 Embankment Materials

Based on survey of various conditions at the proposed dam site, a rock fill dam was selected and a sufficient amount of materials to fulfill construction requirements is available near the site. Based on geological survey, core materials can be obtained from the talus in the upper layer and weathered portion of the bedrock, fill materials from extreme to slightly weathered portions of the bedrock, and transition and rock materials from the slightly weathered portions and fresh portions.

The design constants of the various materials are estimated as follows based on results of experimental tests. Details are discussed in APPENDIX III under GEOLOGY AND EMBANKMENT MATERIALS.

ESTIMATED DESIGN CONSTANT FOR EMBANKMENT MATERIALS

Type of Material	Specific Gravity	Moisture Content (%)	Density -Dry -Wet (t/m^3)	Porosity	Unit Weight -Saturated -Submerged (t/m^3)	Cohesion (t/m^2)	Internal Friction Angle (o)
Core	2.61	24.0	1.60 1.98	0.63	1.99 0.99	0.0	32.0
Filter/ Semi-pervious	2.61	2.6	1.90 1.95	0.37	2.17 1.17	(Filter) 0.0 (S.P.)1/ 0.0	(Filter) 37.0 (S.P.) 39.0
Rock	2.65	2.0	1.90 1.94	0.39	2.19 1.19	0.0	42.0

1/ Semi-pervious

1.4 Preliminary Design of Dam and Appurtenant Structures

The preliminary layout and design are discussed hereunder, while general layout, and typical cross-section and profile are presented in FIG. VIII-5 and VIII-6.

1.4.1 Design Seismic Coefficient

Seismic acceleration at the dam site was calculated by S. Okamoto's formula on the basis of PAGASA earthquake records from 1915 to 1980 shown in TABLE VIII-2 & FIG. VIII-7. The design of a dam is usually made on the basis of a 100-year return period earthquake. According to Okamoto's Formula, maximum acceleration at the proposed dam site is 196 gal., and the value for a 100-year return period is approximately 300 gal.

SEISMIC ACCELERATION AT DAM SITE

Return Period (Year)	Maximum Acceleration (gal)
2	8.3
5	23.0
10	46.6
20	87.3
30	122.4
50	182.0
100	299.8
200	474.7
500	830.0
1,000	1,230.4

A common method for estimating design seismic coefficient is to multiply the maximum acceleration value by the reduction rate, usually 0.5 to 0.6. In the case of the proposed dam site design seismic coefficient is:

$$K = 300 \times (0.5 \text{ to } 0.6)/980 = 0.15 \text{ to } 0.18$$

The design seimic coefficient of the embankment is thus determined at K = 0.18.

Using the coefficient, embankment slope was determined to satisfy a safety factor of 1.2 against surface slide. Furthermore, even in a case where observed maximum K = 196/980 = 0.2, a safety factor of 1.1 against surface slide must be satisfied.

1.4.2 Slope of Embankment

Slopes of the dam were determined at 1:3.0 for upstream and 1:2.1 for downstream as shown below using the above design seismic coefficient and material characteristics by application of surface slide and stability by the sliced slip circle method.

(1) Surface Slide Analysis

As for cohesionless materials, the slip circle method is characterized by a decrease in the safety factor with increase in the shallowness of the slip circle. Therefore the analysis for such cases is made by the surface plate sliding method as shown below:

For upstream slope $$F.S = \frac{(1-K\cdot\frac{\gamma sat}{\gamma sub}\cdot m)}{K\cdot\frac{\gamma sat}{\gamma sub} + m} \tan\phi$$

For downstream slope $$F.S = \frac{1 - K\cdot m}{K + m} \tan\phi$$

Where,
- F.S. : Safety Factor (=1.2)
- K : Seismic coeffeicient (=1.8)
- γsat : Saturated unit weight of rock material (=2.19 t/m^3)
- γsub : Submerged unit weight of rock material (=1.19 t/m^3)
- ϕ : Internal friction angle of rock material (=42^o)
- m : Gradient of slope

Assuming a safety factor and material characteristics as presented above, slopes for upstream and downstream were calculated at 1:3.0 and 1:2.0, respectively. (Although the downstream slope was determined at 1:2.1 on the basis of study by the sliced slip circle method below.)

(2) Slip Circle Method Analysis

Safety factor obtained by the slip circle method is derived by the following formula:

$$Fs = \frac{\Sigma\ (C'\ell + (N - Ne) \cdot \tan\phi')}{\Sigma\ (T + Te)}$$

Where,
- Fs : Safety Factor (=1.2)
- N : Normal effective force acting on sliced slip circle
- T : Tangental effective force acting on sliced slip circle
- Ne : Normal force by seismic load on sliced slip circle
- Te : Tangental force by seismic load on sliced slip circle
- C',ϕ : Cohesion and internal friction angle of materials, respectively on sliced slip circle
- ℓ : Arc length of sliced slip circle

Stability analyses against sliding are performed for the following three cases: (i) full water level (Case A), (ii) intermediate water level (Case B), and (iii) rapid drawdown of water level (Case C).

Assuming upstream and downstream slopes of 1:3.0 and 1:2.1, respectively, the results of analysis are presented in FIG. VIII-8 and summarized in the following table.

RESULT OF SLIP CIRCLE METHOD ANALYSIS

Case	Reservoir Condition	Water Level	Seismic Coefficient	Slope	Safety Factor
A	Full water level	EL.124.0m	0.18	upstream	1.204
				downstream	1.255
B	Intermediate water level	EL.115.0m	0.18	upstream	1.204
C	Rapid drawdown	drawdown from	0.18	upstream	1.204 1/
		EL.124.0m to	0.09	upstream	1.695 1/
		EL.109.0m	0.18	upstream	1.204 2/
			0.09	upstream	1.663 2/

Note: 1/ Not considering pore pressure in transition zone
2/ Considering pore pressure in transition zone

1.4.3 Dam Crest Elevation and Width

Freeboard should be added to the High Water Level to prevent wave over topping during design flood. The elevation of the dam crest was determined by the method presented below. As the dam has a spillway without gates, the surcharge water level was not considered.

$$EL \geqq Hf + hw + he + 1 \quad \text{or} \quad Hf + 3$$

$$\geqq Hh + hw + 1 \quad \text{or} \quad Hh + 2$$

Where, EL: Dam crest elevation

Hf: Normal highwater level

Hh: Design floodwater level

hw: Wave height due to wind

he: " " " earthquake

(1) Determination of hw (Wave Height due to Wind)

The wave uprush was determined by the SMB and Saville methods. These methods are based on maximum distance from the dam axis to the shore at Full Water Level, roughness of the dam slope, slope incline, and maximum wind during an average 10 minute period.

hw (smooth slope) = 1.5m

hw (riprap slope) = 0.6m

When; Fetch of reservoir = 1.5km

Wind velocity= 30m/s

Gradient of slope = 1:3.0

The upstream slope of the dam is planned as riprap construction. However, some fine gravel occurs between the larger rocks and accordingly, the average value of wave uprush for smooth and riprap slopes on the diagram were determined on the safe side at 1.0m.

(2) Determination of Wave Height Due to Earthquake

Wave height due to earthquake was determined by the method below.

$$ {}_2he = \frac{K\tau}{2\pi} \cdot (g\ Ho)^{\frac{1}{2}} $$

Where; K: design horizontal seismic coeffecient (0.18)

: earthquake duration (1.0 sec)

Ho: water depth at normal high water level (124.0 - 85 = 40m)

g: acceleration of gravity (9.8m/sec)

he: 0.6m

The larger of the following dam crest elevations obtained by surge height calculation was selected.

Normal High Water Level $\geqq$ 124.0 + 1.0 + 0.6 + 1.0 or

124.0 + 3.0

$\fallingdotseq$ 127.0m

Design Flood Level $\geqq$ 127.0 + 1.0 + 1.0 or 127.0 + 2

$\fallingdotseq$ 129.0m

Based on the above, non-overflow dam height (crest elevation) was determined to be El 129.0m and, with the addition of a crest protection layer, a dam crest elevation of 129.5m was adopted.

Crest width is usually determined on the basis of safety with regards to wind wave, infiltration, earthquake, crest utilization objective and construction conditions. Generally, a dam with a height of 50m or more has a 10-15m crest width. In this study, a 10m crest width has been adopted referring to similar projects in the Philippines.

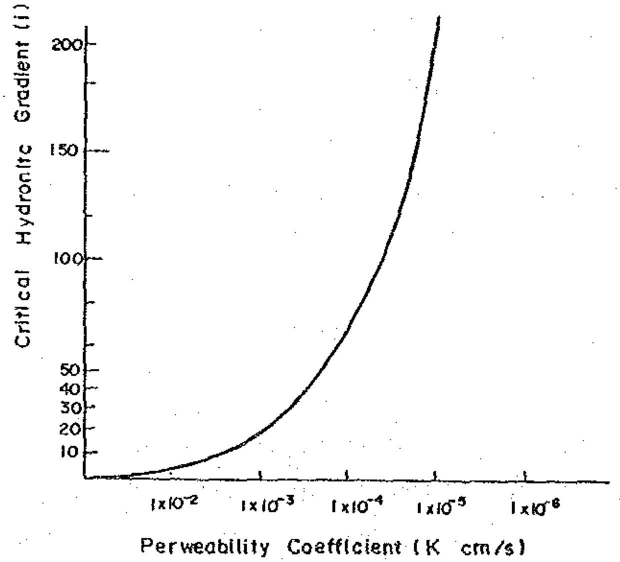
1.4.4 Zoning of Embankment

The center core type, which has superior earthquake resistance, was adopted for the proposed dam. Impervious zone width was determined considering construction conditions, available material volume, earthquake resistance and permeability.

As the bedrock of site C is composed of fresh rock, deep excavation will not be required in the cutoff trench. Accordingly, rock excavation volume will not be a prominent factor for the determination of bottom width of the impervious zone. As the stockpile yard is limited around the proposed dam site, less impervious volume is advantageous. In consideration of the above factors, stability of dam slopes and safety against seepage, a slope of 1:0.25 was adopted for the impervious zone.

In general, an impervious zone width of 30-50% of the water depth is considered safe even if construction conditions are quite poor. In the case of the proposed dam, core width/water depth = 28m/(EL.127m-EL.81m) = 60% which is on the safe side for design.

Subsequently, hydraulic gradient was also checked as follows:



- critical hydraulic gradient (ic)

  ic = 200 when K = 1 x 10^{-5}cm/sec

- results for other dams show hydraulic gradients of 1 - 4.

Hydraulic gradient (i) for the proposed dam was determined as follows:

$$i = \frac{H}{L} = \frac{\text{Used Water Depth}}{\text{Core Width}} = \frac{\text{EL.127m-EL.81m}}{\text{28m}} = 1.64$$

The same is considered sufficiently safe in view of the general value of 1 to 4. Critical velocity was calculated by the Justin Method with regards to granular diameter of soil, and checked.

$$\text{Critical Velocity } Vc = \sqrt{\frac{\gamma_s \cdot g}{A \cdot w}}$$

γs : Specific gravity of soil granules: (Gs - 1.0) x V

A : Flow area

V : 3/4

Gs : Specific gravity of core materials (Gs = 2.61)

Granule Diameter	Critical Velocity Vc (cm/sec)
5	17.2
1	7.7
0.1	2.4
0.01	0.77
0.001	0.24

The hyraulic gradient (i) of the impervious zone foundation is:

$$i = \frac{H}{L} = 1.64$$

As the Coefficient of Permeability is:

k = 1 x 10^{-5}cm/sec (design value)

then velocity is:

$$v = i \cdot k$$

$= 1.64 \times (1 \times 10^{-5})$

$= 1.64 \times 10^{-5} m^3/sec < Vc$

This is sufficiently less than critical velocity and is therefore considered safe. However, after completion of dam construction, measurement of seepage volume as well as sediment content will be required.

1.4.5 Service Spillway

(1) Location and type

The spillway is vitally influenced by the condition of the site. The right bank of the proposed dam site is elevated above the spillway crest and has a steep slope of about 30°; the contour line is parallel to the river course. There are creeks on the upstream and downstream portion of the right bank, and one creek in the latter location has a catchment of about $1.2km^2$.

The left bank mountain slope has a comparatively gentle slope, on the other hand, of about 15° at higher elevations than the spillway crest. Geologically, the weathered zone thickness at both abutments is thin at about 5m, presenting no problem. In the case of the right bank, a side channel spillway is appropriate, while on the left bank, if the spillway width is less than 100m, either a side channel or overflow type spillway can be adopted.

Topography provides an advantage for the left bank spillway which will have a comparatively short cut slope of 50m and linear alignment is technically possible. An overflow type is more advantageous than the side channel type on the basis of the rough cost estimation presented below.

In the case of the side canal type, the weir portion must be installed upstream of the dam. Comparative analysis revealed that the transition length is about 20m longer for the side canal type and similarly wall height with the same is increased by 4m. Comparative calculations are presented below. Excavation cost is not considered because the same will be used for embankment material.

COST COMPARISON FOR SPILLWAY TYPE

	Unit Cost	Side Canal Type	Overflow Type	Remark
Concrete weir	₱2,105/m^3	3,205m^3 ₱6.8 million	2,000m^3 ₱4.2 million	
Invert	₱1,601/m^3	5,500m^3 ₱8.8 milion	8,700m^3 ₱13.9 million	
Wall	₱2,105/m^3	20,200m^3 ₱42.5 million	13,000m^3 ₱27.4 million	Reinforcing was not considered.
Total Cost		₱58.1 million	₱45.5 million	

On the basis of the above comparison an overflow type spillway on the left bank with linear alignment is proposed considering topographical, geological and economic advantages.

(2) Structure

No gate is provided for the spillway of the dam in view of the following.

a) The reservoir will be affected by a high volume of flood-discharge for a considerably long period every year, because the dam is small-scale as compared with its catchment area.

b) The operation of a gate under such conditions is very difficult, resulting in substantial operation and maintenance costs. In addition, improper operation of a gate will cause disaster in the downstream area.

(3) Comparative Study of Spillway Type

Two types of spillway, the gate and non-gate types, were first compared to determine the most appropriate spillway.

The possibility of accidents due to delayed or inappropriate operation and other factors is greater with the gate type. Moreover, operation of the gate type is more complicated. As there are no bridges within reasonable distance downstream of the dam, area residents must ford the river on foot. In consideration of the risk of a sudden increase in downstream discharge with the gate type spillway, the non-gate type was judged to be the most appropriate. With this type, downstream flow increases naturally corresponding to the increased water level in the dam reservoir.

Comparative study was subsequently carried out on non-gate type spillways as outlined below.

1) Alternative A: (Selected Plan) Overflow Chute Type

Main Features

Overflow depth	3.0m	
Crest length	76m	
Total spillway length	368m	
Inlet		110m
Weir portion		8m
Chute		200m
Settling basin		50m

Cost: ₱61.6 million (See TABLE XI-2)

2) Alternative B: Combined Dam (Where the center of the dam is constructed of concrete and functions as a spillway)

If dam height is the same as Alternative A the features are as follows:

Main Features

Concrete dam height	50m
Concrete dam length	76m + 4m = 80m
Slope of upstream face	n = 0.1
Slope of downstream face	m = 0.8
Concrete volume ($90,000m^3$ = $10,000m^3$(wing))	$100,000m^3$
Embankment volume for both sides	$350,000m^3$

Cost:

Concrete ($100,000m^3$ x ₱1,601.2/m^3)	₱160.1 million
Embankment ($350,000m^3$ x ₱51,137.8 x 10^3 ÷ $670,000m^3$)	₱26.7 million
Total Cost	₱186.3 million

The combined dam has several structural problems such as waterproofing of the joints, which must be overcome. In addition, construction cost is comparatively high.

1) Alternative C: Side Channel Spillway with Tunnel Spillway

Side Channel Spillway

Main Features

Length	76m
Depth	9m
Bottom width (downstream)	18m
Concrete volume (approximate)	5,000m^3
Excavation (earth)	20,000m^3
" (rock)	20,000m^3

Cost: P =17.1 million
(5,000m^3 x ₱2,900/m^3 + 20,000m^3 x ₱33.9/m^3 (indluding Steel bars) + 20,000m^3 x ₱96/m^3)

Tunnel Portion

Main Features

Length	350m
Required section (2Q/V = 2 x 850m^3/S/30m/s)	57m^2
Diameter	8.5m
Material thickness	50cm (approximate)
Concrete volume	2,400m^3
Excavation (rock)	22,300m^3
Timbering	D = 9.5m L = 350m
Center	D = 8.5m L = 350m

Cost: = ₱36.4 million
(2,400m^3 x ₱2,200/m^3 (including steel bars) + 22,300m^3 x ₱800/m^3 + 350m x ₱35,000/m x ₱1,000,000 (center-9Lm))

Settling basin ₱24.6 million

(same as for Alternative A at 40% of A's spillway construction cost)

Total Cost: ₱61.6 million
(₱17.1M + ₱36.4 + 24.6M = ₱78.1M Alternative A)

In the case of the side channel spillway with tunnel, the tunnel entrance may be blocked by floating debris during floods. Moreover, construction cost is higher than that for Alternative A. If the side channel spillway with tunnel is used, control gates must be installed at the entrance or inside the tunnel and auxiliary facilities such as tower, inclined shaft, gates, and an energy dissipator are required. These features not only increase overall construction cost but also the costs of operation and maintenance.

Based on the results of the above comparison, the overflow chute type (Alternative A) was selected as the safest and most economical spillway.

(4) Design Discharge

Design discharge has been determined on the basis of a 200-year return period flood considering a safety factor of 20%. As discussed in APPENDIX II METEOROLOGY AND HYDROLOGY, 200-year return period flood peak discharge was estimated at $703.7m^3/sec$, and accordingly spillway design discharge was estimated at $850m^3/sec$ considering the above safety factor.

This design discharge is also safe for a 1000-year return period peak discharge of $847m^3/sec$ as an extraordinary flood. Furthermore, the Team examined safety for probable flood by heavier rainfall in the vicinity such as in Leyte and Samar islands, and the results showed that even in such cases the dam is sufficiently safe after considering the storage effect of the reservoir.

Storage effect can be considered in the reservoir area, although it is very difficult to fix the authentic hydrograph for storage effect due to insufficient meteorological data. Accordingly, for the design of spillway, storage effect was not taken into account.

(5) Optimum Crest Length

Optimum crest length of the spillway was determined in consideration of economic feasibility based on spillway overflow depth and dam height. Comparison of construction costs according to changes in overflow depth is presented below.

Crest Length $L = \frac{Qd}{C \cdot H^{3/2}}$

Qd: Design discharge $850 m^3/sec$

C : Discharge coefficient

C = 2.15

H : Overflow depth

Overflow depth (m)	2.5	3.0	3.5
Crest Length (m)	100	76	61
Difference of Embankment Cost	0	₱2.07x10^6	₱4.17x10^6
Difference of Spillway Construction cost	₱5.16x10^6	₱0.16x10^6	0
Total Cost	₱5.16x10^6	₱2.23x10^6	₱4.17x10^6
Order	3	1	2

(6) Hydraulic Design

Although a hydralic model test must be carried out at the detailed design stage, basic approaches are presented below.

1) Approach

Overflow coefficient Cd

$$Cd = 2.20-0.0416\ (Hd/w)^{0.990}$$
$$= 2.169$$
$$= 2.15 \text{ (on the safe side)}$$

Overflow crest length L

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

then $$L = \frac{Qd}{CH^{3/2}} = \frac{850}{2.15x3^{3/2}} = 76m$$

Approach velocity: $$V = \frac{gLd^2 - (gLd^2)^2 - 2gdQd^2)^{\frac{1}{2}}}{Qd}$$

$$= \frac{9.8x75x7^2 - ((9.8x76x7^2)^2 - 2x9.8x7x850^2)^{\frac{1}{2}}}{850}$$

$$= 1.63m/s < 4.0m/s \text{ X O.K.}$$

2) Determination of the transition section

In order to obtain favorable hydraulic conditions within the approach and transition portions, transition section was selected to maintain sub-critical flow. The transition terminus was designated as the control section. Hydraulic conditions from the control point directly downstream from the spillway weir were studied according to water surface routing.

When transition width is small, the length of the transition portion is shortened and as a result, water surface conditions are disrupted. Accordingly, canal width for the transition chute portion were determined at 30m on the basis of geological and topographical conditions at the approach, transition and chute portions, and also the results hydraulic study.

3) Chute

On the basis of geological conditions, a chute slope of 1:3.0 was adopted.

Freeboard El. $Fb = 0.6 + 0.037 \cdot v \cdot d^{1/3}$

Vertical Wall Height $H = (d + Fb) \times 1/\cos\theta \, (\tan\theta = 1/3)$

Water depth during design flood was adopted for wall height determination.

4) Energy Dissipator

The effective hydraulic jump type energy dissipator was selected. Based on foundation and topographical conditions as well as on hydraulic study, a bed height of EL. 76m was adopted for the dissipator. Moreover, a reinforced hydraulic jump structure was selected to ensure the safety of downstream structures. A subdam dam will also be constructed to facilitate smooth discharge downstream. Wall height of the energy dissipator was determined so that no overtopping occurs during design flood.

Even if dam height is greater than that calculated, a stable hydraulic jump cannot be obtained. In addition, 1/100 years is considered sufficient for energy dissipator design discharge. Based on these conditions, energy dissipator objective discharge was estimated at $590m^3$/sec and the height of the subdam was determined at -4.5m.

1.4.6 Diversion Facilities

Tunnel type diversion facilities are proposed considering topographical conditions. The said facilities will be used as emergency outlet facilities from the reservoir after completion of the dam.

(1) Design Flood Discharge

The design flood discharge for diversion facilities varies case by case and is decided in conformity with the design criteria for the dam. In accordance with these criteria, a flood discharge of $360m^3$/sec with a 10-year return period was adopted for design discharge.

(2) Hydraulic Dimensions

The hydraulic calculations for diversion facilities were carried out to establish the relationship between the diversion discharge and the reservoir route via an upstream coffer-dam with a flood discharge into the reservoir of $360m^3$/sec. A rise in water surface level in the reservoir due to various diversion capacities can be roughly estimated from the following formula taking into account the reservoir route.

$$H = \frac{2}{3} \cdot \frac{Qp}{Qd} \cdot \frac{H}{\frac{1}{3} + \frac{(1 + \quad) AfH}{QdT}}$$

Where,
- H: rise in water surface level due to diversion discharge
- Qp: peak discharge ($360\ m^3$/sec)
- Qd: diversion capacity $Qd = 0.65 \cdot A \cdot (2gH)^{\frac{1}{2}}$
- H : head at diversion capacity $H = 1.5D$
- α : increase ratio of reservoir surface area with rise in water surface level

Af: reservoir surface area at time of head on diversion capacity discharge

T : duration of time discharge over the diversion capacity (T is 2 hours = 7,200sec)

A : sectional area of diversion tunnel $A = 3.317(\frac{D}{2})^2$

D : inside diameter of diversion tunnel

The results of calculation for various discharges are shown in the following table.

D (m)	H (m)	Diversion (m^3/sec)	Water level (El.m)
4.5	13.68	125.5	105.4
5.0	11.66	163.4	104.2
5.5	10.13	207.3	103.4
6.0	8.95	257.7	102.9

In selecting the diversion discharge required for determining cofferdam crest elevation, the following were taken into account:

a) The construction schedule of the dam is assumed at a short period of 4 or 5 years;

b) With a rise in the main dam embankment over the cofferdam, the diversion capacity of the diversion facilities is increased;

c) Debris in the form of plants and tree trunks, will flow into the diversion facilities;

d) Since diversion facilities will be utilized as outlet facilities after completion of the dam, it is desirable for these facilites to have a maximum velocity of 10 m/sec or less to prevent erosion during the diversion period; and,

e) In general, construction of the high cofferdam represents a flood risk in itself and, therefore cofferdam height should be decided at approximately 1/3 of the main dam height.

Taking into account the above mentioned factors, the diversion discharge and the inside diameter of the horse-shoe shaped concrete lined tunnel for the Catipayan dam was decided as follows:

Diversion capacity (m^3/sec)	Inside diameter (m)	Water level (EL.m)
207	5.5	103.4

(3) Crest Elevation of Cofferdam

The freeboard of the cofferdam can be obtained by the same estimation method as that for freeboard of the main dam. The result of calculation and adopted crest elevation of the cofferdam is shown below.

Fetch (m)	R[1/] (m)	Freeboard[2/] (m)	Water level (m)	Crest elevation (EL.m)
900	0.60	1.60	103.4	105.0

1.4.7 Intake Facilities

Intake facilities will be installed so as to utilize the temporary diversion tunnel. A drop inlet type intake with minimum intake level of 109m will be used and the intake will be connected with the temporary tunnel via the shaft. A concrete plug will be installed at the dam axis from which discharge will be conveyed via a steel pipe to a release pond (WL: 90.0m) directly below the dam at the powerhouse and canal terminus site.

For safe operation of the dam, an emergency discharge facility will be installed; however, the outlet pipe line is designed for subsequent use as an intake facility. The outlet pipe will extend from the intake pipe and a discharge slide value will be installed 10m before the junction of the two pipes which will release water into the open channel of the temporary diversion tunnel in emergencies. Central valve height is planned at 81.30m.

1/ wind speed is 30m/sec and upstream slope of fill type cofferdam is 1:3.0 with dumped riprap.

2/ freeboard can be obtained as follows; Fb = R + 1.0

Reservoir level at emergency discharge is FWL 124.0m - DWL 109.0m = 150m. Time required to decrease the water level is estimated at 2 weeks without consideration of inflow from the basin. Required outlet pipe diameter was determined by the following method.

$$Q = K \ (2gH)^{\frac{1}{2}}$$

$$= \frac{1}{\frac{fi}{Ai^2}} \ \frac{1}{2} \ (2gH)^{\frac{1}{2}}$$

Q : discharge (m^3/s)

H : head at low water level (m)

fi : coefficient of water head loss

Ai : flow area of outlet pipe line (m^2)

Based on roughly estimated losses, obtainable discharge by pipe diameter is presented with functions of head at low water level as follows:

When	D = 1.2m,	$Q = 0.498 \ (2gH)^{\frac{1}{2}}$
"	D = 1.4 ,	$Q = 0.727 \ (2gH)^{\frac{1}{2}}$
"	D = 1.6 ,	$Q = 0.998 \ (2gH)^{\frac{1}{2}}$

A maximum average allowable flow velocity of less than V=5.0m/s is desireable for the steel pipe. Accordingly, for a maximum water requirement of $Q=6.0m^3/s$ a pipe diameter greater than $D = 2 \ (Q/V/\ )^{\frac{1}{2}}$ =1.24m will be required. Furthermore, time required for emergency discharge is:

$$T = \Sigma \ T = \Sigma \frac{\Delta V}{K \cdot \ (2gH)^{\frac{1}{2}}}$$

H=Water level-81.30m (Height at center of discharge gate)

(Transdiversion canal water level at starting point)

Water level (m)	H (m)	V (m^3)	V (m^3)	Discharge (m^3/s) D=1.2m	D=1.4m	T days D=1.2m	D=1.4m
124	42.7	21,500,000		12.86	18.77		
120	38.7	13,730,000	7,770,000	12.07	17.63	6.6	4.5
115	33.7	6,270,000	7,460,000	11.02	16.09	6.7	4.6
109	27.7		6,270,000	9.60	14.03	6.3	4.3
				Total		19.6	13.4

The flow velocity in the case of Q=6m^3/s with a steel pipe diameter of 1.3m is 4.52m/s and in consideration of the relationship of the same to outlet pipe capacity, outlet pipe diameter was determined at D=1.3m.

2. TRANS-DIVERSION PLAN

2.1 Alignment

The proposed trans-diversion canal/tunnel will lead irrigation water from the proposed Catipayan dam to the Asue River basin. The capacity of the same is preliminarily determined at 6.0m^3/sec. The required length of the canal portion is approximately 7.8km. As the topography along the canal from the dam to the basin's divide has a climbing slope, and in order to keep a high elevetion for tunnel planning and hydropower planning, the canal route was selected along the mountain skirts and foothills.

The trans-diversion structure is mainly open canal though a tunnel will be planned to pass through the mountain dividing the two basins. Alignment of the route was made based on a 1/4,000 topographical map and site survey. The basic consideratins are as follows:

a) Alignment will be determined considering canal type and canal slope which will satisfy water level relation between the intake outlet and trans-diversion tunnel inlet.

b) Topographical conditions, operation and maintenance, economic and safety factors are also evaluated to determine alignment.

2.2 Canal Type

Three canal types i.e., earth, lined and concrete flume were compared for the trans-diverison canal. Typical section for each type has been illustrated in FIG. VIII-9. Mountain slope along the trans-diversion canal is 1:1.5 to 3.0, with an average slope of about 30° in cross-sectional direction of the canal presenting a disadvantage for a canal type with a long cut surface. The proposed canal route passes the mountain skirts and accordingly, the canal crosses numerous creeks, necessitating the use of many passing structures. Geologically the cut face has a tendency to erode easily also presenting a disadvantage. Construction cost scale has been compared and is provided in TABLE VIII-3.

The results of comparison show that the concrete lined canal requires the minimum construction cost. However, the concrete flume canal is proposed for the trans-diversion canal considering the following.

a) The topography along the canal is greatly varied due to the mountain slope and foothills;

b) The route requires many curved alignments;

c) The cut bank slope of 1:1.5 is similar to the mountain's slope, hence the cut face length is quite long in the cases of earth and concrete lined canals;

d) Earth and lined canals are subject to erosion, especially when located on mountain slopes while landslides will result in a high maintenance and repair cost; and,

e) Cracking may occur for concrete lined canals due to unequal subsidence of embanked abutment.

2.3 Tunnel and Release

The trans-diversion tunnel will be provided to pass through the mountain which divides the Catipayan and Asue basins. Required length of the tunnel is 475m.

At the outlet of the tunnel, a head tank will be installed for hydropower generation. In case required irrigation water is less than the maximum discharge for the canal route power station ($3.0m^3/sec$) water will be lead through the generator. Should required irrigation water exceed the same, on the other hand, water will be released through a release way from the head tank. Based on study results, installation of the release way is more economical than enlargement of the hydropower penstock.

2.4 Preliminary Design of Tunnel and Canal

The total center length of open flume canals is $L_1 = 7,120m$ and the siphon interval is $L_2 = 580m$ for a total of 7,700m. As the construction cost per meter of open canal and siphon type is ₱$7.5x10^3$/m and ₱$13.0x10^3$/m respectively, the required siphon interval is designated as more than twice the open canal extension and, under the present plan, siphons will be installed in only 2 locations.

(1) Siphon Section

Although maximum capacity is Qmax = $6.0m^3$/sec, average flow under normal conditions, is only $3.0m^3$/sec. On this basis, a double box culvert was adopted as the siphon section for optimum structral and economic feasibility and effective siphon maintenance. The siphon interval is designed to be larger than flow velocity of the open portion in order to prevent sedimentation.

$$1.5m/s \leqq \text{flow velocity } V = \frac{Q}{A} \leqq 1.8m/s \quad (Q = \frac{Qmax}{2} = 3.0m^3/s)$$

$$1.5m/s \leqq \frac{3.0}{A} \leqq 1.8m/s$$

$$1.67m^2 < A \leqq 2.0m^2$$

$$\therefore \quad 1.29m < b < 1.41m$$

Accordingly, the siphon section is 1.40m x 1.40m x double box culvert.

Head Loss

Friction Loss

No.1 Siphon Interval distance $L_1 = 320m$,

Slope lenght $L_1' = 340m$

$$ht_1 = \frac{2g \cdot n^2}{R^{4/3}} \cdot L \cdot \frac{V^2}{2g}$$

$$V = \frac{3.0}{1.4x1.4-0.2x0.2x2} \doteqdot 1.60m/sec$$

$$= \frac{2x9.8x0.015^2}{(\frac{1.88}{5.13})^{4/3}} \times 330 \times \frac{1.60^2}{19.6}$$

$$= 0.72m$$

No.2 Siphon interval distance L_2 = 270

$$ht_2 = \frac{2g \cdot n^2}{R^{4/3}} \cdot L \cdot \frac{V^2}{2g},$$

$$= 0.59m$$

Loss due other factors

Screen loss	fr	$= \beta \sin \theta \cdot (\frac{t}{b})^{4/3}$	= 0.237
Gate loss	fg		= 0.2
Turn loss	fb	= 0.1 x 2	= 0.2
Entrance loss	fi		= 0.5
Exit loss	fo		= 1.0
Total	Σf		= 2.137

$$\therefore h\ell = 2.137 \times \frac{V^2}{2g}$$

$$\therefore he_\ell = h\ell_2 = 2.137 \times \frac{1.60^2}{19.6} = 0.28m$$

Based on the above, head loss for siphon No.2 is $h_1 = hf_1 + h\ell$

$= 0.72+0.28$

$= 1.0m$

and head loss for siphon No.2 is $h_2 = hf_2 + h\ell_2$

$= 0.59+0.28$

$= 0.87m$

(2) Open Flume Canal Section

The transfer water level up to the river crossing tunnel and directly downstream from the Catipayan dam is as calculated below.

$$\text{Open Canal Slope } I = \frac{EL.88 - EL.81.0 - (\text{Siphon loss})}{4{,}080m + 2{,}200m + 840m}$$

$$= \frac{7.0 - (1.0 + 0.87)}{7{,}120} = \frac{5.13}{7{,}120} = \frac{1}{1{,}400}$$

When B = 2.20m, depth of flow is h = 1.94m

$$V = \frac{1}{n} \cdot R^{2/3} \cdot I^{1/2}$$

$$= \frac{1}{0.015} \quad \frac{1.94x2.2}{1.94x2+2.2}^{2/3} \quad \frac{1}{1{,}400}$$

$$= 1.41m/sec$$

(3) Tunnel Section

The trans-diversion tunnel route is proposed through the mountain divide between the two basins. Cross-section of the tunnel is the standard horse-shoe type with 2R = 2.15m. Maximum discharge will flow at a water depth of 2R x 0.9($\doteqdot$ 1.94m). Longitudinal gradient of the tunnel is 1/950 and tunnel length is proposed at 472m.

COMPARISON OF DAM SITES

	Site B		Site C		Note
1. Topography and Geology					
- Riverbed width	- 40m		- 20m		
- Mountain slope	- 20° widening to downstream width/height ratio 6.3/1		- 20° to 30°, a little steeper than Site B width/height ratio 5.0/1		
- Topography	- surrounding mountain is massive and stable		- mountain height and width is generally small		
	- left bank has deep creek upstream of dam, mountain ridge at this point is narrow		- right bank has creek in the downstream area but the same will not effect dam planning		
	- possible dam axis is limited to short distance because of bank formation		- right and left banks form parallel topography		
- Geology	- many outrops of andestic tuff are observed in the riverbed		- difference between Site B and C observed; weathered zone seems thinner than at site B		
	- both abutments have some weathered zones, though the same are generally stable with no faults				
2. Embankment material	- Sufficient quantity of embankment materials are available in the vicinity of the dam		- the same as site B		
3. Others					
- Access Road	- already constructed, though widening required for 4km		- in addition to widening, a 4km length road must be newly constructed		
- Trans-diversion canal	- Total length of 5.5km is required		- total length of 8.6km is required, (3.1km longer than at Site B)		
- Land compensation	- 2 households		- none		
- Construction Conditions	- the downstream bottom provides a good stockyard		- river width is small, mountain slope is steeper, so construction condition is worse than at Site B		
- Hydropower	- Dam site hydropower is difficult because of elevation in relation to the trans-diversion canal		- dam site hydropower is possible		
4. Cost					
- Embankment	- $1.42 \times 10^6 m^3$	₱149.1×10^6	- $0.80 \times 10^6 m^3$	₱83.6×10^6	unit cost: ₱105/m^3
- Spillway	- 300m	₱111.6×10^6	- 250m	₱93.0×10^6	Unit cost: ₱372,000/m
- By-pass tunnel	- 550m	₱34.1×10^6	- 400m	₱24.8×10^6	unit cost ₱62,000/m
- Trans-diversion canal	- 5.5km	₱38.5×10^6	- 7.8km	₱54.6×10^6	unit cost: ₱7,000/m (3.5 x 2.0 flume)
- Access road	- 3.5km	₱3.5×10^6	- 6.0km	₱6.0×10^6	unit cost ₱1,000/m (effective width 8m)
- Total		₱336.8×10^6		₱262.0×10^6	

SEISMOLOGICAL DATA FOR THE PHILIPPINES
BY PAG - ASA
1915 - 1980

SEISMOLOGICAL DATA
Catipayan Dam
Sara, Iloilo

Dam Axis Coordinates; Latitude 11.33°, Longitude 122.99°

Ranking	Date	Epicenter		Magnitude	Distance	Acceleration
		Latitude N	LangitudeE	(M)	(km)	(gal)
8	Mar. 12. 1915	12,000	124,000	7.00	133.3	55.9
2	Apr. 27. 1919	11,000	123,000	6.40	36.8	151.9
24	May. 5. 1925	9,500	123,100	6.75	204.2	12.7
19	May. 25. 1925	12,500	122,500	6.25	140.9	16.7
28	Nov. 13. 1925	13,000	125,000	7.30	287.9	10.8
17	Jun. 15. 1928	12,500	121,500	7.00	208.6	18.4
12	Jun. 15. 1928	11,500	121,500	6.75	164.0	25.5
15	Jul. 12. 1931	12,400	123,800	6.50	148.5	21.6
6	Jul. 12. 1931	12,400	123,000	6.50	74.4	82.3
35	May. 24. 1935	12,000	125,000	6.75	232.0	7.8
26	Feb. 4. 1941	9,500	124,000	6.90	231.8	11.8
9	Nov. 5. 1941	12,500	123,000	6.90	130.3	51.9
21	Jun. 7. 1947	11,500	125,000	6.90	220.5	13.7
3	Jun. 24. 1948	11,000	122,000	8.20	114.3	138.2
5	Jun. 24. 1948	10,500	122,000	8.20	142.3	104.9
18	Mar. 7. 1950	10,000	124,000	6.75	184.8	17.6
10	Mar. 7. 1950	10,500	122,250	6.75	122.8	49.0
4	Mar. 7. 1950	11,000	122,500	6.75	65.0	121.5
11	Jul. 8. 1951	11,000	122,000	6.50	114.3	40.2
29	Jul. 8. 1951	13,000	123,000	6.50	186.0	10.8
30	Jul. 2. 1954	13,000	124,000	6.75	216.3	10.8
27	Aug. 18. 1957	12,000	124,500	6.50	181.1	11.8

SEISMOLOGICAL DATA
Catipayan Dam
Sara, Iloilo

Dam Axis Coordinates; Latitude 11.33°, Longitude 122.99°

Ranking	Date	Epicenter		Magnitude	Distance	Accelerati-on
		Latitude N	LongitudeE	(M)	(km)	(gal)
38	Jul. 11. 1942	11.000	122.000	5.50	114.3	6.9
16	Jul. 11. 1942	11.900	122.100	6.10	116.2	21.6
22	Jul. 11. 1942	12.000	122.500	5.50	91.9	13.7
37	Jul. 13. 1942	10.000	122.500	6.00	157.5	6.9
36	Apr. 17. 1942	12.000	121.500	6.30	179.1	7.8
34	Jun. 23. 1964	11.370	122.460	4.60	58.1	8.8
33	Jun. 5. 1968	11.080	122.340	5.00	76.3	8.8
13	Jul. 25. 1971	12.300	123.700	6.40	133.0	24.5
31	Mar. 28. 1973	11.990	122.790	5.10	76.7	10.8
23	Apr. 5. 1973	11.937	122.499	5.40	86.3	13.7
20	Apr. 5. 1973	11.708	123.202	4.70	48.1	16.7
32	Apr. 5. 1973	11.700	122.800	4.30	46.2	9.8
1	Oct. 31. 1975	11.000	123.000	6.80	36.8	196.0
7	Dec. 22. 1975	11.000	123.000	5.40	36.8	42.7
39	Feb. 20. 1976	10.816	122.648	4.70	68.4	6.9
14	Feb. 5. 1980	11.300	123.080	3.50	10.4	23.5
25	Apr. 9. 1980	11.220	122.800	3.70	24.1	12.7

ACCELERATION (GAL in cm/des^2) = 10

$$= 10g\ 640 + \frac{D + 40}{100}(-7.604 + 1.7244M - 0.1036M^2)$$

D is distance (km) between damsite & epicentre
M is magnitude of earthquake

COEFFICIENT OF SEISMIC FORCE (K) = GAL/G

G = 980cm/sec^2

COST COMPARISON FOR TRANS-DIVERSION CANAL TYPE

(Open Canal Length L = 7,120m)

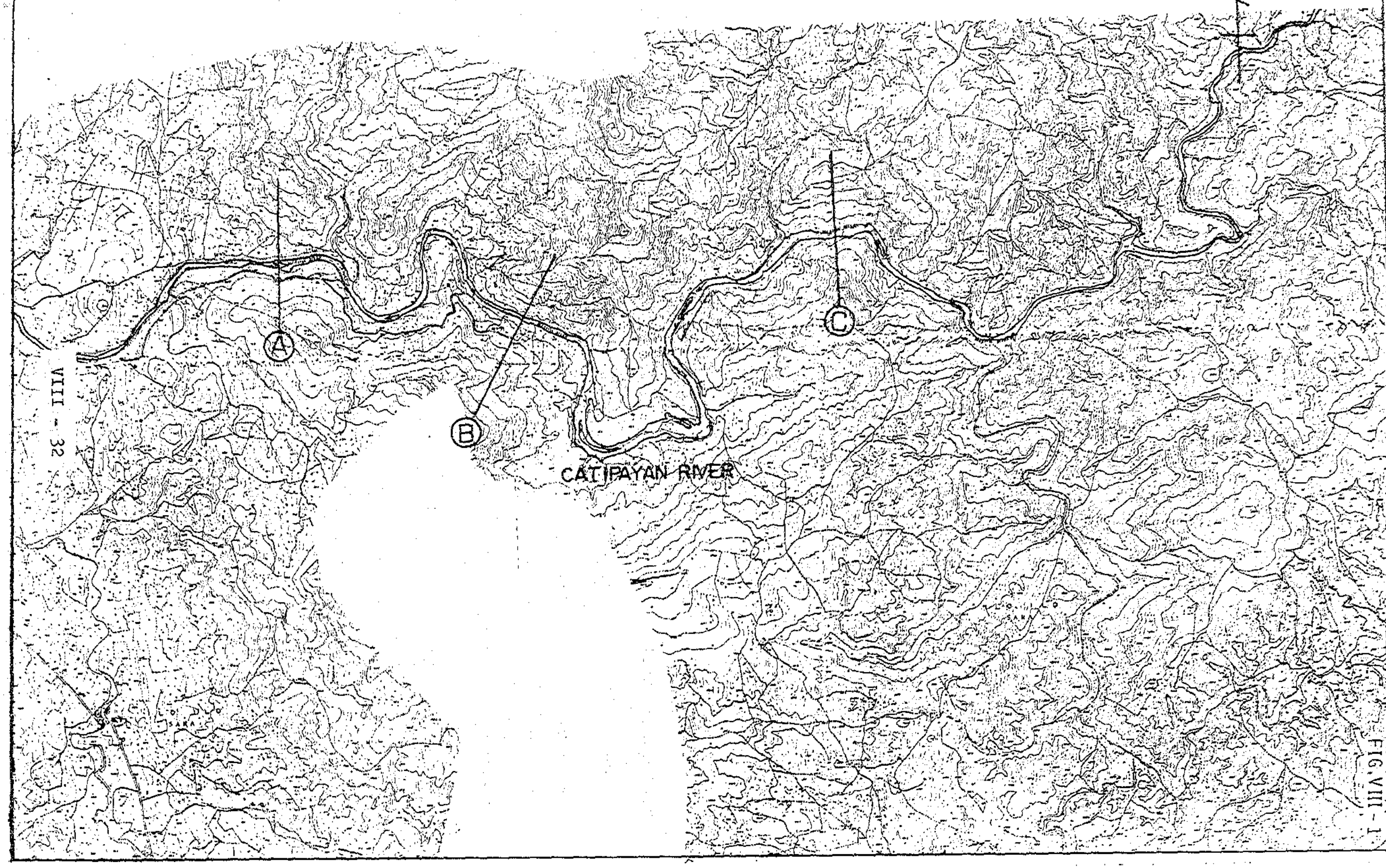
Unit : P

Cost / Canal Type	Item Unit Cost	Earth		Concrete Lined		Concrete Flume	
1) Excavation	$40/m^3$	107	4,280	46	1,840	26	1,040
2) Compacted fill	$24/m^3$	23	552	22	528	26	624
3) Loading & Hauling	$42.33/m^3$	84	3,556	24	1,016	0	-
4) Scataring	$10.5/m^2$	55	578	37	389	25	263
5) Concrete Lining (Class B)	$1,601.19/m^3/m$	-	-	0.8	1,281	-	-
6) Concrete Flume (Class A)	$2,978/m^3/m$	-	-	-	-	2.0	5,956
7) Right of way & Damages	$0.6/m^2$	67	40	41	25	29	17
Sub-Total			9,006		5,079		7,900
Direct Construction Cost			$64.1x10^6$		$36.2x10^6$		$56.2x10^6$
Maintenance Cost	$3.1/m^2$		$1.5x10^6$	41	$0.9x10^6$	29	$0.6x10^6$
Bridge	Pieces	5	$800x10^3$	5	$450x10^3$	5	$245x10^6$
Overchute	Pieces	36	$3,500x10^3$	36	$2,500x10^3$	36	$1,080x10^3$
Total Cost			$699.9x10^6$		$44.6x10^6$		$58.1x10^6$

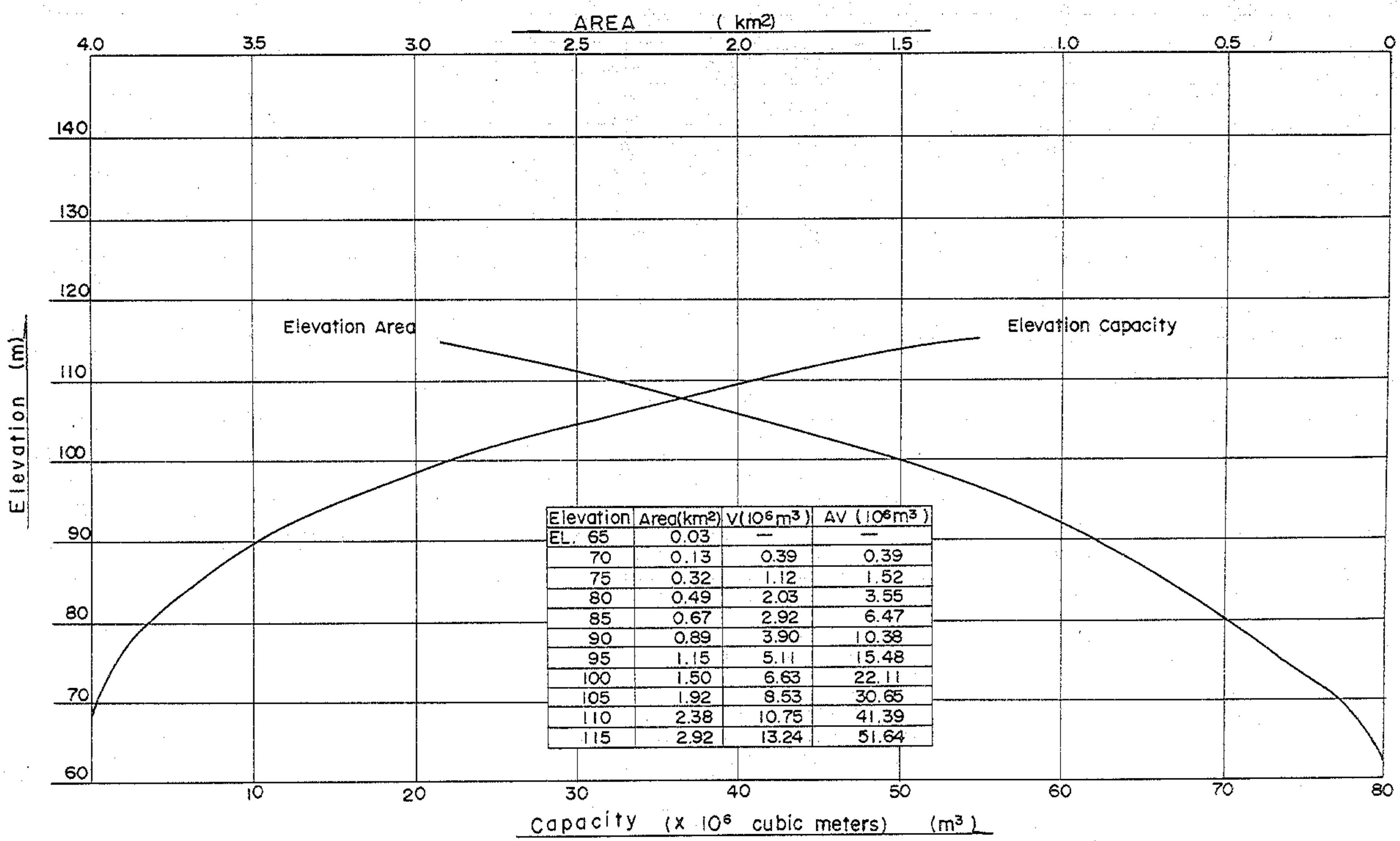
FIG. VIII - 1

LOCATION OF DAM SITE CANDIDATES

S = 1/20000



ELEVATION-CAPACITY, AREA RELATION OF DAM SITE A

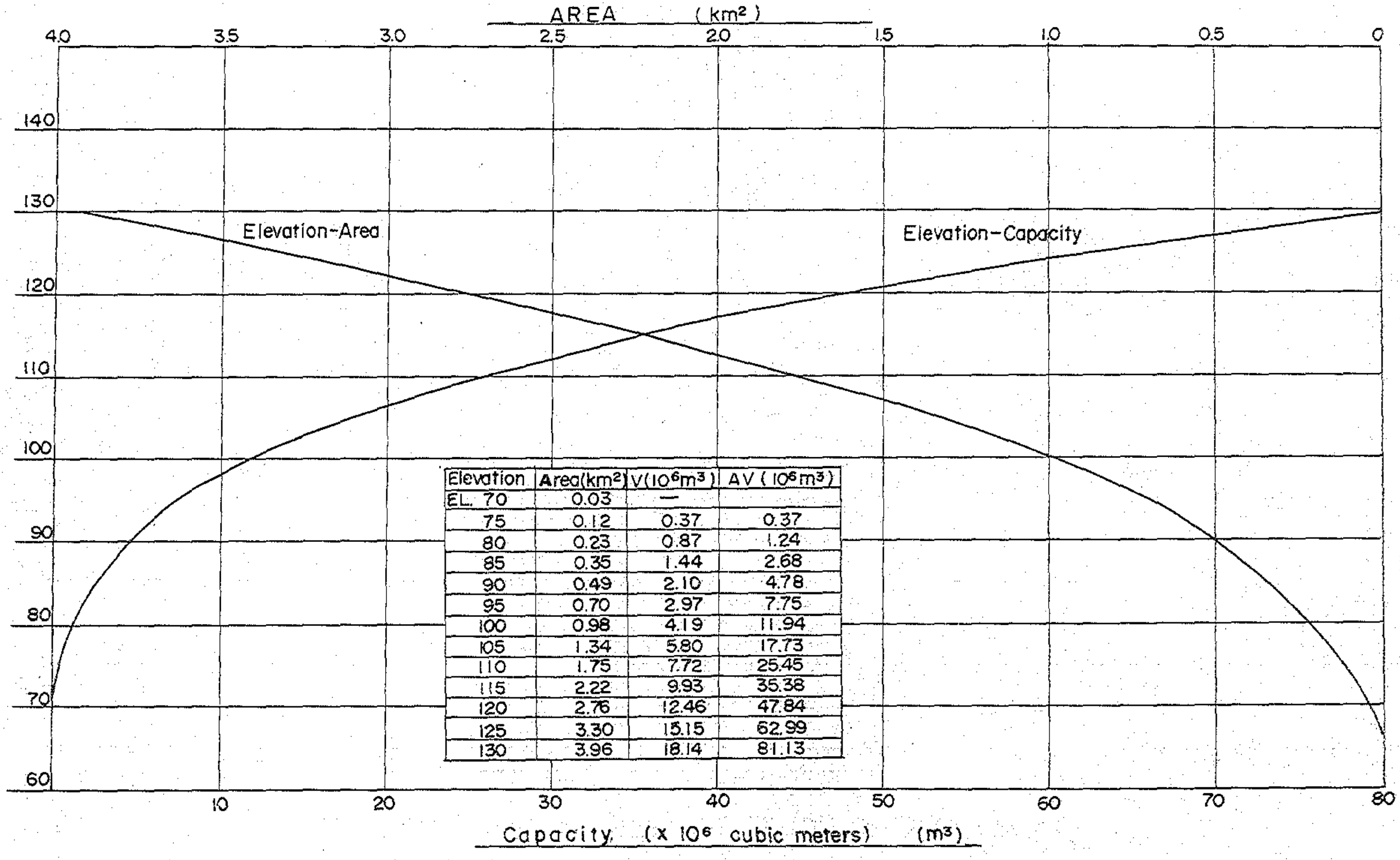


Elevation	Area(km2)	V(10^6m^3)	AV (10^6m^3)
EL. 65	0.03	—	—
70	0.13	0.39	0.39
75	0.32	1.12	1.52
80	0.49	2.03	3.55
85	0.67	2.92	6.47
90	0.89	3.90	10.38
95	1.15	5.11	15.48
100	1.50	6.63	22.11
105	1.92	8.53	30.65
110	2.38	10.75	41.39
115	2.92	13.24	51.64

FIG. VIII - 2

FIG. VIII-3

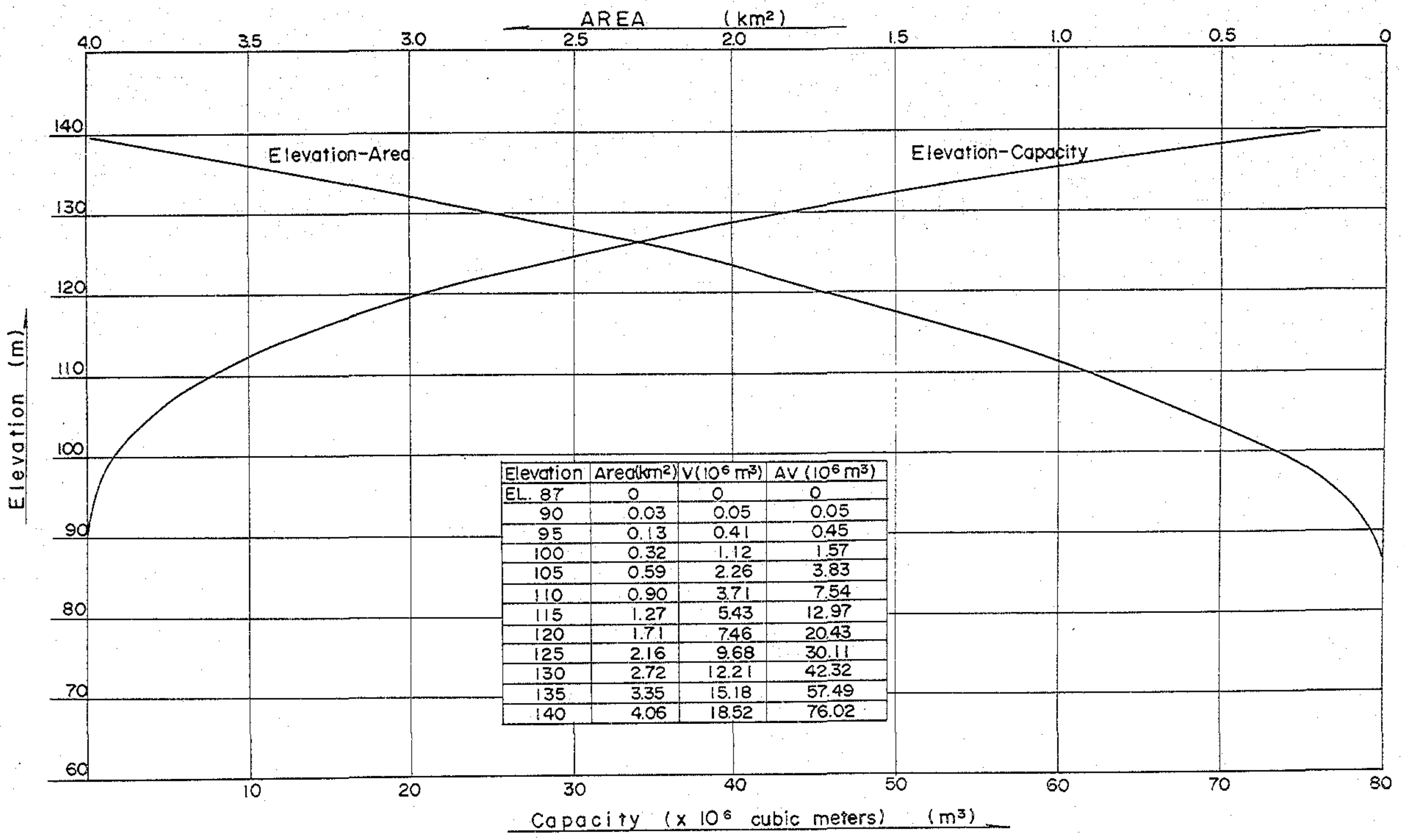
ELEVATION-CAPACITY, AREA RELATION OF DAM SITE B



Elevation	Area(km²)	V(10⁶m³)	AV (10⁶m³)
EL. 70	0.03	—	
75	0.12	0.37	0.37
80	0.23	0.87	1.24
85	0.35	1.44	2.68
90	0.49	2.10	4.78
95	0.70	2.97	7.75
100	0.98	4.19	11.94
105	1.34	5.80	17.73
110	1.75	7.72	25.45
115	2.22	9.93	35.38
120	2.76	12.46	47.84
125	3.30	15.15	62.99
130	3.96	18.14	81.13

FIG. VIII-4

ELEVATION-CAPACITY, AREA RELATION OF DAM SITE C



Elevation	Area(km²)	V(10⁶ m³)	AV (10⁶ m³)
EL. 87	0	0	0
90	0.03	0.05	0.05
95	0.13	0.41	0.45
100	0.32	1.12	1.57
105	0.59	2.26	3.83
110	0.90	3.71	7.54
115	1.27	5.43	12.97
120	1.71	7.46	20.43
125	2.16	9.68	30.11
130	2.72	12.21	42.32
135	3.35	15.18	57.49
140	4.06	18.52	76.02