The elevation of the river bed is E. L 9.5 m at the point where crosses National Route 32, and EL. 3.9 m at the confluence with Rio Matina. Its average gradient is 1/600.

The upper reach of the river is wide and shallow, while the river becomes narrower and deeper as it goes downstream, with the width varying in a range between 70 and 315 m.

F.3.2 Latest Flooding

The heavy rainfalls that swept through the areas on the Atlantic side in the late January, 1988, caused overflows at various rivers including rio Matina, Rio barbilla and Rio Chirripo. The inundated areas confirmed through interviews at the time of flooding are shown below;

. Left side of Rio Matina	- Meandering section south of Matina
	- Most downstream near Cuatro Millas
. Right side of Rio Matina	- Stretch between Estrada and banana plantations in Agrodisa
. Rio Barbilla	- Stretch between meandering section south of Davao and the confluence with Rio Matina
. Rio Chirripo	- Stretch between the point approx. 1.5 km downstream National Route 32 and the confluence with Rio Matina

However, the flood did not affect the La Luisa banana plantation on the left side of Rio Matina and the Agrodisa banana plantation on the right side of Rio Matina because of the exsisting bank.

F.4 Drainage Improvement Plan

F.4.1 Scope of Drainage Improvement

Drainage improvement is to be made for the area of category II - IV. However, the following areas, which also fall in this category, will be excluded from the improvement plan.

. The farm land with an elevation less than 2 meters, draining of which by gravity method is not feasible.

. The areas not included in the land-utilization plan for farm land development, such as the virgin forest, banana plantations, etc.

The breakdown of the improvement areas is shown below.

The Improvement Area (ha)

	II	III	IV	Total
Poor Drainage Area	1,960	3,480	6,230	11,670
Farmland under Elevation El.2 m	- 410			- 410
Exception Area	- 260	- 390	- 360	- 1,010
Total (Improvement Area)	1,920	3,090	5,870	10,250

F.4.2 Planning for Principal Canals

F.4.2.1 Arrangement of Principal Canals

1) Object and Classification

Principal and other small secondary drainage canals will be arranged in such a way as to collect excessive rain water and discharge it from the Area.

The principal and the secondary drainage canals are classified by the area of basins as shown below.

. Principal Drainage Canal : the basin area is more than 10 km^2
. Secondary Drainage Canal : the basin area is less than 10 km^2

2) Arrangement

(1) Principal Canal No.1

This canal, which is to be located at the very eastern end of the Study Area will have as its origin the principal canal in the banana plantation between Matina and Bataan.

The existing canal will be modified and linked to the lake Laguna Chorejal in the low-lying virgin forest. Since the bottom elevation of the lake is estimated at EL.-2.00 m, enough gradient for the planned canal can be maintained even at its lower reach.

(2) Principal Canal No.2

The existing canal flowing into Laguna Chorejal through Bataan and Santa Marta will be utilized. The canal between Bataan and Luzon, which runs adjacent to the banana plantation, has adequate drainage capacity and is well maintained.

(3) Principal Canal No.3

The existing canal, which runs along the border between the virgin forests north of Goschen and the farm land, will be utilized.

Taking into consideration the gradient directions and the arrangement of secondary canals, its middle course will be arranged so as to run parallel to Principal Canal No. 2.

(4) Principal Canal No. 4

The existing canal, canal Sara which runs through Sara and Goschen and flows into Madre de Dios, will be utilized.

(5) Principal Canal No.5

This canal, which is to be constructed at Madre de Dios in as much a straight line as possible, will have its upper section at the *CATIE and its lower section in the lower reach of Madre de Dios with adequate gradient for drainage and less meandering. The following reasons are offered as a justification to rehabilitate and utilize the river for the principal canal.

. Madre de Dios meanders radically.
. Madre de Dios is serving as the main drainage canal for the farm land in the surrounding area.
. Construction of river banks is not desirable, because it would substantially alter the farming method in the surrounding area.

*The upper section of the CATIE is a slope. The land on both sides of Madre de Dios in the area is mainly used as general and secondary forests, and any short-time overflows of the river are expected to have a very little effect on the farming.

(6) Principal Canal No.6

The canal to be constructed will replace Rio Veintiseis which now serves as a drain canal. It will mainly accept water drained from the mountainous areas and will be linked to Principal Canal No 5.

Because of the inadequate cross section of the point where the railroad crosses the river, the river's upper section is occasionally inundated when there is a heavy rainfall, then, enlargement work at the point is required. The cross sections located at the upstream of crossing point of the railroad will be used without enlargement work.

(7) Principal Canal No. 7 and No. 8

The two rivers, which located the south of National route 32 and have their watersheds outside the Study Area, run across the road and flow through the Study Area before joining Rio Barbilla. The section between the road-crossing point and rio Barbilla will be rehabilitated as the new principal canals No. 7 and No. 8.

F.4.2.2 Drainage Capacity

1) Formula

The flood protection facility to be constructed is designed to protect the Areas from floods originating in Rio Matina, Rio Chirripo and Rio Barbilla. The design discharge capacity of the principal canals is calculated using the formula shown below.

$$Q = 1/3.6 \times I \times f \times A$$

where

Q: Design Discharge (m^3/s)

I: Rainfall Intensity at concentration time of Flood (mm/hr)

f: Runoff Coefficient = 0.45 (Master Plan Report Annex F F.5.3.1)

A: Watershed (km^2)

2) Flood Concentration Time

Flood concentration time "t" is obtained using the following formula.

$t = 0.0083 \cdot L/(H/L)^{0.6}$

where

t: Concentration Time of Flood (hr)
H: Difference of Elevation of River Course (m)
L: Length of River Course (m)

Table F.4.1 shows the calculation result, as shown in the table, there are no noticeable differences in the flood concentration time. The following table summarizes the concentration time to be used in the planning.

Proposed Arrrival Time (Hour)

Drainage Canal	Downstream	Upstream
Principal Canal No. 1 to 5	12 hr	5 hr
Principal Canal No. 6 to 8	2 hr	-

3) Rainfalls

The Rainfalls with a return period of five year at the CATIE is to be used as the standard for the planning (Master Plan Report Annex F.5.3).

Rainfall intensity to be used is obtained by the formula shown below.

$I = 229/(t+4.0)$

where

I: Average Rainfall Intensity in t hour (mm/hr)
t = Concentration Time of Flood (hr)

Rainfall intensity classified by the flood concentration time is shown below.

$I = 229/(t+4.0)$

where

I : Average Rainfall Intensity in t hour (mm/hr)

t = Concentration Time of flood (hr)

Rainfall intensity classified by the flood concentration time is shown below.

Downstream in Principal Canal No.1 to 5: t = 12 hr

$I = 229/(12 + 4) = 14.3$ mm/hr

Upstream in Principal Canal No. 1 to 5 : t = 5 hr

$I = 229/(5 + 4) = 25.4$ mm/hr

Downstream in Principal Canal No.6 to 8: t = 5 hr

$I = 229/(2 + 4) = 38.2$ mm/hr

4) Design Yield Discharge

The design yield discharge to be classified by the flood concentration time has been calculated as shown below.

12 hr

$q = 1/3.6 \times I \times F \times A = 1.784\ m^3/s/km^2$

5 hr

$q = 1/3.6 \times I \times F \times A = 3.175\ m^3/s/km^2$

2 hr

$q = 1/3.6 \times I \times F \times A = 4.775\ m^3/s/km^2$

where

I: 14.3 mm/hr, 25.4 mm/hr, 38.2 mm/hr

f: 0.45

A: 1.0 km^2

Design yield discharge to be used for the design of each principal canal is shown below.

Design Yield Discharge ($m^3/s/km^2$)

Canal	Downstream Location	Downstream Design Discharge	Upstream Location	Upstream Design Discharge
Principal Canal No.1	No.0~No.6+0.80	1.787	No.6 +0.80~EP	3.175
Principal Canal No.2	No.0~No.6+0.50	1.787	No.6 +0.50~EP	3.175
Principal Canal No.3	No.0~No.6+0.95	1.787	No.6 +0.95~EP	3.175
Principal Canal No.4	No.0~No.6+0.55	1.787	No.6 +0.55~EP	3.175
Principal Canal No.5	No.0~No.6+0.15	1.787	No.6 +0.15~EP	3.175
Principal Canal No.6	No.0~EP	4.775		
Principal Canal No.7	No.0~EP	4.775		
Principal Canal No.8	No.0~EP	4.775		

The design discharge for the plan is obtained by multiplying design yield discharge shown above by the drainage area. Table F.4.2 shows the result of the calculation and Fig. F.4.2 shows the drainage system plan, Fig. F.4.3 shows the drainage network model plan.

F.4.2.3 Structure and Dimensions

1) Structure

Taking into account the following points, the side slope of the planned principal canals is to be set at 1:1.5.

- The soil property in the Study Area is sandy clay.
- Many existing canals in the Study Area have the side slope of 1:1.0, and some of them are crumbling.
- With the side slope of 1:15, maintenance would be easier.

With the following points taken into account, depth of the new canals are scheduled to be in a range between 3.0 and 5.0 m according to the design drainage discharge.

. Depth of the canals inside the banana plantations
. Depth of the existing canals to be incorporated in the new canal system.
. Depth of the new secondary canals.

The planned cross section of the new principal canals are shown in the volume IV (Drawing). A 4.0 m wide service road will be constructed along the canals to facilitate maintenance service. Levels of the road surface are to be designed 0.5 m higher than the calculated water levels of canals, because they temporarily rise above the ground levels in low-lying areas when there is a flood. (See Fig. F.4.4)

2) Dimensions

Manning's formula shown below is used to determine the dimensions of the principal canals in relation to the design discharge.

$V = 1/.I^{1/2}R^{2/3}$

$Q = A \times V$

where

V : Avarage Flow Velocity (m/s)

n : Coeficient of Roughness = 0.03

I : Gradient of Canal

R : Hydraulic Radius (m)

A : Sectional-area (m^2)

Q : Discharge)m^3/s)

Gradient of the principal canals has been determined in such a way that the maximum average flow velocity will not exceed 1.5 m/2. For this reason, it is necessary in some places to plan drop structures in the canals (Refer to Volume VII, Principal Canal Profile Drawing).

Canal Profile Drawing).

Overflows of water at the time of a flood affect lower reaches of the principal canals from No.1 to No. 5, and the flow of canal become a non-uniform flow. Therefore, calculations to determine cross sections were made with this factor being taken into consideration. The following table shows the sections with non-uniform flows and other relevant information for each canal.

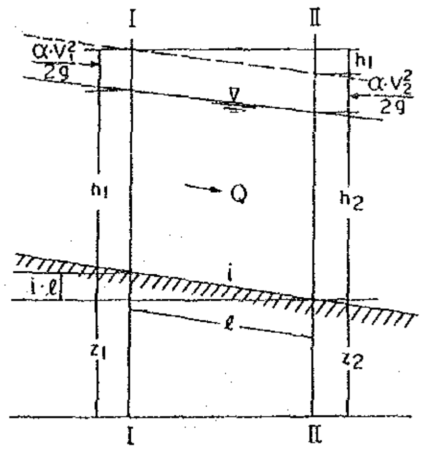
Section of Principal Drainage Canal with Non-uniform Flow

Canal	Location	Design Discharge (m^3/s)	Gradient of Canal
Principal Canal No.1	No.0~No.4+0.80	74.9~44.0	1/1,800~1/1,400
Principal Canal No.2	No.0~No.2+0.80	40.1	1/1,200
Principal Canal No.3	No.0~No.3+0.75	35.6~23.1	1/2,000
Principal Canal No.4	No.0~No.6+0.45	85.8~29.8	1/2,000
Principal Canal No.5	No.0~No.3	208.0~193.1	1/2,400~1/2,200

The calculation was made according to the formulas shown below with the lower-end water lever at E. L 1.40 m (lower inundation level at a time of flooding with a return period of 5 year; refer to F.4.4).

$$h + \frac{\alpha \cdot Q^2}{2g \cdot A_1^2} + Z_1 = h_2 \frac{\alpha \cdot Q^2}{2g \cdot A_2^2} + Z_2 + h_f$$

$$h_f \quad \frac{Q^2 \ell}{2} \left(\frac{n_1^2}{R_1^{4/3} A_1^2} + \frac{n_2^2}{R_2^{4/3} A_2^2} \right)$$



Z: Height from the standard level to the canal bottom level (m)

Q: Discharge (m^3/s)

I: Distance between the cross sections I and II (m)

A: Cross section of flow (m^2)
g: Gravity acceleration (m/S^2)
h: Water depth (m)
h': Water head loss at cross sections I and II (m)
R: Hydraulic mean depth (m)
n: Roughness coefficient (0.030)
α: Energy compensation coefficient

* Subscripts numbers 1 and 2 next to the letters respectively show the cross sections I and II.

Table F.4.3 shows the result of hydraulic calculations of the planned principal canals and Table 4.4 the result of irregular-flow hydraulic calculations. Table F.4.5 shows the planned canal extension.

F.4.2.4 Crossing Structure

A railway or a road crosses the principal canals No.1, No.2 and No.6. During the field study, measurement was made on the area of the cross section at these point. Drainage capacities of the canals were calculated on the basis of the measurement results.

The result shows that the rehabilitation of cross section is necessary for the railway-crossing point and road-crossing point of the principal canal No.6. Table F.4.6 summarizes the results of hydraulic calculations and the need of such modifications. Table F.4.7 shows details of the calculations.

F.4.3 Planning of Secondary Drainage Canal

F.4.3.1 Drainage Discharge

1) Formula

The same formula is used for calculation of design discharge the secondary canals as for the principal canals.

2) Flood Concentration Time

The flood concentration time of secondary canals No.(4-2) is used in the planning because of its typical topographic characteristics. The same Ruziha formula is used to calculate the flood concentration time.

The result of the calculation shows the concentration time to be 4 hours as shown below.

$H = 2.0$ m (difference of elevation of canal course)

$L = 3{,}700$ m (length including the tertiary canal)

$$t = 0.0083.L/(H/L)^{0.6}$$

$$= 280 \text{ min} = 4 \text{ hr}$$

3) Rainfall Intensity

Intensity of rainfalls (with a return period of 5-year) within the flood concentration time is calculated using the same formula as the one used for the principal canals.

The result shows the intensity within the concentration time to be 28.6 mm/hr as shown below.

$$I = 229/(t+4.0) = 28.6 \text{ mm/hr}$$

t : 4 hr

4) Drainage Capacity

The Rational formula is used. The calculation result indicates the volume of design discharge to be 3.575 $m^3/s/km^2$ as shown below. Table F.4.8 shows the drainage area and the drainage capacity of the secondary canals in the planning.

$$Q = 1/3.6 \times I \times f \times A = 3.575\ m^3/s/km^2$$

where

I: 28.6 mm/hr

f: 0.45

A: 1.0 km^2

F.4.3.2 Structure and Dimensions

Manning formula is used to calculate the dimensions of the secondary canals. (See Fig. F.4.5)

Table F.4.9 shows the result of calculation for cross-section of flow. The plan incudes many new secondary canals in the position of the existing canals and the Study was made whether such plan was feasible or not using the data on the canal cross sections obtained during the on-site survey.

The existing canals with adequate drainage capacity and enough gradient will be used without modifications. For the canals with smaller drainage capacity than the one required by the plan, enlargement work will be made on the cross section.

F.4.3.3 Drainage Capacity of Existing Main Crossing Structure

The major road and railway cross the planned secondary canals which utilize the existing canals. The study has been made to determine whether the area of cross sections of such crossing points has adequate drainage capacity required by the plan. The result of the study is shown below.

No.(2-6) Secondary Canals (Veinticuatro Millas)

Road Bridge .... to be rehabilitated

Railway Bridge .... to be used without rehabilitation

No.(10-1) Secondary Canals (near national Route 32)
Road Bridge .... to be used without rehabilitation

Table F.4.10 shows the result of hydraulic calculations.

F.4.4 Inundation Analysis

F.4.4.1 Drainage Analysis Method

The flood protection facilities are installed for Rio Matina, Rio Barbilla and Rio Chirripo, and the drainage facilities within the Area will be improved.

The forecasted flow phenomenon of large rivers and outflow phenomenon after completion of these facilities in the area to be developed are analyzed, and forecasted inundation of the low flatland is estimated.

1) Flow of Large River

In the same way as the analysis of present conditions, the flow phenomena are analyzed using the equations of motion and continuity shown in F.2.2.1. However, river water does not overflow to the Area.

2) Flow in the Drainage Canal

The flow is analyzed by the kinematic runoff method. The analysis is made by using the equations of motion and continuity shown in Master Plan Report Annex B B.2.1. In this analysis, the flow is analyzed assuming that principal canal No.2 is a model. The drainage discharge from the all principal drainage canals are estimated by using the drainage discharge obtained from the analysis of the canal No.2.

Figure F.4.6 shows the drainage analysis model of principal drainage canal No.2.

3) Inundation in the Low Flatland

Water flowed out from the drainage canal is storaged in the low flatland, and flows into the Caribbean Sea. This storaging volume is analyzed by using the equation of storage described in F.2.2.1.

F.4.4.2 Result of Analysis

Figure F.4.7 shows the results of inundation analysis in the low flatland for 1/5 probability flood. Maximum inundation water level in the low flatland is estimated to be EL. 1.27 m.

The backwater occurs due to inundation in the low flatland at downstream of the principal drainage canals during the time of flood. The dimension of the principal drainage canal is determined assuming that the downstream end water level is EL. 1.30 m (maximum water level EL. 1.27 m + allowance) taking the backwater into consideration.

F.5 Flood Protection Plan

F.5.1 Comparative Study of Flood Protection Facilities

F.5.1.1 Methods of Protection

The following points should be taken into consideration for the planning of flood protection facilities in the study area.

. Existing river's cross section is too small to drain out the design discharge, and it will be costly to enlarge the cross section by a digging method,

. Cost for land acquisition and/or compensation is cheep,

. Most of the designed drains are scarcely connected with existing rivers (existing drainage system is also scarcely drainage into existing rivers). With these conditions, flood protection using the embankment method causes no harmful effects to formulate the drainage plan, and

. Flood protection with the embankment method is employed in the Study Area.

With above mentioned, flood protection in the Study Area should be constructed by the embankment method though a digging method is also included in comparative study for selection of adequate countermeasures.

F.5.1.2 Basic Concept for Comparison

The existing rivers meander remarkably at the following sections.

. Rio Matina - Location of 3 km and 9.5 km upstream from the place where Rio Matina flow together with Canal del Tortuguero.

. Rio Barbilla - Locations of 4 km downstream from where National Route 32 crosses, and 15 km upstream from the confluence with Rio Matina.

The existing conditions of the meandering sections and their vicinity are as follows;

. Remarkable meandering makes worse hydraulic conditions and cause inundation at the time of a flood.

. The surrounding areas by meandering are used for cattle grazing or ill-managed old cacao plantations.

The flood protection by using embankment method without the modification of river course has following disadvantages;

. Construction of dike along the existing meandering courses is not desirable in terms of hydraulics and may cause problems on the safety of dike in the worst cases.

. When the dike is constructed at the both side on meandering courses, unavailable areas for agriculture is increase.

For these reasons, shortcuts will be made for the four sections with remarkable meandering courses before flood protection plans are formulated. The followings are considered to establish the flood protection plan.

CASE I : Major river bed makes to be wider, but banking height keeps low.

CASE II : Making the higher bank than CASE I, therefore, total river bed width comes narrow as compared with CASE I.

CASE III : Enlargement of existing river cross section especially of river bed portion by digging.

To construct the dimension of flood protection facilities on the above cases, the followings area taking into consideration;

. Destruction of banana plantations along Rio Matina should be kept minimum.

. The railway bridge crossing over Rio Matina should be kept free from the flood.

. Erosion is proceeding on the existing river shore. Distance distance between dike and river bed (width of major riverbed) is taken 20 m in minimum to ensure the safety of the dikes.

F.5.1.3 Rio Matina Comparative Plans

Three detailed comparative plans (CASE I - CASE III) for Rio Matina are set out as below:

CASE I : Banking height is 2.5 m in an average including free boad of 1.0 m.

CASE II : Banking height is 3.0 m in an average including free boad of 1.0 m.

CASE III : Digging river bed and flood discharge will be managed within a minor river bed.

Banking heights for CASE I and CASE II are set at 2.5 and 3.0 m respectively with the following reasons:

. With the banking height at 2.0 m, required width of major river bed is estimated at 1.0 km on one side. These are resulted that the most of the banana plantations located along Rio Matina situate in the major river bed and people lived in both side of river course required migration.

- With the banking height at 3.0 m, free boad at the railway bridge crossing over Rio Matina is estimated at 0.7 m (EL. 13.5 m - EL. 12.83 m = 0.67 m). When the banking height takes over 3.0 m, no safety margin is put on the railway bridge.

For CASE III, the following points should be considered in proceeding with the flood protection plans.

- River gradient should be designed so as to keep even surface.

- Designed river bed at the railway crossing over Rio Matina should be kept as existing height.

- Because of backwater by Canal del Tortuguero, water level in the most downstream reaches would be higher than the ground elevation of the farm land on both sides of the river. These phenomenon required dikes at the downstream reaches though the existing river will be maintained by digging.

F.5.1.4 Rio Barbilla and Rio Chirripo Comparative Plans

The followings are pointed out to construct designed hydraulic conditions on both Rio Barbilla and Rio Chirripo.

- Difference of designed flood water level is very small (EL. 13.66 - EL. 13.46 = 0.20 m) between CASE I and CASE II of Rio MAtina at the confluence with Rio Barbilla and Rio Chirripo (Most upstream reaches of Rio Matina).

- Lower reaches of both rivers are greatly affected by backwater from Rio Matina. With this reason, the differences of designed flood water level is small even if the width of major river bed makes to be wider.

. At the section which the effects of backwater of Rio Matina does not reach, designed flood discharge can be drained out with width of major river bed. Such width is set out in view of safety of dike.

In case that the flood protection plan is constructed by embankment method at Rio Barbilla and Rio Chirripo, only water level only Rio Matina of CASE II is to be considered foe flood protection plan. Comparison on flood protection plan is as follows:

CASE I : None (the same dimension as CASE II)

CASE II : Banking height is to be calculated on the basis of design flood water level of CASE II on Rio Matina.

CASE III : Enlargement of existing river section by digging and flood discharge is managed within minor river bed.

F.5.1.5 Hydraulic Calculations

Dimension of flood protection facilities is constructed by non-uniform flow computations with design flood discharge. Non-uniform flow computation is carried out by the procedure explained in F 4.2.3. Basic condition on calculation procedure is settled as follows:

Design flood discharge :

Refer to Main Report 3.3.2

River cross section :

Survey results executed in the field

Length of river :

Based on the topographic map of scale in 1/10,000

Roughness coefficient:

0.04, on the premise that the tree in the major river bed are grubbed and well-maintenance will be perfomed after completion of the project

Water levers at the most downstream reaches of Rio Matina

E.L. 2.0 meters, taking the safety margin on the project facilities into account, water level at Canal del Tortuguero is calculated using the non-uniform flow computation when flood with an occurrence interval of once in 5 years is occurred at the time of high-water level of ordinary spring tide on the Caribbean Sea.

The result of the calculations for CASE I to CASE III are tabulated below.

CASE I :	Rio Matina	Table F.5.1
CASE II :	Rio Matina	Table F.5.2 1)
	Rio Barbilla	Table F.5.2 2)
	Rio Chirripo	Table F.5.2 3)
CASE III:	Rio Matina	Table F.5.3 1)
	Rio Barbilla	Table F.5.3 2)
	Rio Chirripo	Table F.5.3 3)

F.5.1.6 Facility Dimensions by Comparative Cases

Based on the results of hydraulic calculation, dimensions on project facilities is shown in Fig. F.5.1 to Fig. F.5.3 of designed river profile. In addition to these, location of dike on CASE I and CASE II is shown in Fig. F.5.4. Summary of dimensions on project facilities are as follows.

Outline of Flood Protection Facilities in Case Study

CASE I

River	Expansion of Embankment Left Bank (km)	Right Bank (Km)	Total (Km)	Height of Embankment (m)	River Width (m)
Rio Matina	15.2	14.4	29.6	0.9 - 3.6	600 - 1,630
Rio Barbilla	9.5	9.7	19.2	0.5 - 3.6	120 - 160
Rio Chirripo	2.0	2.1	4.1	2.0 - 3.6	130
Total	26.7	26.2	52.9		

CASE II

River	Expansion of Embankment Left Bank (km)	Right Bank (Km)	Total (Km)	Height of Embankment (m)	River Width (m)
Rio Matina	16.4	16.4	32.8	1.0 - 4.0	300 - 640
Rio Barbilla	9.5	9.7	19.2	0.5 - 3.6	120 - 160
Rio Chirripo	2.0	2.1	4.1	2.0 - 3.6	130
Total	27.9	28.2	56.1		

CASE I

River	Training River Length (km)	Cross Section Depth (m)	Bottom Width (m)	Top Width (m)
Rio Matina	16.9	5.5 - 9.0	150 - 300	180 - 322
Rio Barbilla	9.5	3.5 - 6.5	80 - 90	94 - 116
Rio Chirripo	2.2	7.0	70	98
Total	28.6			

F.5.1.7 Construction Cost Estimation

The table below shows the construction cost for flood protection facilities on the basis of comparative studies. The cost includes construction cost for required facilities, cost for land acquisition and compensations. As the result of comparison, CASE II is calculated as the most economical case. With this reason, CASE II is taken as the best plan. The break-down of cost estimate shown in Table F.5.4.

Construction Cost for Flood Protection Facilities

Item Works	Construction Cost (₡1,000) CASE I	CASE II	CASE III
Embankment & Cleaning	272,401	285,351	54,359
Excavation	281,050	281,050	6,300,000
Land Acquisions	237,075	80,720	24,500
Resettlements	90,500	4,200	-
Total	881,026	651,321	3,378,859
(%)	135	100	979

F.5.2 Required Facilities and Dimensions

F.5.2.1 Structure and Dimensions

Soil materials for embankment is obtained from protected areas. Cross section of dike is designed at 3.0 m in top width and 1:1.5 in slope on both side. Freeboad on design flood discharge is ensured at 1.0 m. Designed height of dike is determined on the basis of design flood water level and ground elevation at the major river bed, and those heights of dike vary as follows:

Rio Matina : Bank Height (H) = 1.0 m - 4.0 m

Rio Barbilla : (H) = 0.5 m - 3.6 m

Rio Chirripo : (H) = 2.0 m - 3.6 m

F.5.2.2 Stability Analysis

Grading analysis and specific gravity test are executed at MAG with six test pit samples which obtained form river terrace on Rio Matina. Analyzed result is shown in Table F.5.5. From the Table F.5.5, No. 4 samples which obtained from left bank of downstream contains a lot of sandy clay loam, however, other soil samples are categorized silty of loam or loam composing 22%

of sand 52% of loam and 22% of clay in an average. To clarify the stability on dike, invariable values of soils are settled as below based on the result of soil analysis.

Wet unit volume weight	r = 1.8 t/m^3
Saturated unit weight	rs = 2.0 t/m^3
Cohesion	c = 2.0 t/m^3
Internal friction angle	$\phi = 30^0$

Stability analysis is carried out at following two cases with designed height of 4.0 m.

CASE I : At the time of flooding without earthquake
CASE II : At the time of earthquake without flooding

Following equation is employed for stability analysis and calculation result on CASE I and II is shown in Fig. F.5.5.

Based on the calculation result, factor of safety comes to more than 1.5 for each case. Therefore, proposed dike has enough stability on its structure.

$$FS = \frac{\Sigma\ (C.1 + (N-No)\ \tan\phi)}{\Sigma\ (T + Te)}$$

where,

Fs : Safety of safety
C : Cohesion, based on the total stress, of materials on the slip circle
L : Base length of slice
N : Normal force of slice
Ne : Normal force of seismic load

ϕ : Internal friction angle, on the basis of total stress, of materials on the slip circle

T : Tangential force of slice

Te : Tangential force of seismic load

Table F.1.1. Present Drainage Area by the System

No.	Drainage System	Area (ha) Inside the Study Area	Outside the Study Area	Total	Main Drainage Canal
1	The left bank of Rio Matina 1	30	-	30	Rio Matina
2	The left bank of Rio Matina 2	80	-	80	Rio Matina
3	The left bank of Rio Matina 3	140	-	140	Rio Matina
4	The left bank of Rio Matina 4	10	-	10	Rio Matina
5	The left bank of Rio Matina 5	20	-	20	Rio Matina
6	The left bank of Rio Matina 6	30	-	30	Rio Matina
7	Direct Flow into Canal del Tortuguero 1	1,730	-	1,730	Canal del Tortuguero
8	Direct Flow into Canal del Tortuguero 2	1,610	-	1,610	Canal del Tortuguero
9	Canal Malcriado	1,650	-	1,650	Quebrada Lyon
10	Quebrada Lyon	2,390	-	2,390	Laguna Chorejal
11	Canal Principal	2,140	-	2,140	Laguna Chorejal
12	The left bank of Rio Barbilla 1	360	-	360	Rio Barbilla
13	The left bank of Rio Barbilla 2	310	-	310	Rio Barbilla
14	Canal Pama	1,850	-	1,850	Canal Sara
15	Canal Sara	2,310	-	2,310	Rio Madre de Dios
16	Rio Veintiseis	1,240	500	1,740	Rio Madre de Dios
17	Rio Madre de Dios	2,780	9,070	11,850	Rio Madre de Dios
18	The left bank of Rio Chirripo	120	80	200	Rio Chirripo
19	The right bank of Rio Barbilla 1	440	4,150	4,590	Rio Barbilla
20	The right bank of Rio Barbilla 2	210	200	410	Rio Barbilla
	Sub-total	19,450	14,000	33,450	
	Rio Barbilla Area	50	-	50	
	TOTAL	19,450	14,000	33,500	

Table F.1.2. Present Drainage System

Unit : km

No.	Drainage System	Small River	Main Canal	Small Canal	Total
1	The left bank of Rio Matina 1	1			1
2	The left bank of Rio Matina 2				
3	The left bank of Rio Matina 3	1			1
4	The left bank of Rio Matina 4				
5	The left bank of Rio Matina 5				
6	The left bank of Rio Matina 6				
7	Direct Flow into Canal del Tortuguero 1	11			11
8	Direct Flow into Canal del Tortuguero 2	11		2	13
9	Canal Malcriado	3	10	5	18
10	Quebrada Lyon	18	7	7	32
11	Canal Principal	1	21	7	29
12	The left bank of Rio Barbilla 1	3			3
13	The left bank of Rio Barbilla 2	4			4
14	Canal Pama	25	6	15	46
15	Canal Sara	17	13	20	50
16	Rio Veintiseis	23			23
17	Rio Madre de Dios	52	3	21	76
18	The left bank of Rio Chirripo	1			1
19	The right bank of Rio Barbilla 1	10		5	15
20	The right bank of Rio Barbilla 2	2			2
14	Total	183	60	82	325

Note: 1. The length of Rio Madre de Dios is included.
2. The drainage canals of the banana plantation and the CATIE laboratory are not included.
3. The length of Canal del Tortuguero is not included.

Table F.1.3.(1) Drainage Capacities of Existing Facilities

$V = 1/n \times T^{1/2} \times R^{2/3}$ m/s
$Q = V \times A$ m^3/s
$N = Q/Q'$

P : Wetted Perimeter (m)
R : Hydraulic Mean Depth (m)
I : Gradient

A : Cross-section (m^2)
n : Roughness Coefficient
Q': Discharge (m^3/s)

Drainage System	Facilities	Location	I	n	A	P	R	V	Q	Probability of 5 years: Watershed	Probability of 5 years: Q	N
					m^2	m	m	m/s	m^3/s	ha	m^3/s	%
9	Canal Cocaleca	Downstream (virgin forest)	1/3,000	0.040	12.0	10.0	1.20	0.52	6.2	360	11.4	54
9	Canal Malcriado	Banana plantation	1/1,000	0.035	22.4	14.4	1.56	1.22	27.3	250	7.9	345
9	Canal Malcriado	Near banana plantation	1/1,000	0.035	13.9	14.3	0.97	0.88	12.2	300	9.5	128
9	Canal Malcriado	Downstream of banana plantation (remote)	1/1,500	0.040	12.0	10.0	1.20	0.73	8.8	1,520	48.3	18
10	Canal Abaca	Near banana plantation	1/1,000	0.040	35.9	17.2	2.09	1.30	46.7	220	7.3	640
10	Canal Abaca	Downstream of banana plantation (remote)	1/1,500	0.040	6.3	12.1	0.52	0.42	2.7	850	27.3	10
10	Quebrada Lyon	Near banana plantation	1/1,500	0.035	20.0	13.0	1.33	0.98	19.6	310	9.8	200
10	Quebrada Lyon	Midstream of the canal	1/3,000	0.040	16.4	11.7	1.40	0.57	9.4	860	27.3	34
10	Quebrada Lyon	Downstream of the canal	1/1,500	0.040	2.3	7.6	0.30	0.29	0.7	1,150	20.5	3
11	Canal Principal	Banana plantation	1/500	0.035	17.2	14.6	1.18	1.43	24.6	360	11.4	216
11	Canal Principal	Banana plantation	1/1,500	0.035	21.0	13.0	1.62	1.02	21.4	585	18.6	115
11	Canal Principal	Banana plantation	1/1,000	0.035	73.2	24.5	2.99	1.88	137.6	980	31.1	442
11	Canal Principal	Near banana plantation	1/1,000	0.035	29.6	14.5	2.04	1.45	42.9	1,825	32.5	132
11	Canal Principal	Downstream of banana plantation (remote, virgin forest)	1/3,000	0.035	27.7	14.0	1.98	0.82	22.7	2,140	38.2	59

Table F.1.3.(2) Drainage Capacities of Existing Facilities

$V = 1/n \times I^{1/2} \times R^{2/3}$ m/s
$Q = V \times A$ m^3/s
$N = Q/Q'$ -

P : Wetted Perimeter (m)
R : Hydraulic Mean Depth (m)
I : Gradient

A : Cross-section (m^2)
n : Roughness Coefficient
Q': Discharge (m^3/s)

Drainage System	Facilities	Location	I	n	A	P	R	V	Q	Probability of 5 years		N
										Watershed	Q	
					m^2	m	m	m/s	m^3/s	ha	m^3/s	%
12	-	Downstream of the canal	1/800	0.040	4.0	6.1	0.61	0.66	2.6	120	5.7	46
13	Quebrada Laguna	Midstream of the canal	1/1,500	0.040	7.6	7.4	1.02	0.66	5.0	150	7.2	69
14	Canal Pama	Midstream of the canal	1/3,000	0.035	8.4	9.0	0.93	0.50	4.2	575	18.3	22
14	Canal Pama	Downstream of the canal	1/3,000	0.035	27.5	14.5	1.90	0.80	22.0	1,850	58.7	37
14	-	Midstream of the canal	1/2,000	0.040	22.3	14.5	1.54	0.74	16.5	345	11.0	150
15	Canal Sara	Upstream of the canal	1/2,500	0.040	23.2	9.3	2.49	0.92	21.3	640	20.3	105
15	Canal Sara	Midstream of the canal	1/1,700	0.035	30.8	15.1	2.04	1.12	34.5	1,985	63.0	55
15	Canal Sara	Downstream of the canal	1/3,000	0.035	42.0	19.4	2.16	0.87	36.5	4,160	74.3	49
16	Rio Veintiseis	Upstream of the canal (upstream side of railway)	1/300	0.040	37.0	20.8	1.78	2.12	78.4	785	37.5	209
16	Rio Veintiseis	Midstream of the canal (upstream side of railway)	1/400	0.040	42.3	18.0	2.35	2.21	93.5	1,270	60.6	154
16	Rio Veintiseis	Downstream of the canal (Downstream side of railway)	1/2,000	0.040	24.4	17.0	1.44	0.72	17.6	1,690	53.7	33
17	Rio Madre de Dios	National Road No.32	1/60	0.040	30.3	31.0	0.98	3.16	95.7	2,350	112.2	85
17	Rio Madre de Dios	Railway Bridge	1/60	0.040	49.3	51.9	0.95	3.12	153.8	2,630	125.6	122
17	Rio Madre de Dios	ASUBANA Laboratory	1/100	0.040	114.6	67.7	1.58	3.54	405.7	3,280	104.1	390
17	Rio Madre de Dios	1km from Sara toward the south	1/2,200	0.040	77.5	24.9	3.11	1.14	88.4	5,395	171.3	52

Table F.1.3.(3) Drainage Capacities of Existing Facilities

$V = 1/n \times I^{1/2} \times R^{2/3}$ m/s
$Q = V \times A$ m^3/s
$N = Q/Q'$

P : Wetted Perimeter (m)
R : Hydraulic Mean Depth (m)
I : Gradient

A : Cross-section (m^2)
n : Roughness Coefficient
Q': Discharge (m^3/s)

Drainage System	Facilities	Location	I	n	A	P	R	V	Q	Probability of 5 years: Watershed	Probability of 5 years: Q	N
					m^2	m	m	m/s	m^3/s	ha	m^3/s	%
17	Rio Madre de Dios	2km from Sara toward the north	1/1,000	0.040	44.4	21.2	2.09	1.29	57.3	6,255	198.6	29
17	Rio Madre de Dios	Water Gauge Station	The capacity of Rio Madre de Dios was analyzed by using the uniform flow based on the river measurement.					1.16	110.0	10,800	193.0	57
17	Rio Madre de Dios	Point meeting with Canal Sara						0.75	140.0	16,960	303.1	46
17	Rio Madre de Dios	1km downstream from point meeting with Canal Sara						0.27	80.0	16,960	303.1	25
17	Rio Madre de Dios	Point meeting with navigation canal						0.86	120.0	19,050	340.4	35
17	Small canals of Sara settlement		1/1,500	0.040	5.6	7.1	0.79	0.55	3.1	6	1.9	164
17	Main canals of Sara settlement		1/1,000	0.040	8.1	6.3	1.28	0.93	7.5	520	18.6	40
19	Rio Aquas Claras	Downstream of the canal	1/1,000	0.040	27.2	14.2	1.92	1.22	33.2	2,680	128.5	26
19	Rio San Miguel	Downstream of the canal	1/1,000	0.040	16.4	16.3	1.01	0.80	13.1	1,430	68.3	19

Table F.1.4. Drainage Capacities of Crossing-Sections of Existing Facilities

$V = 1/n \times I^{1/2} \times R^{2/3}$ m/s
$Q = V \times A$ m^3/s
$N = Q/Q'$

P : Wetted Perimeter (m)
R : Hydraulic Mean Depth (m)
I : Gradient

A : Cross-section (m^2)
n : Roughness Coefficient
Q': Discharge (m^3/s)

Drainage System	Facilities	Section	Location	I	n	A	P	R	V	Q	Probability of 5 years: Watershed	Probability of 5 years: Q	N
						m^2	m	m	m/s	m^3/s	ha	m^3/s	%
9	Canal Malcriado	Road Bridge	Cuatro Millas	1/1,500	0.035	12.0	10.0	1.20	0.83	9.7	1,110	35.2	28
9	Canal Malcriado	Conduit	La Luisa	1/1,000	0.030	2.0	5.0	0.40	0.57	1.1	330	10.5	10
10	Quebrada Lyon	Road Bridge	Veinticuatro Millas	1/420	0.035	20.3	14.8	1.37	1.72	34.9	360	11.4	306
10	Quebrada Lyon	Iron Bridge	Veinticuatro Millas	1/420	0.035	20.0	21.0	0.95	1.34	26.8	360	11.4	235
11	Canal Principal	Road Bridge	Bataan	1/700	0.035	24.0	14.0	1.71	1.55	37.2	285	9.0	413
11	Canal Principal	Iron Bridge	Bataan	1/700	0.035	28.9	11.8	2.45	1.96	56.7	285	9.0	630
11	Canal Principal	Road Bridge	Santa Marta	1/1,000	0.035	29.6	14.5	2.04	1.69	50.0	1,825	31.4	159
11	Canal Principal	Road Bridge	Helvetia	1/3,000	0.035	27.7	14.0	1.98	0.82	22.7	2,140	38.2	59
12	-	Conduit	Matina	1/1,500	0.030	0.8	3.1	0.26	0.35	0.3	120	5.7	5
13	Quebrada Laguna	Drain Work	Davao	1/1,500	0.030	0.8	3.1	0.26	0.35	0.3	150	7.2	4
14	Canal Pama	Road Bridge	Helvetia	1/3,000	0.035	10.8	9.4	1.15	0.58	6.3	575	18.3	34
15	Canal Sara	Road Bridge	Goschen	1/1,700	0.035	30.8	15.1	1.61	1.12	34.5	1,985	35.5	97
15	-	Conduit	Goschen	1/1,500	0.030	0.3	1.9	0.15	0.24	0.1	50	1.6	6
15	-	Conduit	Goschen	1/1,500	0.030	0.8	3.1	0.26	0.35	0.3	50	1.5	19
15	-	Conduit	Goschen	1/1,500	0.030	0.5	2.5	0.20	0.29	0.2	200	6.4	3
16	Rio Veintiseise	Road Bridge	Veintiseis Millas	1/300	0.035	37.0	20.8	1.78	2.42	89.7	780	37.2	240
16	Rio Veintiseise	Road Bridge	Veintiseis Millas	1/1,000	0.035	35.0	18.1	1.93	1.40	49.0	1,465	70.0	70
16	Rio Veintiseise	Iron Bridge	Veintiseis Millas	1/1,000	0.035	39.2	28.3	1.39	1.12	43.9	1,465	70.0	63
19	-	Road Bridge	Near Ruta 32	1/800	0.035	30.5	20.0	2.53	1.88	94.9	450	15.1	590

Table F.2.1. Present Drainage Condition

Unit : ha

Classification											
Swamp Area	Poor Drainage Area				Good Drainage Area				Others	Total	Remarks
I	II	III	IV	Sub-total	V	VI	VII	Sub-total			
3,380	1,960	3,480	6,230	11,670	350	1,170	2,070	3,590	860	19,500	

I : Swamp area
II.: Poor drainage area on the plain land, which more than 0.6m of water depth occurs under flood condition of probability 5 years.
III : Poor drainage area on the plain land, which 0.6m - 0.3m of water depth occurs under flood condition of probability 5 years.
IV : Poor drainage area on the plain land, which less than 0.3m of water depth occurs under flood condition of probability 5 years.
V : Good drainage area around downstream of Rio Matina.
VI : Good drainage area on the slope land
VII : Banana plantation and Luzon settlement area of 110ha

Others : Include rural area and examination area.

Table F.2.2. Increment of Agricultural Production with the Drainage Improvement

(US$1.0 = ₡68.75)

Crops	Whole Area (ha)	Damaged Area (ha)	Increased Production per ha (t/ha)	Total Production (t)	Unit Price (₡/t)	Increased Production Cost	
						1,000 ₡	1,000 US$
Rice	2,900	2,900	0.80	2,320	14,200	32,944	479
Kidney Bean	40	30	0.50	15	35,788	537	8
Maize	350	280	0.50	140	13,669	1,914	28
Anual Crops	240	190	0.80	152	14,000	2,128	31
Banana	1,960	-	-	-	-		
Cacao	1,540	1,160	0.45	522	95,000	49,590	721
Perennial Crops	620	620	3.00	1,860	8,500	15,810	230
Cattle Breeding	3,510	3,250	0.05	163	50,000	8,150	119
Total	11,160	8,430		5,172		111,073	1,616

Table F.3.1. Capacity of Large Rivers

Name of River	Distance from Beginning Point	Water Depth (H)	Cross-Sectional Area (A)	Wetted Perimeter (P)	Hydraulic Mean Depth (R)	Velocity (V)	Capacity (Q)	1/5 Probability Discharge (Q')	Q/Q'
	km	m	m^2	m	m	m/s	m^3/s	m^3/s	%
MATINA	0	5.4	385	167	2.3	0.52	200	2,248	9
	2	6.5	828	169	4.9	2.17	1,800	2,248	80
	4	9.6	702	160	4.4	1.35	950	2,248	42
	6	5.5	571	139	4.1	0.53	300	2,248	13
	8	10.0	600	171	3.5	0.50	300	2,248	13
	10	7.5	603	123	4.9	1.41	850	2,248	38
	12	7.0	567	135	4.2	1.32	750	2,248	33
	14	6.0	439	116	3.8	1.37	600	2,248	27
	16	5.3	498	128	3.9	1.11	550	2,248	24
	18	5.5	473	135	3.5	0.64	300	2,248	13
	20	5.0	364	114	3.2	1.37	500	2,248	22
	22	4.7	435	181	2.4	1.15	500	2,248	22
BARBILLA	1	3.0	248	87	2.8	2.25	550	528	106
	3	4.3	369	132	2.8	0.54	200	528	38
	5	4.6	219	95	2.3	1.60	350	528	66
	7	5.8	411	133	3.1	1.10	450	528	85
	9	5.8	242	110	2.2	0.41	100	528	19
CHIRRIPO	0	6.2	1,396	325	4.3	1.29	1,800	1,760	102
	1	6.9	468	90	5.2	2.78	1,300	1,760	74

Table F. 4.1 Estimation of Flood Concentration Time

Main Drainage Canal	Lower Courses					Middle Courses				
	Position	L(m)	H(m)	t(min)	t'(hr)	Position	L(m)	H(m)	t(min)	t'(hr)
No.1 (l=11.75km)	No.0	13,650	13	736	12	No.6+0.80	6,850	8	326	5
No.2 (l=11.70km)	No.0	13,700	14	708	11	No.6+0.50	7,200	7	383	6
No.3 (l= 9.50km)	No.0	12,500	10	749	12	No.5+0.95	6,550	8	304	5
No.4 (l=11.60km)	No.0	12,950	11	746	12	No.6+0.55	6,400	7	318	5
		(13,000)	(380)	(90)			(13,000)	(380)	(90)	
No.5 (l=14.40km)	No.0	14,400	19	639	12	No.6+0.15	8,250	14	314	6
		(4,000)	(74)	(36)						
No.6 (l= 2.65km)	No.0	2,650	4	108	2					
		(4,500)	(170)	(26)						
No.7 (l= 1.90km)	No.0	5,450	16	149	2					
		(3,000)	(170)	(14)						
No.8 (l= 1.00km)	No.0	5,000	16	130	2					

$t = 0.00083L/(H/L)^{0.6}$

t : Time of concentration in min.
t': Time of concentration in hour
H : The difference in elevation between the outlet and the most remote point measured by the map of 1/10,000 or 1/50,000 scale
L : Maximum length of flow in m

Note: The parentheses show the estimation of steep slope courses.
The times of concentration for the main drainage canals of No.5 - No.8 include the value of steep slope courses.

Table F.4.2.(1) Design Discharge of Proposed Main Drainage Canal

Name of Canal	Section	Length of Canal	Watershed	Unit Discharge	Design Discharge	Name of Secondary Canal	Remarks
		km	km^2	$m^3/s/km^2$	m^3/s		
Main Drainage Canal No.1	No.0 - No.2+0.60	2.60	41.90	1.787	74.9	No.1-1	
	No.2+0.60 - No.3+0.75	1.15	34.75	1.787	62.1	No.1-2, 1-3, 1-4	
	No.3+0.75 - No.4+0.80	1.05	24.65	1.787	44.0	No.1-5	
	No.4+0.80 - No.5+0.80	1.00	23.45	1.787	42.4	No.1-6	
	No.5+0.80 - No.6+0.80	1.00	22.25	1.787	42.4	No.1-7, 1-8	
	No.6+0.80 - No.7+0.80	1.00	13.35	3.175	42.4	No.1-9	
	No.7+0.80 - No.8+0.80	1.00	12.15	3.175	38.6	No.1-10	
	No.8+0.80 - No.9+0.50	0.70	11.00	3.175	34.9	No.1-11	
	No.9+0.50 - No.10+0.20	0.70	9.70	3.175	30.8	No.1-12	
	No.10+0.20- No.11+0.75	1.55	3.60	3.175	11.4		
	Sub-total	11.75					
Main Drainage Canal No.2	No.0 - No.1+0.85	1.85	23.55	1.787	42.1	No.2-1	
	No.1+0.85 - No.2+0.80	0.95	22.45	1.787	40.1	No.2-2	
	No.2+0.80 - No.3+0.80	1.00	21.25	1.787	38.0	No.2-3	
	No.3+0.80 - No.4+0.80	1.00	20.05	1.787	35.8	No.2-4	
	No.4+0.80 - No.5+0.80	1.00	18.85	1.787	34.1	No.2-5	
	No.5+0.80 - No.6+0.50	0.70	17.60	1.787	34.1	No.2-6	
	No.6+0.50 - No.6+0.95	0.45	10.75	3.175	34.1		
	No.5+0.95 - No.9	2.05	9.60	3.175	30.5		
	No.9 - No.9+0.85	0.85	7.10	3.175	22.5	No.2-7	
	No.9+0.85 - No.11	1.15	3.85	3.175	12.2		
	No.11 - No.11+0.70	0.70	2.85	3.175	9.0		
	Sub-total	11.70					
Main Drainage Canal No.3	No.0 - No.2+0.10	2.10	19.90	1.787	35.5	No.3-1	
	No.2+0.10 - No.3+0.75	1.65	12.95	1.787	23.1	No.3-2	
	No.3+0.75 - No.5+0.95	2.20	8.05	1.787	15.9	No.3-3	
	No.5+0.95 - No.7	1.05	5.00	3.175	15.9	No.3-4	
	No.7 - No.9+0.50	2.50	2.20	3.175	7.0		
	Sub-total	9.50					

Table F.4.2.(2) Design Discharge of Proposed Main Drainage Canal

Name of Canal	Section	Length of Canal	Watershed	Unit Discharge	Design Discharge	Name of Secondary Canal	Remarks
		km	km^2	$m^3/s/km^2$	m^3/s		
Main Drainage Canal No.4	No.0 - No.1+0.10	1.10	48.00	1.787	85.8		Main Drainage Canal No.3 joins.
	No.1+0.10 - No.2+0.60	1.50	27.85	1.787	49.8	No.4-1	
	No.2+0.60 - No.4+0.30	1.70	18.45	1.787	33.0	No.4-2, 4-3	
	No.4+0.30 - No.6+0.55	2.25	13.00	1.787	29.8	No.4-4, 4-5	
	No.6+0.55 - No.8+0.30	1.75	9.40	3.175	29.8	No.4-6, 4-7	
	No.8+0.30 - No.9+0.60	1.30	6.15	3.175	19.5	No.4-8	
	No.9+0.60 - No.10+0.60	1.00	4.60	3.175	14.6	No.4-9, 4-10	
	No.10+0.60- No.11+0.60	1.00	2.05	3.175	6.5	No.4-11, 4-12	
	Sub-total	11.60					
Main Drainage Canal No.5	No.0 - No.0+0.60	0.60	169.60	1.787	303.1		Main Drainage Canal No.4 joins.
	No.0+0.60 - No.2+0.60	2.00	116.40	1.787	208.0		
	No.2+0.60 - No.4+0.45	1.85	108.05	1.787	193.1		
	No.4+0.45 - No.6+0.45	1.70	62.55	1.787	176.1	No.5-1	
	No.6+0.15 - No.10	3.85	55.45	3.175	176.1		Main Drainage Canal No.6 joins.
	No.10 - No.13+0.25	3.25	53.95	3.175	171.3		
	No.13+0.25- No.14+0.40	1.15	32.80	3.175	104.1		
	Sub-total	14.40					
Main Drainage Canal No.6	No.0 - No.1+0.50	1.50	16.20	4.775	77.4		
	No.1+0.50 - No.2+0.65	1.15	14.65	4.775	70.0		
	Sub-total	2.65					
Main Drainage Canal No.7	No.0 - No.1+0.90	1.90	26.80	4.775	128.0		
	Sub-total	1.90					
Main Drainage Canal No.8	No.0 - No.1	1.00	14.30	4.775	68.3		
	Sub-total	1.00					
	Grand Total	64.50					

Table F.4.3.(1) Length and Capacity of Main Drainage Canal

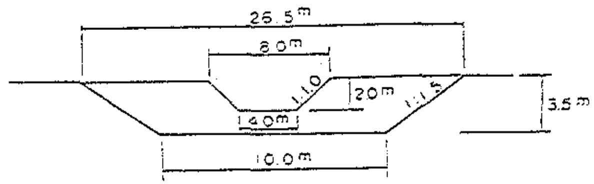
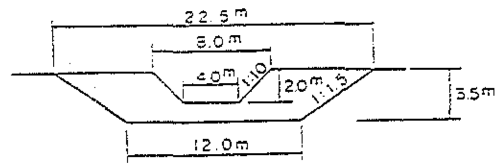
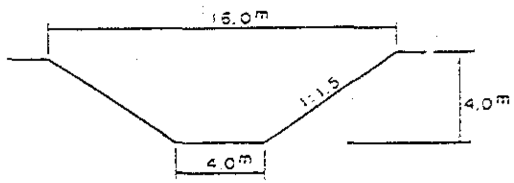
Name of Canal	Section	Design Discharge	Length: Existing	Length: Rehabilitation	Length: New Construction	Length: Total	Cross-Section Capacity (by Manning Formula)
		m^3/s	km	km	km	km	
No.1	No.0 - No.2+0.60	74.9		2.60		2.60	 I = 1/1,800 n = 0.030 V = 0.91m/s - 1.18m/s Water Depth (H) = 3.09m - 3.60m Note: The capacity is shown in Table F.4.4.
	No.2+0.60 - No.3+0.75	62.1		1.15		1.15	 I = 1/1,800 n = 0.030 V = 1.18m/s - 1.26m/s Water Depth (H) = 3.00m - 3.09m Note: The capacity is shown in Table F.4.4.
	No.3+0.75 - No.4+0.80	44.0			1.05	1.05	 I = 1/1,400 n = 0.030 V = 1.26m/s - 1.47m/s Water Depth (H) = 3.09m - 3.32m Note: The capacity is shown in Table F.4.4.

Table F.4.3.(2) Length and Capacity of Main Drainage Canal

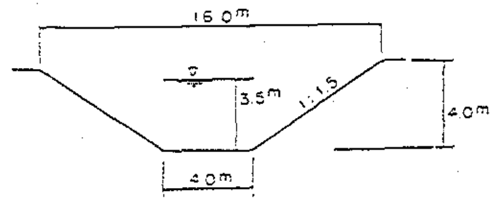
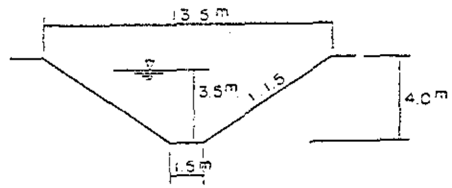
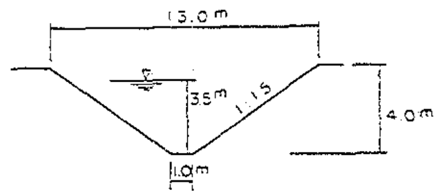
Name of Canal	Section	Design Discharge	Length Existing	Length Rehabilitation	Length New Construction	Length Total	Cross-Section Capacity (by Manning Formula)
		m^3/s	km	km	km	km	
No.1	No.4+0.80 - No.8+0.80	42.4 - 38.6			4.00	4.00	 I = 1/1,400 n = 0.030 A = 32.4m^2 P = 16.6m R = A/P = 1.95 $R^{2/3}$ = 1.561 V = 1/n x $R^{2/3}$ x $I^{1/2}$ = 1.39m/s Q = VxA =45.0m^3/s>42.4m^3/s
	No.8+0.80 - No.9+0.50	34.9			0.70	0.70	 I = 1/1,000 n = 0.030 A = 23.6m^2 P = 14.1m R = A/P = 1.67 $R^{2/3}$ = 1.410 V = 1/n x $R^{2/3}$ x $I^{1/2}$ = 1.48m/s Q = VxA =34.9m^3/s>34.9m^3/s
	No.9+0.50 - No.10+0.20	30.8			0.70	0.70	 I = 1/1,000 n = 0.030 A = 21.9m^2 P = 13.6m R = A/P = 1.61 $R^{2/3}$ = 1.374 V = 1/n x $R^{2/3}$ x $I^{1/2}$ = 1.45m/s Q = VxA =31.8m^3/s>30.8m^3/s

Table F.4.3.(3) Length and Capacity of Main Drainage Canal

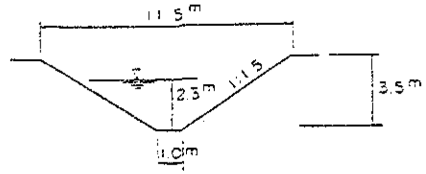
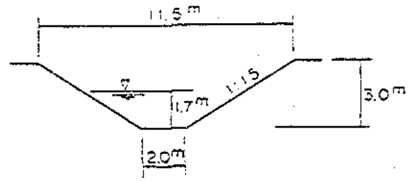
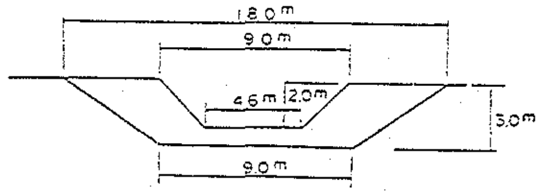
Name of Canal	Section	Design Discharge	Length: Existing	Length: Rehabilitation	Length: New Construction	Length: Total	Cross-Section Capacity (by Manning Formula)
		m^3/s	km	km	km	km	
No.1	No.10+0.20 - No.11+0.10	11.4			0.90	0.90	 I = 1/1,000 n = 0.030 A = $10.2m^2$ P = 9.3m R = A/P = 1.10 $R^{2/3}$ = 1.066 V = $1/n \times R^{2/3} \times I^{1/2}$ = 1.12m/s Q = VxA =$11.4m^3/s > 11.4m^3/s$
	No.11+0.10 - No.11+0.75	11.4	0.65			0.65	 I = 1/420 n = 0.030 A = $8.0m^2$ P = 8.4m R = A/P = 0.95 $R^{2/3}$ = 0.966 V = $1/n \times R^{2/3} \times I^{1/2}$ = 1.55m/s Q = VxA =$12.4m^3/s > 11.4m^3/s$
Sub-Total			0.65	3.75	7.35	11.75	
No.2	No.0 - No.1+0.85	42.1		1.85		1.85	 I = 1/1,200 n = 0.030 V = 0.75m/s - 1.15m/s Water Depth (H) = 2.78m - 3.80m Note: The capacity is shown in Table F.4.4.

Table F.4.3.(4) Length and Capacity of Main Drainage Canal

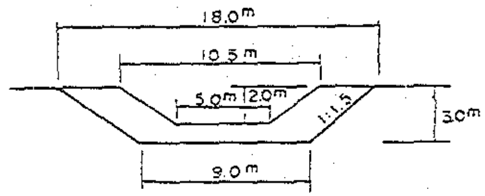
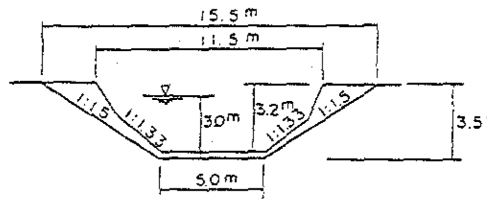
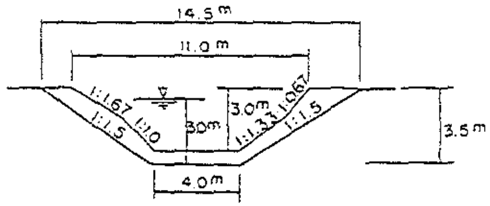
Name of Canal	Section	Design Discharge	Length: Existing	Length: Rehabilitation	Length: New Construction	Length: Total	Cross-Section Capacity (by Manning Formula)
		m^3/s	km	km	km	km	
No.2	No.1+0.85 - No.2+0.80	40.1		0.95		0.95	 I = 1/1,200 n = 0.030 V = 1.15m/s - 1.34m/s Water Depth (H) = 2.62m - 2.78m Note: The capacity is shown in Table F.4.4.
	No.2+0.80 - No.4+0.80	38.0 - 35.8		2.00		2.00	 I = 1/1,200 n = 0.030 $A = 28.5m^2$ P = 15.8m R = A/P = 1.80 $R^{2/3} = 1.480$ $V = 1/n \times R^{2/3} \times I^{1/2} = 1.42m/s$ $Q = V \times A = 40.5m^3/s > 38.0m^3/s$
	No.4+0.80 - No.5+0.80	34.1		1.00		1.00	 I = 1/1,200 n = 0.030 $A = 25.5m^2$ P = 14.8m R = A/P = 1.72 $R^{2/3} = 1.435$ $V = 1/n \times R^{2/3} \times I^{1/2} = 1.38m/s$ $Q = V \times A = 35.2m^3/s > 34.1m^3/s$

Table F.4.3.(5) Length and Capacity of Main Drainage Canal

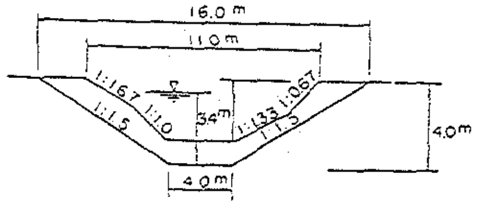
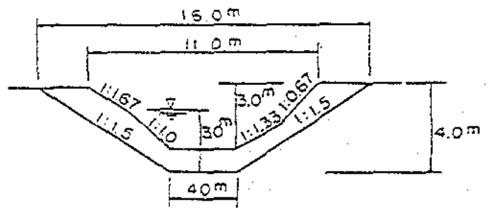
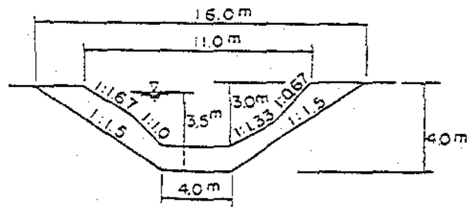
Name of Canal	Section	Design Discharge	Length: Existing	Length: Rehabilitation	Length: New Construction	Length: Total	Cross-Section Capacity (by Manning Formula)
		m^3/s	km	km	km	km	
No.2	No.5+0.80 - No.6+0.50	34.1		0.70		0.70	 $I = 1/2,000$ $n = 0.030$ $A = 30.9m^2$ $P = 16.3m$ $R = A/P = 1.90$ $R^{2/3} = 1.535$ $V = 1/n \times R^{2/3} \times I^{1/2} = 1.14m/s$ $Q = VxA = 35.4m^3/s > 34.1m^3/s$
	No.6+0.50 - No.6+0.95	34.1		0.45		0.45	 $I = 1/1,200$ $n = 0.030$ $A = 25.5m^2$ $P = 14.8m$ $R = A/P = 1.72$ $R^{2/3} = 1.435$ $V = 1/n \times R^{2/3} \times I^{1/2} = 1.38m/s$ $Q = VxA = 35.2m^3/s > 34.1m^3/s$
	No.6+0.95 - No.7+0.50	30.5		0.55		0.55	 $I = 1/1,200$ $n = 0.030$ $A = 23.0m^2$ $P = 14.1m$ $R = A/P = 1.63$ $R^{2/3} = 1.385$ $V = 1/n \times R^{2/3} \times I^{1/2} = 1.33m/s$ $Q = VxA = 30.5m^3/s > 30.5m^3/s$

Table F.4.3.(6) Length and Capacity of Main Drainage Canal

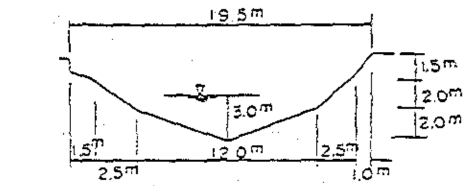
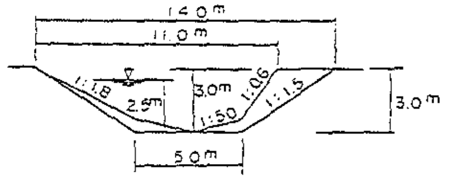
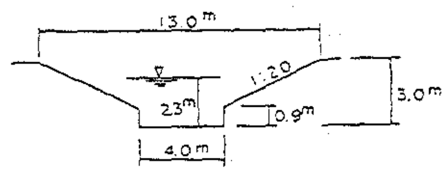
Name of Canal	Section	Design Discharge	Length: Existing	Length: Rehabilitation	Length: New Construction	Length: Total	Cross-Section Capacity (by Manning Formula)
		m^3/s	km	km	km	km	
No.2	No.7+0.50 - No.9	30.5	1.50			1.50	 I = 1/1,250 n = 0.030 A = 25.3m^2 P = 15.8m R = A/P = 1.60 $R^{2/3}$ = 1.368 V = 1/n x $R^{2/3}$ x $I^{1/2}$ = 1.29m/s Q = VxA =32.6m^3/s>30.5m^3/s
	No.9 - No.9+0.85	22.5		0.85		0.85	 I = 1/1,700 n = 0.030 A = 21.9m^2 P = 14.0m R = A/P = 1.56 $R^{2/3}$ = 1.345 V = 1/n x $R^{2/3}$ x $I^{1/2}$ = 1.09m/s Q = VxA =23.9m^3/s>22.5m^3/s
	No.9+0.85 - No.11	12.2	1.15			1.15	 I = 1/1,280 n = 0.030 A = 13.1m^2 P = 12.1m R = A/P = 1.08 $R^{2/3}$ = 1.053 V = 1/n x $R^{2/3}$ x $I^{1/2}$ = 0.98m/s Q = VxA =12.8m^3/s>12.2m^3/s

Table F.4.3.(7) Length and Capacity of Main Drainage Canal

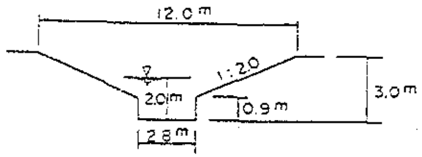
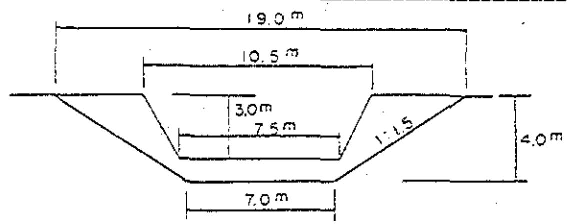
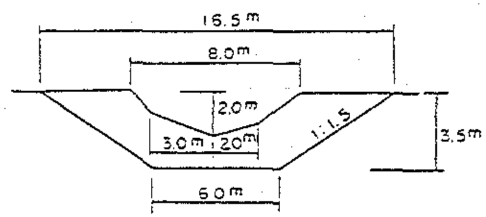
Name of Canal	Section	Design Discharge	Length: Existing	Length: Rehabilitation	Length: New Construction	Length: Total	Cross-Section Capacity (by Manning Formula)
		m^3/s	km	km	km	km	
No.2	No.11 - No.11+0.70	9.0	0.70			0.70	 I = 1/700 n = 0.030 A = $8.2m^2$ P = 9.9m R = A/P = 0.83 $R^{2/3}$ = 0.883 V = 1/n x $R^{2/3}$ x $I^{1/2}$ = 1.11m/s Q = VxA = $9.1m^3/s>9.0m^3/s$
Sub-Total			3.35	8.35		11.70	
No.3	No.0 - No.2+010	35.6		2.10		2.10	 I = 1/2,000 n = 0.030 V = 0.77m/s - 0.96m/s Water Depth (H) = 3.15m - 3.69m Note: The capacity is shown in Table F.4.4.
	No.2+0.10 - No.3+0.75	23.1		1.65		1.65	 I = I/2,000 n = 0.030 V = 0.84m/s - 0.96m/s Water Depth (H) = 2.73m - 3.15m Note: The capacity is shown in Table F.4.4.

Table F.4.3.(8) Length and Capacity of Main Drainage Canal

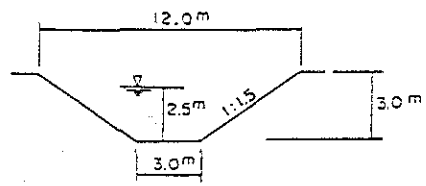
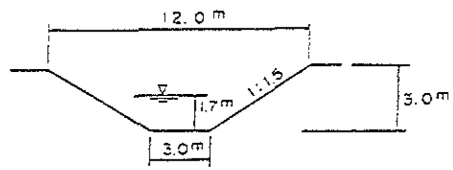
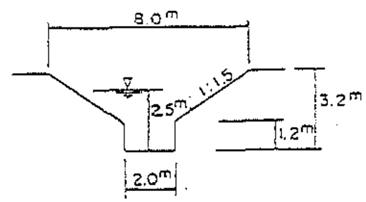
Name of Canal	Section	Design Discharge	Length: Existing	Length: Rehabilitation	Length: New Construction	Length: Total	Cross-Section Capacity (by Manning Formula)
		m^3/s	km	km	km	km	
No.3	No.3+0.75 - No.7	15.9			3.25	3.25	 $I = 1/1,800$ $n = 0.030$ $A = 16.9m^2$ $P = 12.0m$ $R = A/P = 1.41$ $R^{2/3} = 1.258$ $V = 1/n \times R^{2/3} \times I^{1/2} = 0.99m/s$ $Q = V \times A = 16.7m^3/s > 15.9m^3/s$
	No.7 - No.8+0.20	7.0			1.20	1.20	 $I = 1/1,800$ $n = 0.030$ $A = 9.4m^2$ $P = 9.1m$ $R = A/P = 1.08$ $R^{2/3} = 1.020$ $V = 1/n \times R^{2/3} \times I^{1/2} = 0.80m/s$ $Q = V \times A = 7.5m^3/s > 7.0m^3/s$
	No.8+0.20 - No.9+0.50	7.0	1.30			1.30	 $I = 1/870$ $n = 0.030$ $A = 7.5m^2$ $P = 9.1m$ $R = A/P = 0.82$ $R^{2/3} = 0.876$ $V = 1/n \times R^{2/3} \times I^{1/2} = 0.99m/s$ $Q = V \times A = 7.4m^3/s > 7.0m^3/s$
Sub-Total			1.30	3.75	4.45	9.50	

Table F.4.3.(9) Length and Capacity of Main Drainage Canal

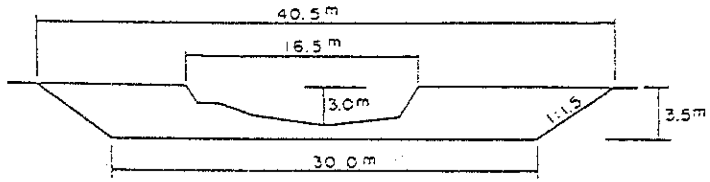
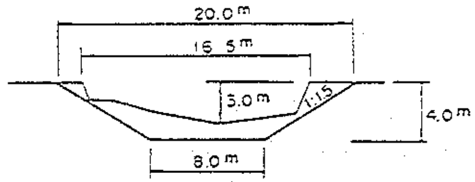
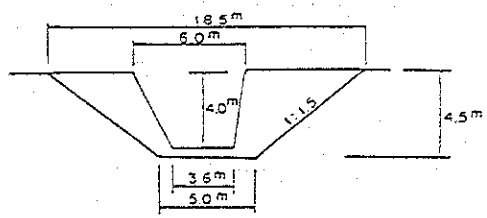
Name of Canal	Section	Design Discharge	Length: Existing	Length: Rehabilitation	Length: New Construction	Length: Total	Cross-Section Capacity (by Manning Formula)
		m^3/s	km	km	km	km	
No.4	No.0 - No.1+0.10	85.8		1.10		1.10	 I = 1/2,000 n = 0.030 V = 0.66m/s - 0.76m/s Water Depth (H) = 3.24m - 3.67m Note: The capacity is shown in Table F.4.4.
	No.1+0.10 - No.2+0.60	49.8		1.50		1.50	 I = 1/2,000 n = 0.030 V = 0.78m/s - 1.22m/s Water Depth (H) = 3.19m - 3.24m Note: The capacity is shown in Table F.4.4.
	No.2+0.60 - No.4+0.30	33.0		1.70		1.70	 I = 1/2,000 n = 0.030 V = 1.09m/s - 1.22m/s Water Depth (H) = 3.12m - 3.19m Note: The capacity is shown in Table F.4.4.

Table F.4.3.(10) Length and Capacity of Main Drainage Canal

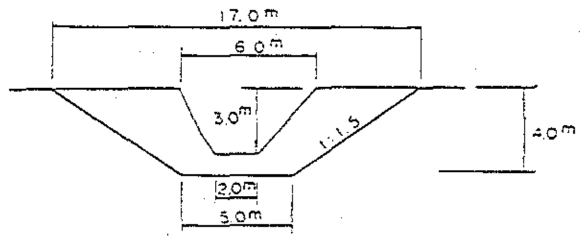
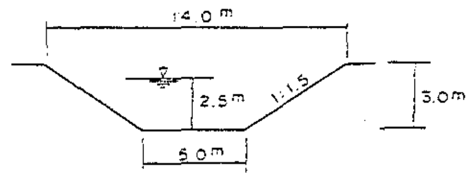
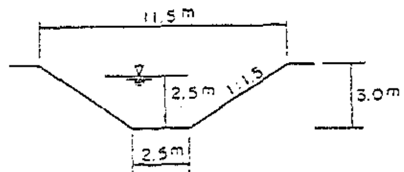
Name of Canal	Section	Design Discharge	Length: Existing	Length: Rehabilitation	Length: New Construction	Length: Total	Cross-Section Capacity (by Manning Formula)
		m^3/s	km	km	km	km	
No.4	No.4+0.30 - No.6+0.45	29.8 -		2.15		2.15	 I = 1/2,000 n = 0.030 V = 1.09m/s - 1.18m/s Water Depth = 2.77m - 3.12m Note: The capacity is shown in Table F.4.4.
	No.6+0.45 - No.8+0.30	29.8			1.85	1.85	 I = 1/1,000 n = 0.030 A = $21.9m^2$ P = 14.0m R = A/P = 1.56 $R^{2/3}$ = 1.345 V = 1/n x $R^{2/3}$ x $I^{1/2}$ = 1.42m/s Q = VxA = $31.1m^3/s > 29.8m^3/s$
	No.8+0.30 - No.9+0.60	19.5			1.30	1.30	 I = 1/1,000 n = 0.030 A = $15.6m^2$ P = 11.5m R = A/P = 1.36 $R^{2/3}$ = 1.227 V = 1/n x $R^{2/3}$ x $I^{1/2}$ = 1.29m/s Q = VxA = $20.1m^3/s > 19.5m^3/s$

Table F.4.3.(11) Length and Capacity of Main Drainage Canal

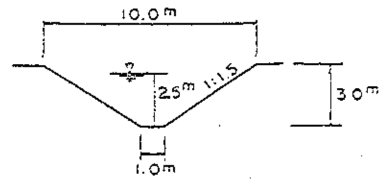
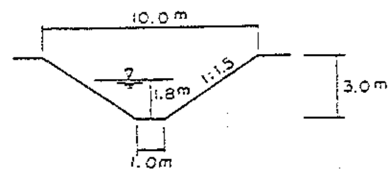
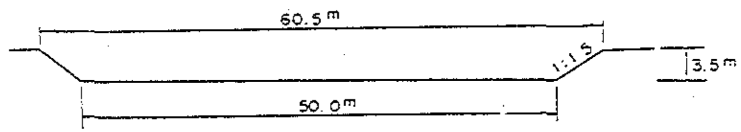
Name of Canal	Section	Design Discharge	Length: Existing	Length: Rehabilitation	Length: New Construction	Length: Total	Cross-Section Capacity (by Manning Formula)
		m^3/s	km	km	km	km	
No.4	No.9+0.60 - No.10+0.60	14.6			1.00	1.00	 I = 1/900 n = 0.030 A = 11.9m^2 P = 10.0m R = A/P = 1.19 $R^{2/3}$ = 1.123 V = 1/n x $R^{2/3}$ x $I^{1/2}$ = 1.25m/s Q = VxA =14.9m^3/s>14.6m^3/s
	No.10+0.60 - No.11+0.60	6.5			1.00	1.00	 I = 1/900 n = 0.030 A = 6.7m^2 P = 7.5m R = A/P = 0.89 $R^{2/3}$ = 0.925 V = 1/n x $R^{2/3}$ x $I^{1/2}$ = 1.03m/s Q = VxA =6.9m^3/s>6.5m^3/s
Sub-Total				6.45	5.15	11.60	
No.5	No.0 - No.0+0.60	303.1			0.60	0.60	 I = 1/2,400 n = 0.030 V = 1.27m/s - 1.31m/s Water Depth (H) = 3.90m - 3.82m *Note: The capacity is shown in Table F.4.4.*

Table F.4.3.(12) Length and Capacity of Main Drainage Canal

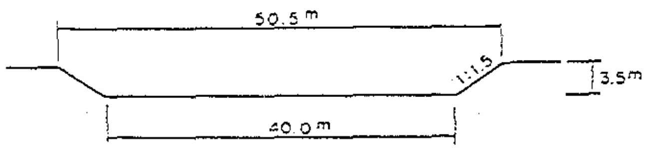
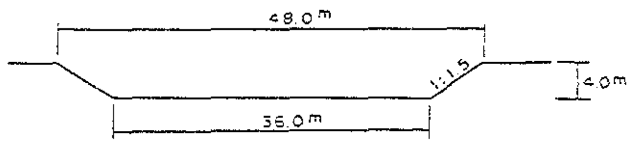
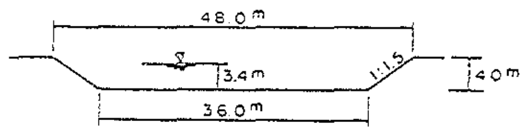
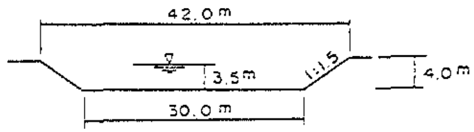
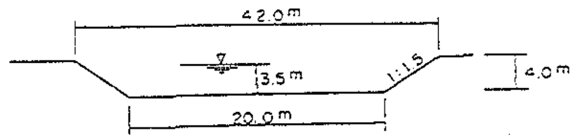
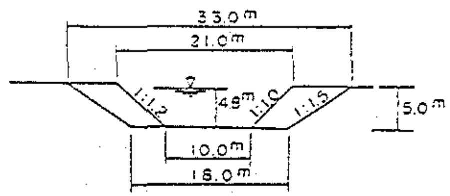
Name of Canal	Section	Design Discharge	Length: Existing	Length: Rehabilitation	Length: New Construction	Length: Total	Cross-Section Capacity (by Manning Formula)
		m^3/s	km	km	km	km	
No.5	No.0+0.60 - No.2+0.60	208.0			2.00	2.00	 I = 1/2,200 n = 0.030 P = 1.28m/s - 1.31m/s Water Depth (H) = 3.59m - 3.82m Note: The capacity is shown in Table F.4.4.
	No.2+0.60 - No.3	193.1			0.40	0.40	 I = 1/2,200 n = 0.030 V = 1.28m/s - 1.31m/s Water Depth (H) = 3.56m - 3.59m Note: The capacity is shown in Table F.4.4.
	No.3 - No.4+0.45	193.1			1.45	1.45	 I = 1/2,200 n = 0.030 $A = 139.7m^2$ P = 48.3m $R = A/P = 2.89$ $R^{2/3} = 2.030$ $V = 1/n \times R^{2/3} \times I^{1/2} = 1.44m/s$ $Q = V \times A = 201.2m^3/s > 193.7m^3/s$

Table F.4.3.(13) Length and Capacity of Main Drainage Canal

<table>
<tr><th rowspan="2">Name of Canal</th><th rowspan="2">Section</th><th rowspan="2">Design Discharge</th><th colspan="4">Length</th><th rowspan="2">Cross-Section Capacity (by Manning Formula)</th></tr>
<tr><th>Existing</th><th>Rehabilitation</th><th>New Construction</th><th>Total</th></tr>
<tr><td></td><td></td><td>m^3/s</td><td>km</td><td>km</td><td>km</td><td>km</td><td></td></tr>
<tr><td>No.5</td><td>No.4+0.45 - No.10</td><td>176.1</td><td></td><td></td><td>5.55</td><td>5.55</td><td>

I = 1/2,200 n = 0.030 A =123.4m²
P = 42.6m R = A/P = 2.90 $R^{2/3}$ = 2.034
V = 1/n x $R^{2/3}$ x $I^{1/2}$ = 1.44m/s Q = VxA =177.7m^3/s>176.1m^3/s</td></tr>
<tr><td></td><td>No.10 - No.11+0.25</td><td>171.3</td><td></td><td></td><td>1.25</td><td>1.25</td><td>

I = 1/2,200 n = 0.030 A =123.4m^2
P = 42.6m R = A/P = 2.9 $R^{2/3}$ = 2.034
V = 1/n x $R^{2/3}$ x $I^{1/2}$ = 1.44m/s Q = VxA =177.7m^3/s>171.3m^3/s</td></tr>
<tr><td></td><td>No.11+0.25 - No.13+0.25</td><td>171.3</td><td></td><td>2.00</td><td></td><td>2.00</td><td>

I = 1/2,200 n = 0.030 A =111.4m^2
P = 34.2m R = A/P = 3.26 $R^{2/3}$ = 2.199
V = 1/n x $R^{2/3}$ x $I^{1/2}$ = 1.56m/s Q = VxA =173.8m^3/s>171.3m^3/s</td></tr>
</table>

Table F.4.3.(14) Length and Capacity of Main Drainage Canal

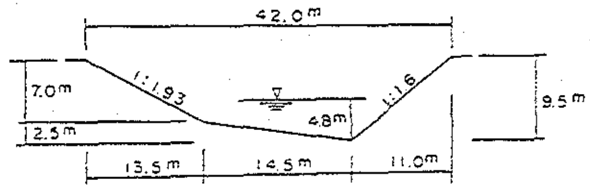
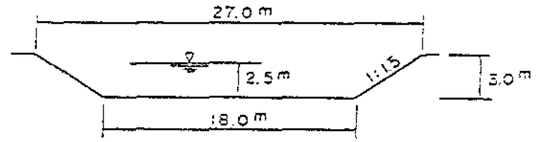
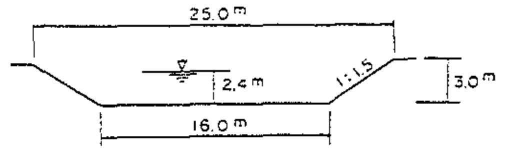
Name of Canal	Section	Design Discharge	Length: Existing	Length: Rehabilitation	Length: New Construction	Length: Total	Cross-Section Capacity (by Manning Formula)
		m^3/s	km	km	km	km	
No.5	No.13+0.25 - No.14+0.40	104.1	1.15			1.15	 I = 1/2,300 n = 0.030 A = 80.6m^2 P = 30.0m R = A/P = 2.69 $R^{2/3}$ = 1.935 V = 1/n x $R^{2/3}$ x $I^{1/2}$ = 1.35m/s Q = VxA =108.8m^3/s>104.1m^3/s
Sub-Total			1.15	2.00		11.70	
No.6	No.0 - No.1+050	77.4			1.50	1.50	 I = 1/1,400 n = 0.030 A =54.4m^2 P = 27.0m R = A/P = 2.01 $R^{2/3}$ = 1.593 V = 1/n x $R^{2/3}$ x $I^{1/2}$ = 1.42m/s Q = VxA =77.3m^3/s>77.4m^3/s
	No.1+0.50 - No.2+0.65	70.0			1.15	1.15	 I = 1/1.200 n = 0.030 A =47.0m^2 P = 24.7m R = A/P = 1.90 $R^{2/3}$ = 1.534 V = 1/n x $R^{2/3}$ x $I^{1/2}$ = 1.48m/s Q = VxA =69.6m^3/s>70.0m^3/s
Sub-Total					2.65	2.65	

Table F.4.3.(15) Length and Capacity of Main Drainage Canal

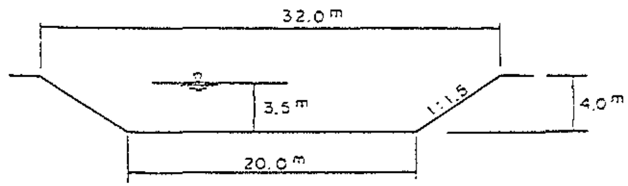
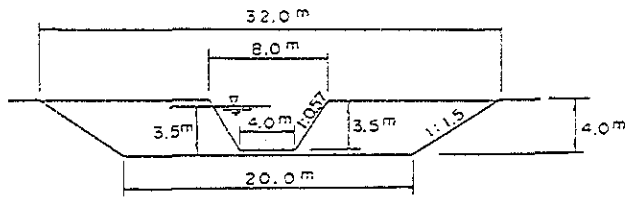
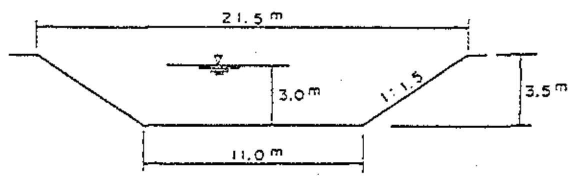
Name of Canal	Section	Design Discharge	Length: Existing	Length: Rehabilitation	Length: New Construction	Length: Total	Cross-Section Capacity (by Manning Formula)
		m^3/s	km	km	km	km	
No.7	No.0 - No.0+025	128.0			0.25	0.25	 I = 1/2,000 n = 0.030 $A = 88.4m^2$ P = 32.6m R = A/P = 2.71 $R^{2/3} = 1.944$ $V = 1/n \times R^{2/3} \times I^{1/2} = 1.45m/s$ $Q = V \times A = 128.2m^3/s > 128.0m^3/s$
	No.0+0.25 - No.1+0.90	128.0		1.65		1.65	 I = 1/2,000 n = 0.030 $A = 88.4m^2$ P = 32.6m R = A/P = 2.71 $R^{2/3} = 1.944$ $V = 1/n \times R^{2/3} \times I^{1/2} = 1.45m/s$ $Q = V \times A = 128.2m^3/s > 128.0m^3/s$
Sub-Total				1.65	0.25	1.90	
No.8	No.0 - No.1	68.3			1.00	1.00	 I = 1/1,400 n = 0.030 $A = 46.5m^2$ P = 21.8m R = A/P = 2.13 $R^{2/3} = 1.656$ $V = 1/n \times R^{2/3} \times I^{1/2} = 1.47m/s$ $Q = V \times A = 68.4m^3/s > 68.3m^3/s$
Sub-Total					1.00	1.00	

Table F.4.4.(1) Hydraulic Analysis of Main Drainage Canal by Uniform Flow

(1) Main Drainage Canal No.1

Point NO.	Section (M)	Discharge (M*3/S)	Water Level EL.(M)	Water Depth H(M)	Difference of Water Level (M)	Velocity V(M/S)	Cross-Sectional Area A(M*2)	Hydraulic Mean Depth R(M)	Roughness Coefficient N	Energy Gradient E(M)
0	.00	74.900	1.300	3.800	.000	.908	82.460	2.776	.0300	1.342
0 +500.00	500.00	74.900	1.399	3.621	.099	.965	77.604	2.671	.0300	1.446
1	500.00	74.900	1.516	3.461	.117	1.021	73.336	2.575	.0300	1.569
1	.00	74.900	1.516	3.461	.000	1.021	73.336	2.575	.0300	1.569
1 +500.00	500.00	74.900	1.654	3.321	.138	1.075	69.677	2.491	.0300	1.713
2	500.00	74.900	1.814	3.202	.159	1.124	66.622	2.419	.0300	1.878
2	.00	74.900	1.814	3.202	.000	1.124	66.622	2.419	.0300	1.878
2 +500.00	500.00	74.900	1.994	3.105	.181	1.168	64.146	2.359	.0300	2.064
2 +600.00	100.00	74.900	2.033	3.088	.039	1.176	63.716	2.348	.0300	2.103
2 +600.00	.00	62.100	2.029	3.084	-.004	1.211	51.277	2.218	.0300	2.103
3	400.00	62.100	2.213	3.046	.184	1.230	50.475	2.196	.0300	2.290
3	.00	62.100	2.213	3.046	.000	1.230	50.475	2.196	.0300	2.290
3 +500.00	500.00	62.100	2.455	3.010	.242	1.249	49.715	2.175	.0300	2.534
3 +750.00	250.00	62.100	2.579	2.996	.125	1.257	49.414	2.167	.0300	2.660
3 +750.00	.00	44.000	2.494	2.910	-.086	1.807	24.344	1.680	.0300	2.660
4	250.00	44.000	2.849	3.087	.356	1.651	26.649	1.761	.0300	2.988
4	.00	44.000	2.849	3.087	.000	1.651	26.649	1.761	.0300	2.988
4 +500.00	500.00	44.000	3.388	3.269	.538	1.512	29.099	1.843	.0300	3.504
4 +800.00	300.00	44.000	3.657	3.324	.270	1.473	29.867	1.869	.0300	3.768
4 +800.00	.00	42.400	3.666	3.332	.009	1.414	29.988	1.872	.0300	3.768

(2) Main Drainage Canal No.2

Point NO.	Section (M)	Discharge (M*3/S)	Water Level EL.(M)	Water Depth H(M)	Difference of Water Level (M)	Velocity V(M/S)	Cross-Sectional Area A(M*2)	Hydraulic Mean Depth R(M)	Roughness Coefficient N	Energy Gradient E(M)
0	.00	42.100	1.300	3.800	.000	.754	55.860	2.461	.0300	1.329
0 +500.00	500.00	42.100	1.385	3.468	.085	.855	49.256	2.290	.0300	1.422
1	500.00	42.100	1.505	3.172	.120	.965	43.638	2.135	.0300	1.553
1	.00	42.100	1.505	3.172	.000	.965	43.638	2.135	.0300	1.553
1 +500.00	500.00	42.100	1.673	2.923	.168	1.076	39.122	2.002	.0300	1.732
1 +850.00	350.00	42.100	1.824	2.782	.151	1.149	36.654	1.926	.0300	1.891
1 +850.00	.00	40.100	1.813	2.771	-.011	1.241	32.302	1.847	.0300	1.891
2	150.00	40.100	1.905	2.738	.092	1.262	31.779	1.829	.0300	1.986
2	.00	40.100	1.905	2.738	.000	1.262	31.779	1.829	.0300	1.986
2 +500.00	500.00	40.100	2.237	2.654	.333	1.316	30.468	1.785	.0300	2.326
2 +800.00	300.00	40.100	2.455	2.622	.218	1.338	29.978	1.768	.0300	2.547

Table F.4.4.(2) Hydraulic Analysis of Main Drainage Canal by Uniform Flow

(3) Main Drainage Canal No.3

Point	Section	Discharge	Water Level	Water Depth	Difference of Water Level	Velocity	Cross-Sectional Area	Hydraulic Mean Depth	Roughness Coefficient	Energy Gradient
NO.	(M)	(M*3/S)	EL.(M)	H(M)	(M)	V(M/S)	A(M*2)	R(M)	N	E(M)
0	.00	35.600	1.589	3.689	.000	.770	46.236	2.278	.0300	1.619
0 +500.00	500.00	35.600	1.682	3.532	.093	.819	43.444	2.201	.0300	1.717
1	500.00	35.600	1.793	3.393	.111	.868	41.018	2.133	.0300	1.831
1	.00	35.600	1.793	3.393	.000	.868	41.018	2.133	.0300	1.831
1 +500.00	500.00	35.600	1.922	3.272	.129	.914	38.960	2.073	.0300	1.964
2	500.00	35.600	2.070	3.170	.148	.956	37.257	2.022	.0300	2.116
2	.00	35.600	2.070	3.170	.000	.956	37.257	2.022	.0300	2.116
2 +100.00	100.00	35.600	2.102	3.152	.032	.963	36.963	2.013	.0300	2.149
2 +100.00	.00	23.100	2.126	3.176	.024	.676	34.183	1.959	.0300	2.149
2 +600.00	500.00	23.100	2.216	3.016	.090	.728	31.734	1.881	.0300	2.243
3	400.00	23.100	2.301	2.901	.086	.769	30.035	1.825	.0300	2.332
3	.00	23.100	2.301	2.901	.000	.769	30.035	1.825	.0300	2.332
3 +500.00	500.00	23.100	2.428	2.778	.127	.818	28.247	1.764	.0300	2.462
3 +750.00	250.00	23.100	2.500	2.725	.071	.840	27.484	1.737	.0300	2.536
3 +750.00	.00	15.900	2.501	2.726	.001	.823	19.326	1.506	.0300	2.536

Table F.4.4.(3) Hydraulic Analysis of Main Drainage Canal by Uniform Flow

(4) Main Drainage Canal No.4

Point NO.	Section (M)	Discharge (M*3/S)	Water Level EL.(M)	Water Depth H(M)	Difference of Water Level (M)	Velocity V(M/S)	Cross-Sectional Area A(M*2)	Hydraulic Mean Depth R(M)	Roughness Coefficient N	Energy Gradient E(M)
0	.00	85.800	1.472	3.672	.000	.658	130.385	3.015	.0300	1.494
0 +200.00	200.00	85.800	1.490	3.590	.018	.675	127.028	2.958	.0300	1.513
0 +200.00	.00	85.800	1.490	3.590	.000	.675	127.028	2.958	.0300	1.513
0 +700.00	500.00	85.800	1.540	3.390	.050	.721	118.948	2.817	.0300	1.567
1	300.00	85.800	1.576	3.276	.035	.750	114.360	2.735	.0300	1.604
1	.00	85.800	1.576	3.276	.000	.750	114.360	2.735	.0300	1.604
1 +100.00	100.00	85.800	1.589	3.239	.013	.760	112.899	2.709	.0300	1.618
1 +100.00	.00	49.800	1.543	3.193	-.046	1.220	40.829	2.093	.0300	1.618
1 +600.00	500.00	49.800	1.793	3.193	.250	1.220	40.831	2.093	.0300	1.869
2	400.00	49.800	1.993	3.193	.200	1.220	40.832	2.093	.0300	2.069
2	.00	49.800	1.993	3.193	.000	1.220	40.832	2.093	.0300	2.069
2 +500.00	500.00	49.800	2.243	3.193	.250	1.220	40.833	2.093	.0300	2.319
2 +600.00	100.00	49.800	2.293	3.193	.050	1.220	40.833	2.093	.0300	2.369
2 +600.00	.00	33.000	2.313	3.213	.020	1.046	31.547	1.902	.0300	2.369
3	400.00	33.000	2.482	3.182	.169	1.061	31.094	1.888	.0300	2.539
3	.00	33.000	2.482	3.182	.000	1.061	31.094	1.888	.0300	2.539
3 +500.00	500.00	33.000	2.702	3.152	.220	1.076	30.660	1.874	.0300	2.761
4	500.00	33.000	2.930	3.130	.228	1.088	30.341	1.863	.0300	2.990
4	.00	33.000	2.930	3.130	.000	1.088	30.341	1.863	.0300	2.990
4 +300.00	300.00	33.000	3.069	3.119	.139	1.093	30.188	1.858	.0300	3.130
4 +300.00	.00	29.800	3.081	3.131	.012	.982	30.357	1.864	.0300	3.130
4 +800.00	500.00	29.800	3.275	3.075	.194	1.008	29.554	1.837	.0300	3.327
5	200.00	29.800	3.356	3.056	.081	1.017	29.289	1.828	.0300	3.409
5	.00	29.800	3.356	3.056	.000	1.017	29.289	1.828	.0300	3.409
5 +500.00	500.00	29.800	3.568	3.018	.212	1.036	28.754	1.810	.0300	3.623
6	500.00	29.800	3.790	2.990	.222	1.051	28.361	1.797	.0300	3.846
6	.00	29.800	3.790	2.990	.000	1.051	28.361	1.797	.0300	3.846
6 +450.00	450.00	29.800	4.017	2.767	.227	1.177	25.318	1.691	.0300	4.088

Table F.4.4.(4) Hydraulic Analysis of Main Drainage Canal by Uniform Flow

(5) Main Drainage Canal No.5

Point NO.	Section (M)	Discharge (M*3/S)	Water Level EL.(M)	Water Depth H(M)	Difference of Water Level (M)	Velocity V(M/S)	Cross-Sectional Area A(M*2)	Hydraulic Mean Depth R(M)	Roughness Coefficient N	Energy Gradient E(M)
0	.00	303.000	1.300	3.900	.000	1.277	237.315	3.436	.0300	1.383
0 +500.00	500.00	303.000	1.443	3.834	.143	1.301	232.932	3.384	.0300	1.529
0 +600.00	100.00	303.000	1.472	3.822	.030	1.305	232.154	3.375	.0300	1.559
0 +600.00	.00	208.000	1.488	3.838	.015	1.184	175.605	3.262	.0300	1.559
1	400.00	208.000	1.592	3.776	.104	1.206	172.404	3.216	.0300	1.666
1	.00	208.000	1.592	3.776	.000	1.206	172.404	3.216	.0300	1.666
1 +500.00	500.00	208.000	1.732	3.707	.139	1.232	168.871	3.164	.0300	1.809
2	500.00	208.000	1.880	3.647	.148	1.254	165.807	3.120	.0300	1.960
2	.00	208.000	1.880	3.647	.000	1.254	165.807	3.120	.0300	1.960
2 +500.00	500.00	208.000	2.037	3.595	.157	1.275	163.178	3.081	.0300	2.119
2 +600.00	100.00	208.000	2.069	3.586	.033	1.278	162.718	3.074	.0300	2.152
2 +600.00	.00	193.100	2.066	3.583	-.003	1.303	148.233	3.030	.0300	2.153
3	400.00	193.100	2.205	3.555	.139	1.314	146.955	3.010	.0300	2.293

Table F.4.5. Length of Proposed Main Drainage Canal

Unit : km

Name of Canal	Existing	Rehabilitation	New Construction	Total
No.1 Main Drainage Canal	0.65	3.75	7.35	11.75
No.2 Main Drainage Canal	3.36	8.35		11.70
No.3 Main Drainage Canal	1.30	3.75	4.45	9.50
No.4 Main Drainage Canal		6.45	5.15	11.60
No.5 Main Drainage Canal	1.15	2.00	11.25	14.40
No.6 Main Drainage Canal			2.65	2.65
No.7 Main Drainage Canal		1.65	0.25	1.90
No.8 Main Drainage Canal			1.00	1.00
Total	6.45	25.95	32.10	64.50

Table F.4.6 Rehabilitation of Excisting Major Crossing Structures

Name of Canal	Point	Works	Design Discharge	Capacity	Rehabilitation
			m^3/s	m^3/s	
No.1 Main Drainage Canal	No.11+0.10	Iron Bridge	11.4	11.5(h=1.5m)	Needless
No.1 Main Drainage Canal	No.11+0.20	Road Bridge	11.4	12.4(h=1.7m)	Needless
No.2 Main Drainage Canal	No.11+0.15	Iron Bridge	9.0	10.2(h=1.2m)	Needless
No.2 Main Drainage Canal	No.11+0.15	Road Bridge	9.0	10.0(h=1.1m)	Needless
No.6 Main Drainage Canal	No. 2+0.65	Road Bridge	70.0	57.1(h=3.0m)	To construct the bridge newly
No.6 Main Drainage Canal	No. 2+0.65	Iron Bridge	70.0	51.4(h=2.7m)	To excavate the sectimentation under the iron bridge, by canal bed width of 13.0m and side slope of 1:1.5 (capacity is 77.7m^3/s).

Note: 1. The capacity was estimated by Manning Formula. The detailes are shown in Table F.4.7.
2. The parentheses show the water depth.

Table F.4.7(1) Capcity of Excisting Major Crossing Structures

[Iron Bridge on the Main Drainage Canal No.1 (Matina)]

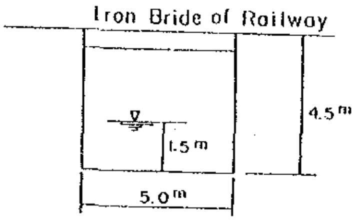
(1) Point

No.11 + 0.10

(2) Design Discharge

$Q = 11.4\ m^3/s$

(3) Cross-Section



(4) Capacity

$I = 1/420$
$A = 7.5\ m^2$
$P = 8.0\ m$
$R = A/P = 0.94 \qquad R^{2/3} = 0.960$
$n = 0.030$
$V = 1/n \times R^{2/3} \times I^{1/2} = 1.53\ m/s$
$Q = V \times A = 11.5\ m^3/s > 11.4\ m^3/s$

The rehabilitation does not need, because of enough capacity.

[Road Bridge on the Main Drainage Canal No.1 (Matina)]

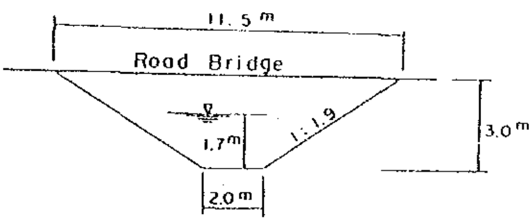
(1) Point

No.11 + 0.20

(2) Design Discharge

$Q = 11.4\ m^3/s$

(3) Cross-Section



(4) Capacity

$I = 1/420$
$A = 8.0\ m^2$
$P = 8.4\ m$
$R = A/P = 0.95 \qquad R^{2/3} = 0.966$
$n = 0.030$
$V = 1/n \times R^{2/3} \times I^{1/2} = 1.55\ m/s$
$Q = V \times A = 12.4\ m^3/s > 11.4\ m^3/s$

The rehabilitation does not need, because of enough capacity.

Table F.4.7(2) Capacity of Excisting Major Crossing Structures

[Iron Bridge on the Main Drainage Canal No.2 (Bataan)]

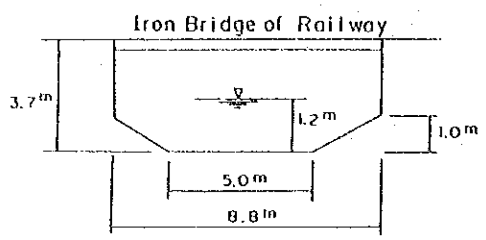
(1) Point

No.11 + 0.15

(2) Design Discharge

$Q = 9.0\ m^3/s$

(3) Cross-Section



(4) Capacity

$I = 1/700$
$A = 8.7\ m^2$
$P = 9.7\ m$
$R = A/P = 0.90 \qquad R^{2/3} = 0.932$
$n = 0.030$
$V = 1/n \times R^{2/3} \times I^{1/2} = 1.17\ m/s$
$Q = V \times A = 10.2\ m^3/s > 9.0\ m^3/s$

The rehabilitation does not need, because of enough capacity.

[Road Bridge on the Main Drainage Canal No.2 (Bataan)]

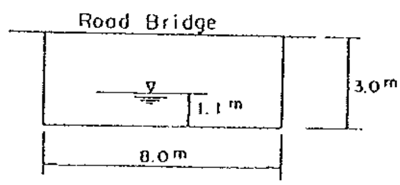
(1) Point

No.11 + 0.15

(2) Design Discharge

$Q = 9.0\ m^3/s$

(3) Cross-Section



(4) Capacity

$I = 1/700$
$A = 8.8\ m^2$
$P = 10.2\ m$
$R = A/P = 0.86 \qquad R^{2/3} = 0.904$
$n = 0.030$
$V = 1/n \times R^{2/3} \times I^{1/2} = 1.14\ m/s$
$Q = V \times A = 10.0\ m^3/s > 9.0\ m^3/s$

The rehabilitation does not need, because of enough capacity.

Table F.4.7(3) Capacity of Excisting Major Crossing Structures

[Road Bridge on the Main Drainage Canal No.6 (Veintiseis)]

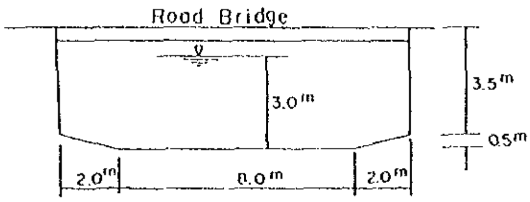
(1) Point

No.2 + 0.65

(2) Design Discharge

$Q = 70.0\ m^3/s$

(3) Cross-Section



(4) Capacity

$I = 1/1000$
$A = 35.0\ m^2$
$P = 18.1\ m$
$R = A/P = 1.93 \qquad R^{2/3} = 1.550$
$n = 0.030$
$V = 1/n \times R^{2/3} \times I^{1/2} = 1.63\ m/s$
$Q = V \times A = 57.1\ m^3/s > 70.0\ m^3/s$

The rehabilitation does not need, because of enough capacity.

[Iron Bridge on the Main Drainage Canal No.6 (Veintiseis)]

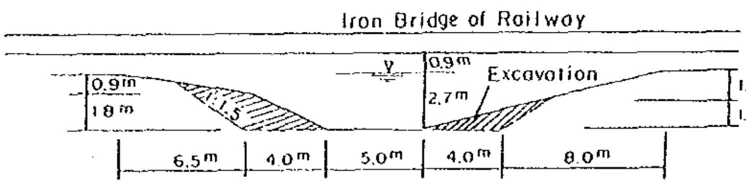
(1) Point

No.2 + 0.65

(2) Design Discharge

$Q = 70.0\ m^3/s$

(3) Cross-Section



(4) Capacity

$I = 1/1000$
$A = 39.2\ m^2$
$P = 28.3\ m$
$R = A/P = 1.39 \qquad R^{2/3} = 1.246$
$n = 0.030$
$V = 1/n \times R^{2/3} \times I^{1/2} = 1.31\ m/s$
$Q = V \times A = 51.4\ m^3/s > 70.0\ m^3/s$

The bed of this section needs to be excavated for keeping enough capacity.

Table F.4.8.(1) Design Discharge of Proposed Secondary Canal

Name of Canal	Section	Length of Canal	Watershed	Unit Discharge	Design Discharge	Remarks
		km	km^2	$m^3/s/km^2$	m^3/s	
No.1-1	No.0 - No.1	1.00	2.90	3.575	10.4	
	No.1 - No.2	1.00	1.60	3.575	5.7	
	No.2 - No.2+0.85	0.85	0.80	3.575	2.9	
	Sub-total	2.85				
No.1-2	No.0 - No.1	1.00	8.90	3.575	31.8	
	No.1 - No.2	1.00	4.80	3.575	17.2	
	No.2 - No.2+0.90	0.90	4.20	3.575	15.0	
	Sub-total	2.90				
No.1-3	No.0 - No.1	1.00	2.05	3.575	7.3	
	No.1 - No.1+0.70	0.70	1.60	3.575	5.7	
	Sub-total	1.70				
No.1-4	No.0 - No.1	1.00	1.20	3.575	4.3	
No.1-5	No.0 - No.1	1.00	1.20	3.575	4.3	
No.1-6	No.0 - No.1	1.00	1.20	3.575	4.3	
No.1-7	No.0 - No.1	1.00	1.20	3.575	4.3	

Table F.4.8.(2) Design Discharge of Proposed Secondary Canal

Name of Canal	Section	Length of Canal	Watershed	Unit Discharge	Design Discharge	Remarks
		km	km^2	$m^3/s/km^2$	m^3/s	
No.1-8	No.0 - No.1	1.00	7.20	3.575	25.7	
	No.1 - No.1+0.40	0.40	5.80	3.575	20.7	
	No.1+0.40 - No.3+0.50	2.10	1.85	3.575	6.6	
	Sub-total	3.50				
No.1-9	No.0 - No.1	1.00	1.20	3.575	4.3	
No.1-10	No.0 - No.1	1.00	1.00	3.575	3.6	
No.1-11	No.0 - No.0+0.90	0.90	0.90	3.575	3.2	
No.1-12	No.0 - No.0+0.70	0.70	5.45	3.575	19.5	
	No.0+0.70 - No.2	1.30	1.65	3.575	5.9	
	No.2 - No.3+0.30	1.30	1.15	3.575	4.1	
	Sub-total	3.30				
No.1-13	No.0 - No.1	1.00	2.80	3.575	10.0	
	No.1 - No.1+0.90	0.90	1.40	3.575	5.0	
	Sub-total	1.90				
	Total	23.05				

Table F.4.8.(3) Design Discharge of Proposed Secondary Canal

Name of Canal	Section	Length of Canal	Watershed	Unit Discharge	Design Discharge	Remarks
		km	km^2	$m^3/s/km^2$	m^3/s	
No.2-1	No.0 - No.1	1.00	1.10	3.575	3.9	
No.2-2	No.0 - No.1	1.00	1.20	3.575	4.3	
No.2-3	No.0 - No.1	1.00	1.20	3.575	4.3	
No.2-4	No.0 - No.1	1.00	1.20	3.575	4.3	
No.2-5	No.0 - No.1	1.00	1.25	3.575	4.5	
No.2-6	No.0 - No.1	1.00	6.85	3.575	24.5	
	No.1 - No.2	1.00	6.55	3.575	23.4	
	No.2 - No.3	1.00	6.25	3.575	22.3	
	No.3 - No.4	1.00	5.90	3.575	21.1	
	No.4 - No.5+0.25	1.25	5.75	3.575	20.6	
	Sub-Total	5.25				
No.2-7	No.0 - No.0+0.70	0.70	1.95	3.575	7.0	
	No.0+0.70 - No.1+0.35	0.65	1.05	3.575	3.8	
	Sub-total	1.35				
	Total	11.60				

Table F.4.8.(4) Design Discharge of Proposed Secondary Canal

Name of Canal	Section	Length of Canal	Watershed	Unit Discharge	Design Discharge	Remarks
		km	km^2	$m^3/s/km^2$	m^3/s	
No.3-1	No.0 - No.1	1.00	3.35	3.575	12.0	
	No.1 - No.2	1.00	1.80	3.575	6.4	
	No.2 - No.2+0.45	0.45	0.50	3.575	1.8	
	Sub-total	2.45				
No.3-2	No.0 - No.1	1.00	4.25	3.575	15.2	
	No.1 - No.2	1.00	3.40	3.575	12.2	
	No.2 - No.2+0.70	0.70	1.90	3.575	6.8	
	Sub-total	2.70				
No.3-3	No.0 - No.1	1.00	3.05	3.575	10.9	
	No.1 - No.2	1.00	2.25	3.575	8.0	
	No.2 - No.2+0.30	0.30	0.80	3.575	2.9	
	Sub-total	2.30				
No.3-4	No.0 - No.1	1.00	2.80	3.575	10.0	
	No.1 - No.2+0.15	1.15	2.20	3.575	7.9	
	Sub-total	2.15				
	Total	9.60				

Table F.4.8(5) Design Discharge of Proposed Secondary Canal

Name of Canal	Section	Length of Canal	Watershed	Unit Discharge	Design Discharge	Remarks
		km	km^2	$m^3/s/km^2$	m^3/s	
No.4-1	No.0 - No.1	1.00	7.55	3.575	27.0	
	No.1 - No.2	1.00	6.50	3.575	23.2	
	No.2 - No.3	1.00	4.10	3.575	14.7	
	No.3 - No.3+0.70	0.70	2.50	3.575	8.9	
	Sub-total	3.70				
No.4-2	No.0 - No.0+0.75	0.75	5.45	3.575	19.5	
	No.0+0.75 - No.1	0.25	2.75	3.575	9.8	
	No.1 - No.2	1.00	2.50	3.575	8.9	
	No.2 - No.2+0.70	0.70	1.75	3.575	6.3	
	Sub-total	2.70				
No.4-3	No.0 - No.1	1.00	2.20	3.575	7.9	
	No.1 - No.1+0.90	0.90	0.50	3.575	1.8	
	Sub-total	1.90				
No.4-4	No.0 - No.1	1.00	1.55	3.575	5.5	
	No.1 - No.1+0.25	0.25	0.40	3.575	1.4	
	Sub-total	1.25				
No.4-5	No.0 - No.0+0.60	0.60	1.20	3.575	4.3	
No.4-6	No.0 - No.1	1.00	1.75	3.575	6.3	

Table F.4.8.(6) Design Discharge of Proposed Secondary Canal

Name of Canal	Section	Length of Canal	Watershed	Unit Discharge	Design Discharge	Remarks
		km	km^2	$m^3/s/km^2$	m^3/s	
No.4-7	No.0 - No.1	1.00	1.50	3.575	5.4	
	No.1 - No.2	1.00	0.40	3.575	1.4	
	Sub-total	2.00				
No.4-8	No.0 - No.1	1.00	1.55	3.575	5.5	
	No.1 - No.1+0.50	0.50	0.60	3.575	2.1	
	Sub-total	1.50				
No.4-9	No.0 - No.1	1.00	1.55	3.575	5.5	
	No.1 - No.1+0.45	0.45	0.60	3.575	2.1	
	Sub-total	1.45				
No.4-10	No.0 - No.0+0.50	0.50	0.65	3.575	2.3	
No.4-11	No.0 - No.1	1.00	1.60	3.575	5.7	
	No.1 - No.1+0.45	0.45	0.50	3.575	1.8	
	Sub-total	1.45				
No.4-12	No.0 - No.0+0.65	0.65	0.45	3.575	1.6	
	Total	18.70				

Table F.4.8.(7) Design Discharge of Proposed Secondary Canal

Name of Canal	Section	Length of Canal	Watershed	Unit Discharge	Design Discharge	Remarks
		km	km^2	$m^3/s/km^2$	m^3/s	
No.5-1	No.0 - No.1	1.00	6.30	3.575	22.5	
	No.1 - No.2	1.00	5.80	3.575	20.7	
	No.2 - No.3	1.00	5.20	3.575	18.6	
	No.3 - No.4	1.00	2.85	3.575	10.2	
	No.4 - No.5	1.00	2.20	3.575	7.9	
	No.5 - No.6	1.00	1.70	3.575	6.1	
	No.6 - No.7+0.40	1.40	0.60	3.575	2.1	
	Sub-total	7.40				
No.9-1	No.0 - No.1	1.00	2.70	3.575	9.7	
	No.1 - No.2	1.00	2.50	3.575	8.9	
	No.2 - No.3+0.35	1.35	1.30	3.575	4.7	
	Sub-total	3.35				
No.10-1	No.0 - No.1+0.15	1.15	4.50	3.575	16.1	
No.11-1	No.0 - No.1	1.00	1.00	3.575	3.6	
	No.1 - No.1+0.60	0.60	0.50	3.575	1.8	
	Sub-total	1.60				
	Grand Total	76.45				

Table F.4.9.(1) Length and Capacity of Proposed Secondary Canal

Name of Canal	Section	Length				Capacity						
		Existing	Rehabilitation	New Construction	Total	Design Discharge	Slope	Width of Bed	Canal Depth	Water Depth	Velocity	Capacity
		km	km	km	km	m^3/s	I/s	m	m	m	m/s	m^3/s
No.1-1	No.0 - No.2+0.05	2.05			2.05	10.4 - 2.9	1,000	to use the Existing		1.7	1.12	10.9
	No.2 0.05 - No.2+0.85			0.80	0.80	2.9	1,000	1.0	2.0	1.7	0.90	4.1
	Sub-total	2.05		0.80	2.85							
No.1-2	No.0 - No.1		1.00		1.00	31.8	1,000	3.0	3.5	3.0	1.45	32.8
	No.1 - No.2		1.00		1.00	17.2	1,000	2.5	3.5	3.0	1.43	30.8
	No.2 - No.2+0.90		0.90		0.90	15.0	1,000	2.5	3.5	3.0	1.43	30.0
	Sub-total		2.90		2.90							
No.1-3	No.0 - No.1+0.70	1.70			1.70	7.3 - 5.3	1,000	to use the Existing		3.0	1.21	18.5
	Sub-total	1.70			1.70							
No.1-4	No.0 - No.1			1.00	1.00	4.3	1,000	1.5	2.0	1.7	0.96	5.2
	Sub-total			1.00	1.00							
No.1-5	No.0 - No.1			1.00	1.00	4.3	1,000	1.5	2.0	1.7	0.96	5.2
	Sub-total			1.00	1.00							
No.1-6	No.0 - No.1			1.00	1.00	4.3	1,000	1.5	2.0	1.7	0.96	5.2
	Sub-total			1.00	1.00							
No.1-7	No.0 - No.1			1.00	1.00	4.3	1,000	1.5	2.0	1.7	0.96	5.2
	Sub-total			1.00	1.00							

Table F.4.9.(2) Length and Capacity of Proposed Secondary Canal

Name of Canal	Section	Length: Existing	Length: Rehabilitation	Length: New Construction	Length: Total	Capacity: Design Discharge	Capacity: Slope	Capacity: Width of Bed	Capacity: Canal Depth	Capacity: Water Depth	Capacity: Velocity	Capacity: Capacity
		km	km	km	km	m^3/s	l/s	m	m	m	m/s	m^3/s
No.1-8	No.0 - No.1+0.40	1.40			1.40	25.7 - 20.7	1,000	to use the Existing		3.5	1.69	60.6
	No.1+0.40 - No.3+0.50			2.10	2.10	6.6	1,000	2.5	2.0	1.7	1.04	7.4
	Sub-total	1.40		2.10	3.50							
No.1-9	No.0 - No.1			1.00	1.00	4.3	1,000	1.5	2.0	1.7	0.96	5.2
	Sub-total			1.00	1.00							
No.1-10	No.0 - No.1			1.00	1.00	3.6	1,000	1.0	2.0	1.7	0.90	4.1
	Sub-total			1.00	1.00							
No.1-11	No.0 - No.0+0.90			0.90	0.90	3.2	1,000	1.0	2.0	1.7	0.90	4.1
	Sub-total			0.90	0.90							
No.1-12	No.0 - No.0+0.70			0.70	0.70	19.5	1,000	2.5	3.0	2.5	1.29	20.2
	No.0+0.70 - No.2			1.30	1.30	5.9	1,000	2.0	2.0	1.7	1.00	6.3
	No.2 - No.3+0.30			1.30	1.30	4.1	1,000	1.0	2.0	1.7	0.90	4.1
	Sub-total			3.30	3.30							
No.1-13	No.0 - No.1			1.00	1.00	10.0	2,000	4.0	2.0	1.7	1.12	10.9
	No.1 - No.1+0.90			0.90	0.90	5.0	2,000	1.5	2.0	1.7	0.96	5.2
	Sub-total			1.90	1.90							
	Total	5.15	2.90	15.00	23.05							

Table F.4.9.(3) Length and Capacity of Proposed Secondary Canal

Name of Canal	Section	Length				Capacity						
		Existing	Rehabilitation	New Construction	Total	Design Discharge	Slope	Width of Bed	Canal Depth	Water Depth	Velocity	Capacity
		km	km	km	km	m^3/s	l/s	m	m	m	m/s	m^3/s
No.2-1	No.0 - No.1			1.00	1.00	3.9	2,000	2.0	2.0	1.7	0.71	4.4
	Sub-total			1.00	1.00							
No.2-2	No.0 - No.1			1.00	1.00	4.3	1,000	1.5	2.0	1.7	0.96	5.2
	Sub-total			1.00	1.00							
No.2-3	No.0 - No.1			1.00	1.00	4.3	1,000	1.5	2.0	1.7	0.96	5.2
	Sub-total			1.00	1.00							
No.2-4	No.0 - No.1			1.00	1.00	4.3	1,000	1.5	2.0	1.7	0.96	5.2
	Sub-total			1.00	1.00							
No.2-5	No.0 - No.1			1.00	1.00	4.5	1,000	1.5	2.0	1.7	0.96	5.2
	Sub-total			1.00	1.00							
No.2-6	No.0 - No.1		1.00		1.00	24.5	2,600	7.0	3.0	2.5	0.92	24.8
	No.1 - No.4		3.00		3.00	23.4 - 21.1	2,600	6.5	3.0	2.5	0.91	23.4
	No.4 - No.5+0.25		1.25		1.25	20.6	1,500	4.0	3.0	2.5	1.12	21.7
	Sub-total		5.25		5.25							
No.2-7	No.0 - No.1	1.00			1.00	7.0 - 3.8	700	to use the Existing		2.0	1.37	7.5
	No.1 - No.1+0.35			0.35	0.35	3.8	700	1.0	2.0	1.7	1.08	4.9
	Sub-total	1.00		0.35	1.35							
	Total	1.00	5.25	5.35	11.60							

Table F.4.9.(4) Length and Capacity of Proposed Secondary Canal

Name of Canal	Section	Length				Capacity						
		Existing	Rehabilitation	New Construction	Total	Design Discharge	Slope	Width of Bed	Canal Depth	Water Depth	Velocity	Capacity
		km	km	km	km	m^3/s	l/s	m	m	m	m/s	m^3/s
No.3-1	No.0 - No.1			1.00	1.00	12.0	800	4.0	2.0	1.7	1.26	12.2
	No.1 - No.2			1.00	1.00	6.4	800	2.0	2.0	1.7	1.12	7.0
	No.2 - No.2+0.45			0.45	0.45	1.8	800	1.0	2.0	1.7	1.01	4.6
	Sub-total			2.45	2.45							
No.3-2	No.0 - No.1		1.00		1.00	15.2	2,000	8.5	2.0	1.7	0.89	15.4
	No.1 - No.2		1.00		1.00	12.2	2,000	7.0	2.0	1.7	0.87	12.8
	No.2 - No.2+0.70		0.70		0.70	6.8	2,000	4.0	2.0	1.7	0.79	7.7
	Sub-total		2.70		2.70							
No.3-3	No.0 - No.1		1.00		1.00	10.9	1,000	4.0	2.0	1.7	1.12	10.9
	No.1 - No.2		1.00		1.00	8.0	1,000	3.0	2.0	1.7	1.07	8.6
	No.2 - No.2+0.05		0.05		0.05	2.9	1,000	1.0	2.0	1.7	0.90	4.1
	No.2+0.05 - No.2+0.30			0.25	0.25	2.9	1,000	1.0	2.0	1.7	0.90	4.1
	Sub-total		2.05	0.25	2.30							
No.3-4	No.0 - No.0+0.75		0.75		0.75	10.0	1,000	4.0	2.0	1.7	1.12	10.9
	No.0+0.75 - No.1			0.25	0.25	10.0	1,000	4.0	2.0	1.7	1.12	10.9
	No.1 - No.1+0.45			0.45	0.45	7.9	1,000	3.0	2.0	1.7	1.07	8.6
	No.1+0.45 - No.2+0.15		0.70		0.70	7.9	1,000	3.0	2.0	1.7	1.07	8.6
	Sub-total		1.45	0.70	2.15							
	Total		6.20	3.40	9.60							

Table F.4.9.(5) Length and Capacity of Proposed Secondary Canal

Name of Canal	Section	Length				Capacity						
		Existing	Rehabilitation	New Construction	Total	Design Discharge	Slope	Width of Bed	Canal Depth	Water Depth	Velocity	Capacity
		km	km	km	km	m^3/s	I/s	m	m	m	m/s	m^3/s
No.4-1	No.0 - No.1		1.00		1.00	27.0	1,000	4.5	3.0	2.5	1.40	28.8
	No.1 - No.1+0.65		0.65		0.65	23.2	1,000	3.5	3.0	2.5	1.35	24.5
	No.1+0.65 - No.2			0.35	0.35	23.2	1,000	3.5	3.0	2.5	1.35	24.5
	No.2 - No.3			1.00	1.00	14.7	1,000	1.5	3.0	2.5	1.22	16.0
	No.3 - No.3+0.70			1.70	0.70	8.9	1,000	1.5	2.5	2.0	1.08	9.7
	Sub-total		1.65	2.05	3.70							
No.4-2	No.0 - No.0+0.75		0.75		0.75	19.5	1,000	2.5	3.0	2.5	1.29	20.2
	No.0+0.75 - No.2+0.70	1.95			1.95	9.8 - 6.3	1,000	to use the Existing		2.7	1.21	10.6
	Sub-total	1.95	0.75		2.70							
No.4-3	No.0 - No.1			1.00	1.00	7.9	2,000	4.5	2.0	1.7	0.81	8.5
	No.1 - No.1+0.9			0.90	0.90	1.8	2,000	1.0	2.0	1.7	0.64	2.9
	Sub-total			1.90	1.90							
No.4-4	No.0 - No.1	1.00			1.00	5.5	1,000	to use the Existing		2.0	1.23	16.3
	No.1 - No.1+0.25			0.25	0.25	1.4	1,000	1.0	2.0	1.7	0.90	4.1
	Sub-total	1.00		0.25	1.25							
No.4-5	No.0 - No.0+0.60		0.60		0.60	4.3	1,000	1.5	2.0	1.7	0.96	5.2
	Sub-total		0.60		0.60							
No.4-6	No.0 - No.0+0.50		0.50		0.50	6.3	1,000	2.0	2.0	1.7	1.00	6.3
	No.0+0.50 - No.1			0.50	0.50	6.3	1,000	2.0	2.0	1.7	1.00	6.3
	Sub-total		0.50	0.50	1.00							

Table F.4.9.(6) Length and Capacity of Proposed Secondary Canal

Name of Canal	Section	Length				Capacity						
		Existing	Rehabilitation	New Construction	Total	Design Discharge	Slope	Width of Bed	Canal Depth	Water Depth	Velocity	Capacity
		km	km	km	km	m^3/s	I/s	m	m	m	m/s	m^3/s
No.4-7	No.0 - No.0+0.55		0.55		0.55	5.4	2,000	2.0	2.0	1.7	1.00	6.3
	No.0+0.55 - No.1			0.45	0.45	5.4	2,000	2.0	2.0	1.7	1.00	6.3
	No.1 - No.2			1.00	1.00	1.4	2,000	1.0	2.0	1.7	0.90	4.1
	Sub-total		0.55	1.45	2.00							
No.4-8	No.0 - No.1			1.00	1.00	5.5	1,000	2.0	2.0	1.7	1.00	6.3
	No.1 - No.1+0.50			0.50	0.50	2.1	1,000	1.0	2.0	1.7	0.90	4.1
	Sub-total			1.50	1.50							
No.4-9	No.0 - No.1			1.00	1.00	5.5	1,000	2.0	2.0	1.7	1.00	6.3
	No.1 - No.1+0.45			0.45	0.45	2.1	1,000	1.0	2.0	1.7	0.90	4.1
	Sub-total			1.45	1.45							
No.4-10	No.0 - No.0+0.50			0.50	0.50	2.3	1,000	1.0	2.0	1.7	0.90	4.1
	Sub-total			0.50	0.50							
No.4-11	No.0 - No.1			1.00	1.00	5.7	500	1.0	2.0	1.7	1.27	5.8
	No.1 - No.1+0.45			0.45	0.45	1.8	500	1.0	2.0	1.7	1.27	5.8
	Sub-total			1.45	1.45							
No.4-12	No.0 - No.0+0.65			0.65	0.65	1.6	500	1.0	2.0	1.7	1.27	5.8
	Sub-total			0.65	0.65							
	Total.	2.95	4.05	11.70	18.70							

Table F.4.9.(7) Length and Capacity of Proposed Secondary Canal

Name of Canal	Section	Length				Capacity						
		Existing	Rehabilitation	New Construction	Total	Design Discharge	Slope	Width of Bed	Canal Depth	Water Depth	Velocity	Capacity
		km	km	km	km	m^3/s	l/s	m	m	m	m/s	m^3/s
No.5-1	No.0 - No.1		1.00		1.00	22.5	1,000	3.5	3.0	2.5	1.42	25.8
	No.1 - No.2		1.00		1.00	20.7	1,000	3.0	3.0	2.5	1.32	22.3
	No.2 - No.3		1.00		1.00	18.6	1,000	2.5	3.0	2.5	1.29	20.2
	No.3 - No.6		3.00		3.00	10.2 - 6.1	1,000	2.0	2.5	2.0	1.11	11.1
	No.6 - No.6+0.35		0.35		0.35	2.1	1,000	2.0	2.0	1.7	1.00	6.0
	No.6+0.35 - No.7+0.40			1.05	1.05	2.1	1,000	1.0	2.0	1.7	0.90	4.1
	Sub-total		6.35	1.05	7.40							
No.9-1	No.0 - No.1			1.00	1.00	9.7	500	2.5	2.0	1.7	1.47	10.5
	No.1 - No.2			1.00	1.00	8.9	500	2.0	2.0	1.7	1.41	8.9
	No.2 - No.3+0.35			1.35	1.35	4.7	500	1.0	2.0	1.7	1.27	5.8
	Sub-total			3.35	3.35							
No.10-1	No.0 - No.0+0.70			0.70	0.70	16.1	800	1.5	3.0	2.5	1.37	17.9
	No.0+0.70 - No.0+0.95	0.25			0.25	16.1	800	to use the Existing		2.5	1.53	25.6
	No.0+0.95 - No.1+0.15			0.20	0.20	16.1	800	1.5	3.0	2.5	1.37	17.9
	Sub-total	0.25		0.90	1.15							
No.11-1	No.0 - No.1			1.00	1.00	3.6	500	1.0	2.0	1.7	1.27	5.8
	No.1 - No.1+0.60			0.60	0.60	1.8	500	1.0	2.0	1.7	1.27	5.8
	Sub-total			1.60	1.60							
	Grand Total	9.35	24.75	42.35	76.45							

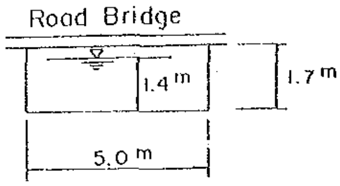
Table F.4.10.(1) Capacity of Existing Major Works Crossing Secondary Drainage Canal

[Road Bridge on the Secondary Drainage Canal No.2-6]

(1) Design Discharge

$Q = 20.6\ m^3/s$

(3) Cross-Section



(4) Capacity

$I = 1/1,500$
$A = 7.00\ m^2$ (Freeboard 0.3m)
$P = 7.80\ m$
$R = 0.90$
$R^{2/3} = 0.932$
$V = 1/n \times R^{2/3} \times I^{1/2} = 1/0.030 \times 0.932 \times (1/1,500)^{1/2}$
$= 0.80\ m/s$
$Q = V \times A = 0.80 \times 7.00 = 5.6\ m^3/s < 20.6\ m^3/s$

This facility is rehabilitated.

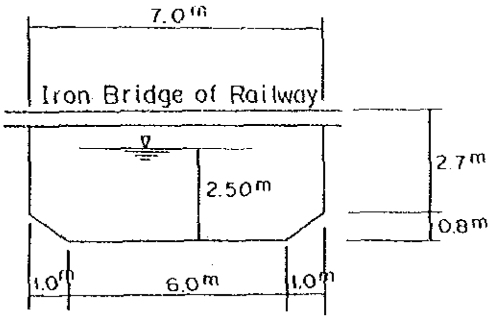
Table F.4.10.(2) Capacity of Existing Major Works Crossing Secondary Drainage Canal

[Iron Bridge on the Secondary Drainage Canal No.2-6]

(1) Design Discharge

$Q = 20.6\ m^3/s$

(3) Cross-Section



(4) Capacity

$I = 1/1,500$
$A = 19.20\ m^2$ (Freeboard 0.5m)
$P = 11.44\ m$
$R = 1.68$
$R^{2/3} = 1.413$
$V = 1/n \times R^{2/3} \times I^{1/2} = 1/0.030 \times 1.413 \times (1/1,500)^{1/2}$
$= 1.21\ m/s$
$Q = V \times A = 1.21 \times 19.20 = 23.2\ m^3/s > 20.6\ m^3/s$

The existing cross-section is utilized.

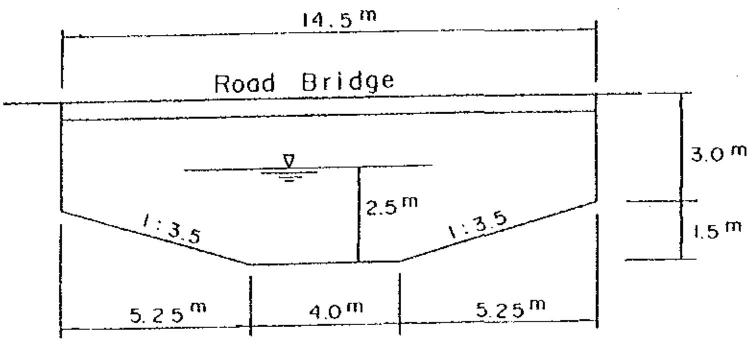
Table F.4.10.(3) Capacity of Existing Major Works Crossing Secondary Drainage Canal

[Road Bridge on the Secondary Drainage Canal No.10-1]

(1) Design Discharge

$Q = 16.1\ m^3/s$

(3) Cross-Section



(4) Capacity

$I = 1/800$
$A = 28.30\ m^2$ (Freeboard 1.50m)
$P = 16.92\ m$
$R = 1.67$
$R^{2/3} = 1.408$
$V = 1/n \times R^{2/3} \times I^{1/2} = 1/0.030 \times 1.408 \times (1/800)^{1/2}$
$= 1.66\ m/s$
$Q = V \times A = 1.66 \times 28.30 = 47.0\ m^3/s > 16.1\ m^3/s$

The existing cross-section is utilized.

Table F.5.1. Case I. Hydraulic Analysis by Uniform Flow (Rio Matina)

Point No.	Section (m)	Discharge (m3/s)	Water Level EL. (m)	Water Depth H (m)	Difference of Water Level (m)	Velocity V (m/s)	Cross-Sectional Area A (m²)	Hydraulic Mean Depth R (m)	Energy Gradient E (m)
0 -1.50	0.00	2248.000	2.000	6.733	0.000	2.423	927.930	2.781	2.299
0 -0.50	1000.00	2248.000	3.723	6.986	1.723	1.705	1318.740	3.959	3.871
0	500.00	2248.000	4.071	7.334	0.349	1.567	1434.297	4.306	4.197
0 + 0.50	500.00	2248.000	4.358	7.055	0.286	0.773	2906.476	3.180	4.388
1	500.00	2248.000	4.457	7.154	0.099	0.750	2996.722	3.279	4.486
1 + 0.50	500.00	2248.000	4.585	7.375	0.128	0.957	2349.875	2.642	4.632
2	500.00	2248.000	4.770	7.560	0.185	0.894	2513.885	2.827	4.811
2 + 0.50	500.00	2248.000	4.938	7.643	0.168	0.957	2349.417	2.789	4.985
3	500.00	2248.000	5.112	7.817	0.174	0.901	2495.560	2.962	5.154
3 + 0.65	650.00	2248.000	6.000	6.953	0.888	1.796	1251.391	2.033	6.165
4	350.00	2248.000	6.570	7.523	0.570	1.406	1599.406	2.590	6.671
4 + 0.65	650.00	2248.000	7.253	9.476	0.683	1.744	1289.095	2.576	7.408
5	350.00	2248.000	7.681	9.904	0.428	1.498	1500.733	2.987	7.795
5 + 0.65	650.00	2248.000	8.174	8.649	0.493	1.136	1978.400	2.761	8.240
6	350.00	2248.000	8.350	8.825	0.177	1.069	2103.518	2.935	8.409
6 + 0.65	650.00	2248.000	8.683	8.550	0.332	1.200	1872.734	2.637	8.756
7	350.00	2248.000	8.889	8.756	0.206	1.114	2017.786	2.842	8.952
7 + 0.65	650.00	2248.000	9.191	9.364	0.302	1.055	2130.096	2.989	9.247
8	350.00	2248.000	9.330	9.503	0.139	1.009	2228.408	3.127	9.382
8 + 0.65	650.00	2248.000	9.589	11.798	0.259	1.084	2074.444	2.843	9.649
9	350.00	2248.000	9.745	11.954	0.156	1.028	2187.431	2.998	9.799
9 + 0.65	650.00	2248.000	10.014	8.485	0.270	1.158	1940.901	3.055	10.083
10	350.00	2248.000	10.177	8.648	0.163	1.100	2043.564	3.216	10.239
10 + 0.65	650.00	2248.000	10.457	8.196	0.281	1.192	1886.158	3.164	10.530
11	350.00	2248.000	10.622	8.361	0.165	1.133	1983.759	3.328	10.688
11 + 0.65	650.00	2248.000	10.898	8.887	0.276	1.076	2089.071	3.081	10.957
12	350.00	2248.000	11.038	9.027	0.140	1.030	2183.190	3.219	11.092
12 + 0.65	650.00	2248.000	11.322	11.712	0.284	1.306	1720.996	3.048	11.409
13	350.00	2248.000	11.527	11.917	0.205	1.224	1836.098	3.252	11.604
13 + 0.65	650.00	2248.000	11.803	7.953	0.276	0.884	2543.368	3.475	11.843
14	350.00	2248.000	11.885	8.035	0.082	0.864	2602.777	3.557	11.923
14 + 0.65	650.00	2248.000	12.202	7.706	0.317	1.300	1729.291	2.269	12.288
15	350.00	2248.000	12.486	7.990	0.284	1.156	1944.973	2.552	12.554
15 + 0.35	350.00	2248.000	12.692	11.583	0.206	1.404	1600.816	2.926	12.793
15 + 0.65	300.00	2248.000	12.945	9.338	0.253	1.238	1816.290	2.355	13.023
16	350.00	2248.000	13.194	9.587	0.249	1.120	2007.718	2.604	13.258
16 + 0.65	650.00	2248.000	13.448	9.580	0.253	0.526	4274.641	4.147	13.462
16 + 0.90	250.00	2248.000	13.464	9.596	0.017	0.524	4291.704	4.163	13.478

Table F.5.2.(1) Case II. Hydraulic Analysis by Uniform Flow (Rio Matina)

Point No.		Section (m)	Discharge (m3/s)	Water Level EL. (m)	Water Depth H(m)	Difference of Water Level (m)	Velocity V(m/s)	Cross-Sectional Area A(m²)	Hydraulic Mean Depth R(m)	Energy Gradient E(m)
0	-1.50	0.00	2248.000	2.000	6.733	0.000	2.447	918.649	2.813	2.306
0	-0.50	1000.00	2248.000	3.737	7.000	1.737	1.728	1300.869	3.991	3.829
0		500.00	2248.000	4.091	7.354	0.355	1.588	1415.873	4.344	4.220
0 +	0.50	500.00	2248.000	4.401	7.098	0.310	1.458	1541.990	3.725	4.509
1		500.00	2248.000	4.678	7.375	0.277	1.358	1655.716	4.000	4.772
1 +	0.50	500.00	2248.000	4.950	7.740	0.272	1.633	1376.230	3.535	5.086
2		500.00	2248.000	5.315	8.105	0.366	1.482	1517.298	3.898	5.427
2 +	0.50	500.00	2248.000	5.590	8.295	0.275	0.969	2320.904	3.612	5.638
3		500.00	2248.000	5.721	8.426	0.131	0.935	2404.613	3.743	5.765
3 +	0.65	650.00	2248.000	6.226	7.179	0.505	1.837	1223.740	2.370	6.398
4		350.00	2248.000	6.739	7.692	0.513	1.513	1486.068	2.859	6.856
4 +	0.65	650.00	2248.000	7.389	9.612	0.650	1.657	1356.314	2.701	7.529
5		350.00	2248.000	7.759	9.982	0.370	1.460	1539.423	3.064	7.868
5 +	0.65	650.00	2248.000	8.243	8.718	0.484	1.396	1610.237	3.096	8.342
6		350.00	2248.000	8.472	8.947	0.229	1.301	1727.563	3.322	8.558
6 +	0.65	650.00	2248.000	8.867	8.734	0.395	1.425	1577.833	3.093	8.970
7		350.00	2248.000	9.105	8.972	0.238	1.324	1697.672	3.328	9.195
7 +	0.65	650.00	2248.000	9.452	9.625	0.347	1.253	1793.489	3.499	9.532
8		350.00	2248.000	9.613	9.786	0.161	1.199	1874.758	3.658	9.686
8 +	0.65	650.00	2248.000	9.894	12.103	0.281	1.232	1824.874	3.421	9.972
9		350.00	2248.000	10.054	12.263	0.160	1.177	1909.462	3.580	10.125
9 +	0.65	650.00	2248.000	10.321	8.792	0.267	1.187	1894.106	3.538	10.393
10		350.00	2248.000	10.464	8.935	0.143	1.141	1969.947	3.679	10.530
10 +	0.65	650.00	2248.000	10.713	8.452	0.249	1.185	1896.582	3.538	10.785
11		350.00	2248.000	10.855	8.594	0.142	1.140	1972.375	3.679	10.922
11 +	0.65	650.00	2248.000	11.109	9.098	0.254	1.229	1829.811	3.531	11.186
12		350.00	2248.000	11.262	9.251	0.153	1.178	1908.476	3.683	11.333
12 +	0.65	650.00	2248.000	11.532	11.922	0.269	1.321	1701.345	3.662	11.621
13		350.00	2248.000	11.700	12.090	0.169	1.264	1779.050	3.829	11.782
13 +	0.65	650.00	2248.000	11.959	8.109	0.258	1.163	1932.903	4.097	12.028
14		350.00	2248.000	12.073	8.223	0.114	1.132	1986.140	4.210	12.138
14 +	0.65	650.00	2248.000	12.381	7.885	0.308	1.539	1460.335	3.160	12.502
15		350.00	2248.000	12.651	8.155	0.270	1.419	1584.006	3.427	12.753
15 +	0.35	350.00	2248.000	12.825	11.866	0.175	1.153	1949.523	6.574	12.893
15 +	0.65	300.00	2248.000	12.907	9.300	0.082	1.790	1256.115	3.384	13.070
16		350.00	2248.000	13.239	9.632	0.333	1.631	1378.623	3.714	13.375
16 +	0.65	650.00	2248.000	13.623	9.755	0.383	0.851	2642.272	4.978	13.660
16 +	0.90	250.00	2248.000	13.657	9.789	0.034	0.845	2660.245	5.012	13.693

Table F.5.2.(2) Case II. Hydraulic Analysis by Uniform Flow (Rio Barbilla)

Point No.	Section (m)	Discharge (m3/s)	Water Level EL. (m)	Water Depth H (m)	Difference of Water Level (m)	Velocity V (m/s)	Cross-Sectional Area A (m²)	Hydraulic Mean Depth R (m)	Energy Gradient E (m)
0	0.00	528.000	13.657	8.097	0.000	0.953	554.217	4.238	13.703
0 + 0.90	900.00	528.000	13.839	8.279	0.182	0.915	577.296	4.415	13.881
1	100.00	528.000	13.857	8.297	0.018	0.911	579.643	4.433	13.900
2	1000.00	528.000	14.143	6.348	0.286	1.083	487.573	3.056	14.203
2 + 0.65	650.00	528.000	14.396	6.601	0.252	1.002	526.889	3.303	14.447
3	350.00	528.000	14.507	6.712	0.112	0.970	544.280	3.412	14.555
3 + 0.65	650.00	528.000	14.689	7.461	0.182	0.970	544.360	3.645	14.737
4	350.00	528.000	14.782	7.554	0.093	0.946	557.918	3.735	14.828
4 + 0.65	650.00	528.000	15.003	6.241	0.221	1.377	383.343	3.420	15.100
5	350.00	528.000	15.202	6.440	0.199	1.303	405.082	3.614	15.288
5 + 0.65	650.00	528.000	15.550	5.513	0.349	1.458	362.045	3.456	15.659
6	350.00	528.000	15.770	5.733	0.220	1.374	384.337	3.654	15.866
7	1000.00	528.000	16.950	2.673	1.180	3.053	172.943	1.589	17.426
7 + 0.30	300.00	528.000	17.500	3.223	0.550	2.257	233.936	1.809	17.760
8	700.00	528.000	19.300	2.547	1.800	2.619	201.611	1.936	19.650
8 + 0.30	300.00	528.000	20.351	3.598	1.051	1.694	311.708	2.909	20.498
9	700.00	528.000	22.300	4.048	1.949	1.957	269.750	2.688	22.495
9 + 0.20	200.00	528.000	22.646	4.394	0.346	1.732	304.868	2.527	22.799
10	800.00	528.000	25.800	5.096	3.154	2.553	206.793	2.738	26.133
10 + 0.30	300.00	528.000	26.539	5.835	0.739	2.015	262.070	3.228	26.746

Table F.5.2.(3) Case II. Hydraulic Analysis by Uniform Flow (Rio Chirripo)

Point No.	Section (m)	Discharge (m3/s)	Water Level EL. (m)	Water Depth H (m)	Difference of Water Level (m)	Velocity V (m/s)	Cross-Sectional Area A (m²)	Hydraulic Mean Depth R (m)	Energy Gradient E (m)
0	0.00	1760.000	13.657	9.357	0.000	2.260	778.681	5.875	13.918
1	1000.00	1760.000	14.533	8.733	0.876	2.521	698.119	5.267	14.858
1 + 0.50	500.00	1760.000	15.097	8.597	0.564	2.591	679.293	5.124	15.440
2	500.00	1760.000	15.734	8.334	0.637	0.922	1909.177	5.677	15.777
3	1000.00	1760.000	15.941	6.841	0.207	1.281	1373.598	4.412	16.025
3 + 0.20	200.00	1760.000	16.012	6.512	0.071	1.332	1321.580	4.274	16.102

Table F.5.3.(1) Case III. Hydraulic Analysis of by Uniform Flow (Rio Matina)

Point No.	Section (m)	Discharge (m3/s)	Water Level EL. (m)	Water Depth H(m)	Difference of Water Level (m)	Velocity V(m/s)	Cross-Sectional Area A(m²)	Hydraulic Mean Depth R(m)	Energy Gradient E(m)
0 -1.50	0.00	2248.000	2.000	5.000	0.000	1.450	1550.000	4.808	2.107
0 -0.50	1000.00	2248.000	2.389	5.322	0.389	1.360	1653.159	5.106	2.483
0	500.00	2248.000	2.555	5.455	0.166	1.325	1696.009	5.228	2.645
0 + 0.50	500.00	2248.000	2.708	5.575	0.153	1.296	1734.732	5.344	2.794
1	500.00	2248.000	2.851	5.684	0.142	1.270	1769.667	5.452	2.933
1 + 0.50	500.00	2248.000	2.984	5.784	0.134	1.247	1802.032	5.552	3.064
2	500.00	2248.000	3.111	5.878	0.127	1.227	1832.187	5.645	3.188
2 + 0.50	500.00	2248.000	3.231	5.964	0.120	1.209	1859.825	5.730	3.305
3	500.00	2248.000	3.309	6.009	0.078	2.309	973.530	5.504	3.581
3 + 0.65	650.00	2248.000	3.861	6.344	0.552	2.178	1032.052	5.786	4.103
4	350.00	2248.000	4.119	6.486	0.258	2.127	1056.963	5.905	4.349
4 + 0.65	650.00	2248.000	4.552	6.702	0.433	2.053	1095.099	6.085	4.767
5	350.00	2248.000	4.766	6.799	0.214	2.021	1112.258	6.165	4.974
5 + 0.65	650.00	2248.000	5.138	6.955	0.372	1.972	1139.915	6.294	5.336
6	350.00	2248.000	5.326	7.026	0.189	1.950	1152.647	6.353	5.520
6 + 0.65	650.00	2248.000	5.660	7.143	0.334	1.916	1173.481	6.450	5.847
7	350.00	2248.000	5.831	7.198	0.171	1.900	1183.340	6.495	6.015
7 + 0.65	650.00	2248.000	6.139	7.289	0.308	1.874	1199.662	6.570	6.318
8	350.00	2248.000	6.299	7.332	0.160	1.862	1207.334	6.605	6.476
8 + 0.65	650.00	2248.000	6.577	8.677	0.278	1.648	1363.816	7.641	6.715
9	350.00	2248.000	6.798	7.598	0.221	2.568	875.259	6.533	7.135
9 + 0.65	650.00	2248.000	7.379	7.854	0.581	1.727	1301.391	7.030	7.531
10	350.00	2248.000	7.503	7.803	0.124	1.740	1292.159	6.989	7.657
10 + 0.65	650.00	2248.000	7.738	7.713	0.236	1.762	1275.971	6.916	7.897
11	350.00	2248.000	7.870	7.670	0.132	1.773	1268.182	6.881	8.030
11 + 0.65	650.00	2248.000	8.120	7.595	0.250	1.792	1254.586	6.820	8.284
12	350.00	2248.000	8.259	7.559	0.139	1.801	1248.099	6.790	8.424
12 + 0.65	650.00	2248.000	8.513	8.913	0.254	1.552	1448.114	7.860	8.636
13	350.00	2248.000	8.583	7.383	0.070	1.837	1223.731	6.622	8.755
13 + 0.65	650.00	2248.000	8.865	7.340	0.282	1.860	1208.785	6.612	9.042
14	350.00	2248.000	9.021	7.321	0.156	1.865	1205.392	6.596	9.199
14 + 0.65	650.00	2248.000	9.314	7.289	0.292	1.874	1199.550	6.569	9.493
15	350.00	2248.000	9.474	7.274	0.160	1.878	1196.836	6.557	9.654
15 + 0.35	350.00	2248.000	9.640	8.740	0.167	1.700	1322.700	7.653	9.788
15 + 0.65	300.00	2248.000	9.721	7.196	0.081	1.900	1182.973	6.493	9.905
16	350.00	2248.000	9.888	7.188	0.167	1.903	1181.530	6.487	10.073
16 + 0.65	650.00	2248.000	10.199	7.174	0.311	1.907	1179.063	6.475	10.385
16 + 0.90	250.00	2248.000	10.320	7.170	0.120	1.908	1178.254	6.472	10.505

Table F.5.3.(2) Case III. Hydraulic Analysis of by Uniform Flow (Rio Barbilla)

Point No.	Section (m)	Discharge (m3/s)	Water Level EL. (m)	Water Depth H(m)	Difference of Water Level (m)	Velocity V(m/s)	Cross-Sectional Area A(m²)	Hydraulic Mean Depth R(m)	Energy Gradient E(m)
0	0.00	528.000	10.320	6.020	0.000	0.866	609.449	5.277	10.358
0 + 0.90	900.00	528.000	10.463	5.163	0.143	1.025	515.048	4.595	10.516
1	100.00	528.000	10.485	5.185	0.022	1.022	516.782	4.616	10.538
2	1000.00	528.000	10.754	4.454	0.269	1.206	437.840	4.022	10.828
2 + 0.65	650.00	528.000	11.016	4.066	0.263	1.331	396.824	3.701	11.107
3	350.00	528.000	11.194	3.894	0.178	1.394	378.802	3.557	11.293
3 + 0.65	650.00	528.000	11.544	4.141	0.350	1.297	406.988	3.750	11.630
4	350.00	528.000	11.714	3.856	0.170	1.401	376.758	3.513	11.814
4 + 0.65	650.00	528.000	12.156	3.454	0.442	1.577	334.736	3.174	12.283
5	350.00	528.000	12.466	3.310	0.310	1.651	319.783	3.051	12.605
5 + 0.65	650.00	528.000	13.149	3.149	0.683	1.741	303.229	2.913	13.304
6	350.00	528.000	13.648	2.773	0.500	2.225	237.262	2.568	13.901
7	1000.00	528.000	16.400	3.025	2.752	2.028	260.301	2.783	16.610
7 + 0.30	300.00	528.000	16.939	2.814	0.539	2.191	240.990	2.603	17.184
8	700.00	528.000	18.567	2.692	1.628	2.297	229.854	2.497	18.836
8 + 0.30	300.00	528.000	19.315	2.690	0.748	2.299	229.630	2.495	19.584
9	700.00	528.000	20.915	2.890	1.601	2.130	247.911	2.668	21.147
9 + 0.20	200.00	528.000	21.307	2.882	0.392	2.136	247.205	2.661	21.540
10	800.00	528.000	25.200	4.496	3.893	3.202	164.889	2.333	25.723
10 + 0.30	300.00	528.000	26.517	5.813	1.317	2.028	260.412	3.210	26.727

Table F.5.3.(3) Case III. Hydraulic Analysis of by Uniform Flow (Rio Chirripo)

Point No.	Section (m)	Discharge (m3/s)	Water Level EL. (m)	Water Depth H(m)	Difference of Water Level (m)	Velocity V(m/s)	Cross-Sectional Area A(m²)	Hydraulic Mean Depth R(m)	Energy Gradient E(m)
0	0.00	1760.000	10.320	6.020	0.000	3.564	493.881	5.096	10.968
1	1000.00	1760.000	12.444	6.606	2.124	3.202	549.657	5.522	12.967
1 + 0.50	500.00	1760.000	13.281	6.673	0.838	3.164	556.215	5.571	13.792
2	500.00	1760.000	14.091	6.714	0.810	3.142	560.159	5.600	14.595
3	1000.00	1760.000	15.534	6.434	1.443	1.410	1247.816	4.042	15.636
3 + 0.20	200.00	1760.000	15.631	6.131	0.097	1.461	1204.747	3.927	15.740

Table F.5.4.(1) Case 1. Construction Cost (1)

Work Item	Quantity	Unit Price	Cost
Embankment Works		₡	1,000₡
The Left Bank Side of Rio Matina			
Spreading & Compaction	143 x 1,000m^3	236	33,748
Slope Finishing	278 x 1,000m^2	17	4,726
Sub-total			38,474
The Right Bank Side of Rio Matina			
Spreading & Compaction	128 x 1,000m^3	236	30,208
Slope Finishing	243 x 1,000m^2	17	4,131
Sub-total			34,339
The Left Bank Side of Rio Barbilla			
Spreading & Compaction	98 x 1,000m^3	236	23,128
Slope Finishing	56 x 1,000m^2	17	952
Sub-total			24,080
The Right Bank Side of Rio Barbilla			
Spreading & Compaction	103 x 1,000m^3	236	24,308
Slope Finishing	59 x 1,000m^2	17	1,003
Sub-total			25,311
The Left Bank Side of Rio Chirripo			
Spreading & Compaction	24 x 1,000m^3	236	5,664
Slope Finishing	13 x 1,000m^2	17	221
Sub-total			5,885
The Right Bank Side of Rio Chirripo			
Spreading & Compaction	26 x 1,000m^3	236	6,136
Slope Finishing	13 x 1,000m^2	17	221
Sub-total			6,357
Total			134,446
Excavation Works of the Rivers			
Rio Matina	436 x 1,000^3	350	152,600
Rio Barbilla	367 x 1,000^3	350	128,450
Total			281,050

Table F.5.4.(1) Case I. Construction Cost(2)

Work Item	Quantity	Unit Price	Cost
Cleaning Works		¢	1,000¢
Cleaning	1,885 ha	73,186	137,955
Total			137,955
Purchase Land			
Cacao	115 ha	150,000	17,250
Rice	-	-	-
Cattle	305 ha	60,000	18,300
Banana Plantation	600 ha	250,000	150,000
Others	850 ha	60,000	51,000
Settlements	15 ha	35,000	525
Total			237,075
Moving Houses and Others			
Equipments for Banana Plantation	2	40,000,000	80,000
Houses	30	350,000	10,500
Total			90,500
Grand Total			881,026

Table F.5.4.(2) Case II. Construction Cost (1)

Work Item	Quantity	Unit Price	Cost
Embankment Works		₡	1,000₡
The Left Bank Side of Rio Matina			
Spreading & Compaction	357 x 1,000m^3	236	84,252
Slope Finishing	171 x 1,000m^2	17	2,907
Sub-total			87,159
The Right Bank Side of Rio Matina			
Spreading & Compaction	357 x 1,000m^3	236	84,252
Slope Finishing	171 x 1,000m^2	17	2,907
Sub-total			87,159
The Left Bank Side of Rio Barbilla			
Spreading & Compaction	98 x 1,000m^3	236	23,128
Slope Finishing	56 x 1,000m^2	17	952
Sub-total			24,080
The Right Bank Side of Rio Barbilla			
Spreading & Compaction	103 x 1,000m^3	236	24,308
Slope Finishing	59 x 1,000m^2	17	1,003
Sub-total			25,311
The Left Bank Side of Rio Chirripo			
Spreading & Compaction	24 x 1,000m^3	236	5,664
Slope Finishing	13 x 1,000m^2	17	221
Sub-total			5,885
The Right Bank Side of Rio Chirripo			
Spreading & Compaction	26 x 1,000m^3	236	6,136
Slope Finishing	13 x 1,000m^2	17	221
Sub-total			6,357
Total			235,951
Excavations Work of the Rivers			
Rio Matina	436 x $1,000^3$	350	152,600
Rio Barbilla	367 x $1,000^3$	350	128,450
Total			281,050

Table F.5.4.(2) Case II. Construction Cost (2)

Work Item	Quantity	Unit Price	Cost
Cleaning Works		₡	1,000₡
Cleaning	675 ha	73,186	49,400
Total			49,400
Purchase Land			
Cacao	65 ha	150,000	9,750
Rice	-	-	-
Cattle	134 ha	60,000	8,040
Banana Plantation	183 ha	250,000	45,750
Others	277 ha	60,000	16,620
Settlements	16 ha	35,000	560
Total			80,720
Moving Houses and Others			
Equipments for Banana Plantation	-	-	-
Houses	12	350,000	4,200
Total			4,200
Grand Total			651,321

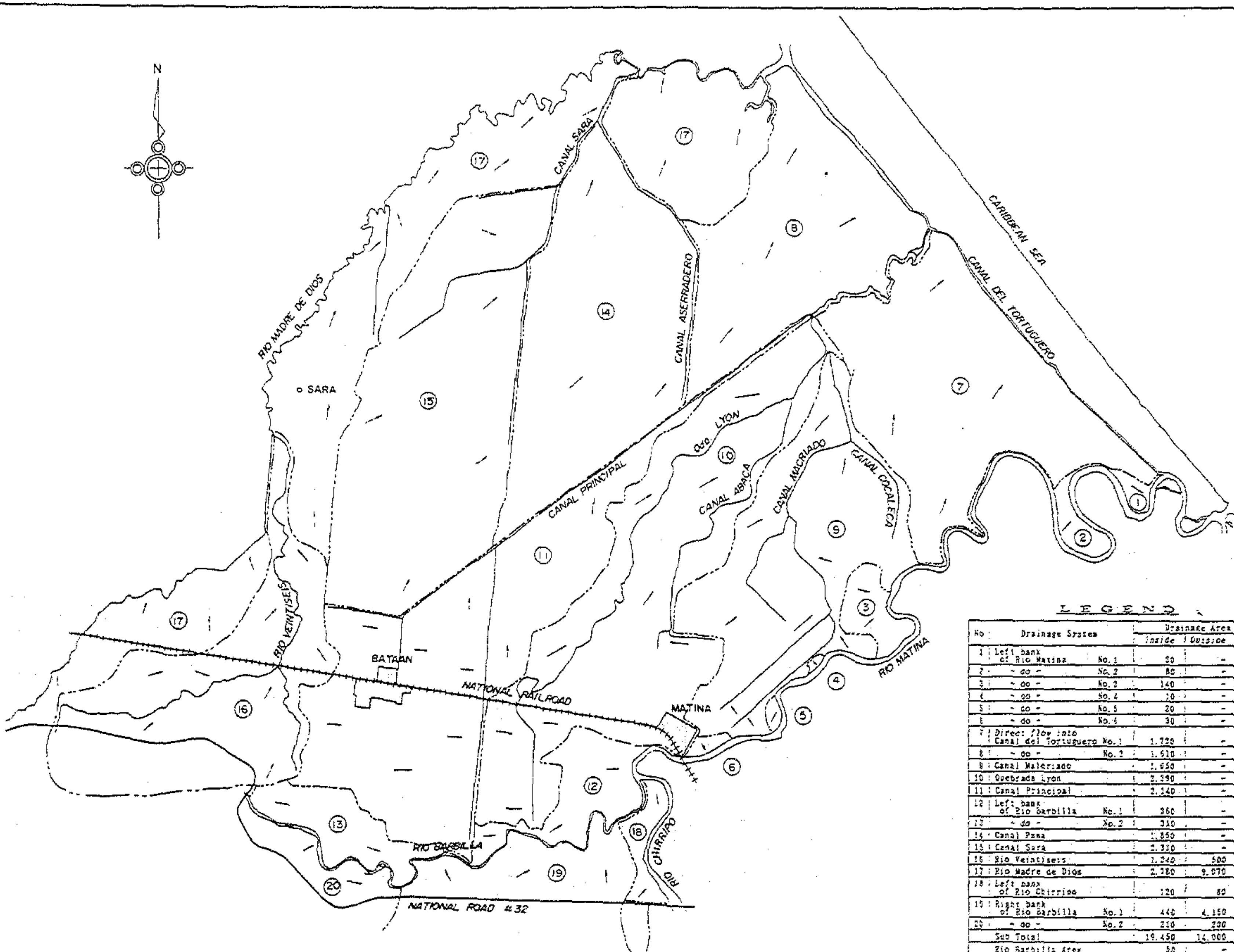
Table F.5.4.(3) Case III. Construction Cost

Work Item	Quantity	Unit Price	Cost
Embankment Works		₡	1,000₡
Rio Matina			
Spreading & Compaction	157 x 1,000m^3	236	37,052
Slope Finishing	71 x 1,000m^2	17	1,207
Total			38,259
Excavation Works of the Rivers			
Rio Matina	13,900 x 1,000^3	350	4,865,000
Rio Barbilla	3,200 x 1,000^3	350	1,120,000
Rio Chirripo	900 x 1,000^3	350	315,000
Total			6,300,000
Cleaning Works			
Cleaning	220 ha	73,186	16,100
Total			16,100
Purchase Land			
Cacao	20 ha	150,000	3,000
Cattle	60 ha	60,000	3,600
Banana Plantation	50 ha	250,000	12,500
Others	90 ha	60,000	5,400
Total			24,500
Grand Total			6,378,859

Table F.5.5. Soil Texture Analysis

Sample	Point	Sand	Loam	Clay	R.S.Gravity	Soil Texture
		%	%	%		
No.1	Left side of midstream of Rio Barbilla	18	59	23	2.12	Silt Loam
No.2	Right side of midstream of Rio Barbilla	42	37	21	2.28	Loam
No.3	Left side of midstream of Rio Matina	22	63	15	2.42	Silt Loam
No.4	Left side of downstream of Rio Matina	68	9	23	2.56	Sandy Clay Loam
No.5	Right side of midstream of Rio Matina	30	43	27	2.19	Loam to Clay Loam
No.6	Right side of downstream of Rio Matina	20	57	23	2.37	Silt Loam

Note: 1. The samples were taken from Test Pit in depth 1m.
2. This analysis was carried out in M.A.G.



LEGEND

No	Drainage System		Drainage Area (ha)		
			Inside	Outside	Total
1	Left bank of Rio Matina	No. 1	30	-	20
2	- do -	No. 2	80	-	80
3	- do -	No. 3	140	-	140
4	- do -	No. 4	10	-	10
5	- do -	No. 5	20	-	20
6	- do -	No. 6	30	-	30
7	Direct flow into Canal del Tortuguero	No. 1	1,720	-	1,720
8	- do -	No. 2	1,610	-	1,510
9	Canal Malcriado		1,650	-	1,650
10	Quebrada Lyon		2,390	-	2,350
11	Canal Principal		2,140	-	2,140
12	Left bank of Rio Barbilla	No. 1	360	-	360
13	- do -	No. 2	310	-	310
14	Canal Pama		1,850	-	1,850
15	Canal Sara		2,310	-	2,310
16	Rio Veintiseis		1,240	500	1,740
17	Rio Madre de Dios		2,780	9,070	11,850
18	Left bank of Rio Chirripo		120	80	200
19	Right bank of Rio Barbilla	No. 1	440	4,150	4,590
20	- do -	No. 2	210	200	410
	Sub Total		19,450	14,000	33,450
	Rio Barbilla Area		50	-	50
	Total		19,500	14,000	33,500

Fig. F.1.1 Present Drainage System

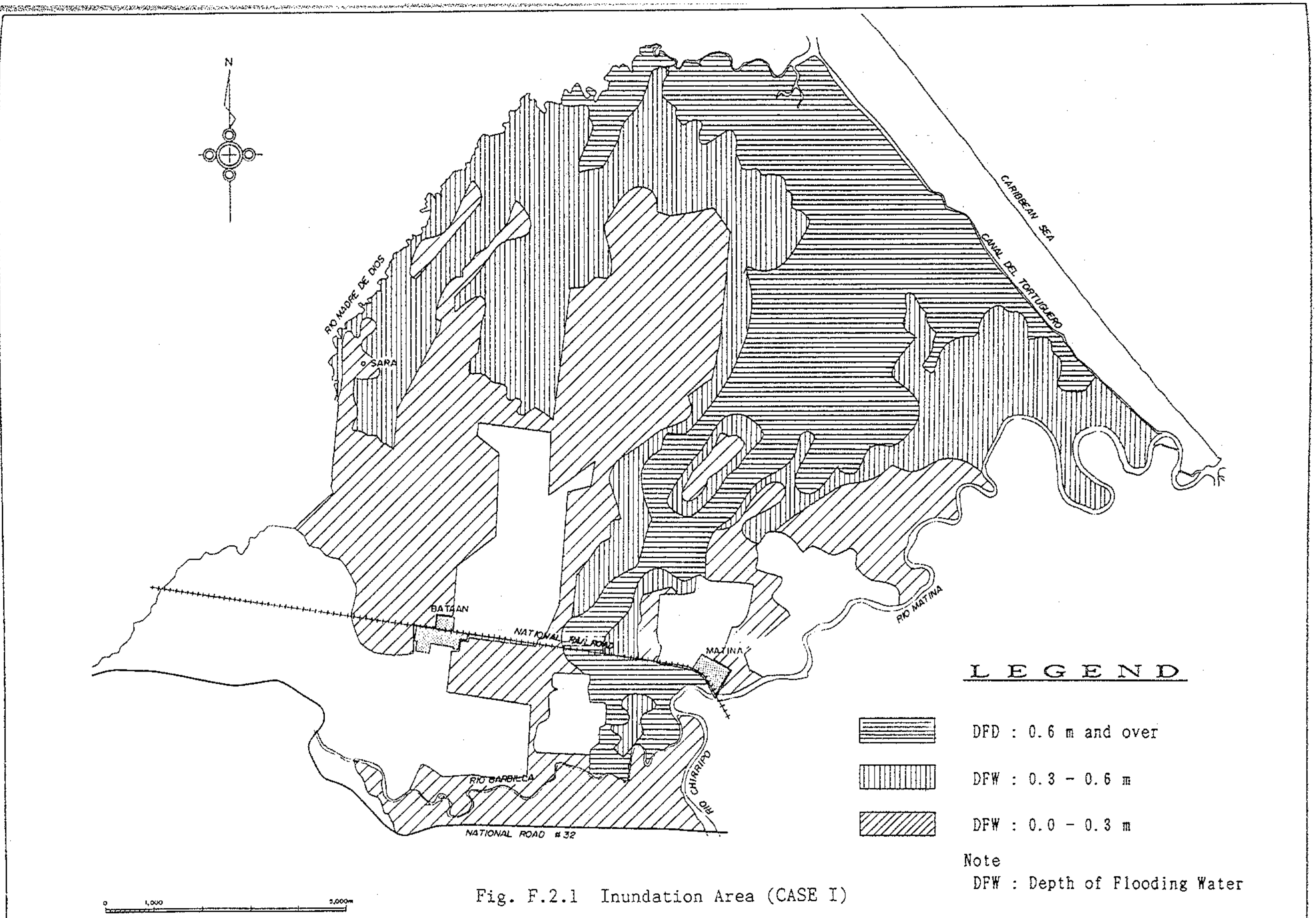


Fig. F.2.1 Inundation Area (CASE I)

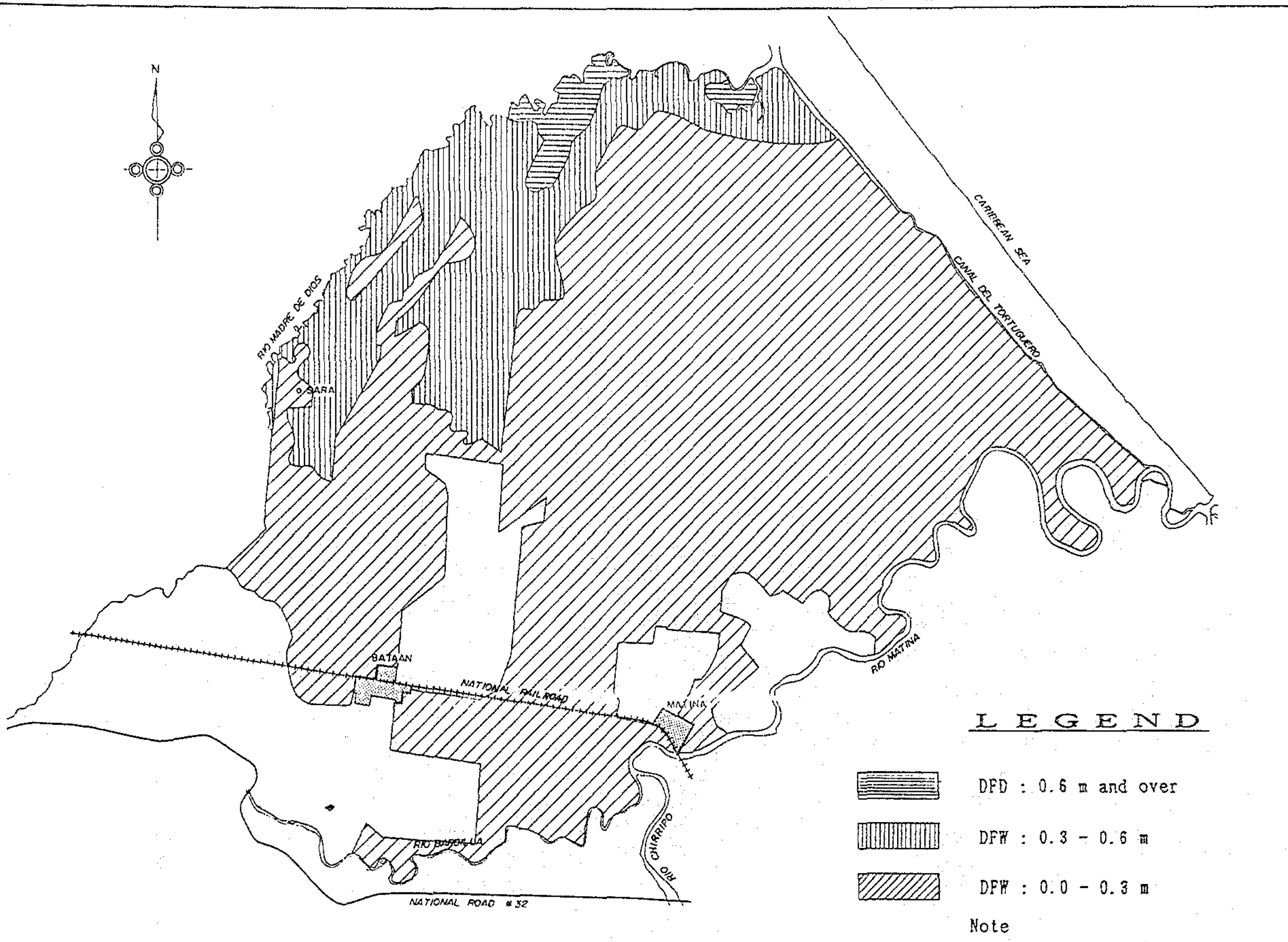


Fig. F.2.2 Inundation Area (CASE II)

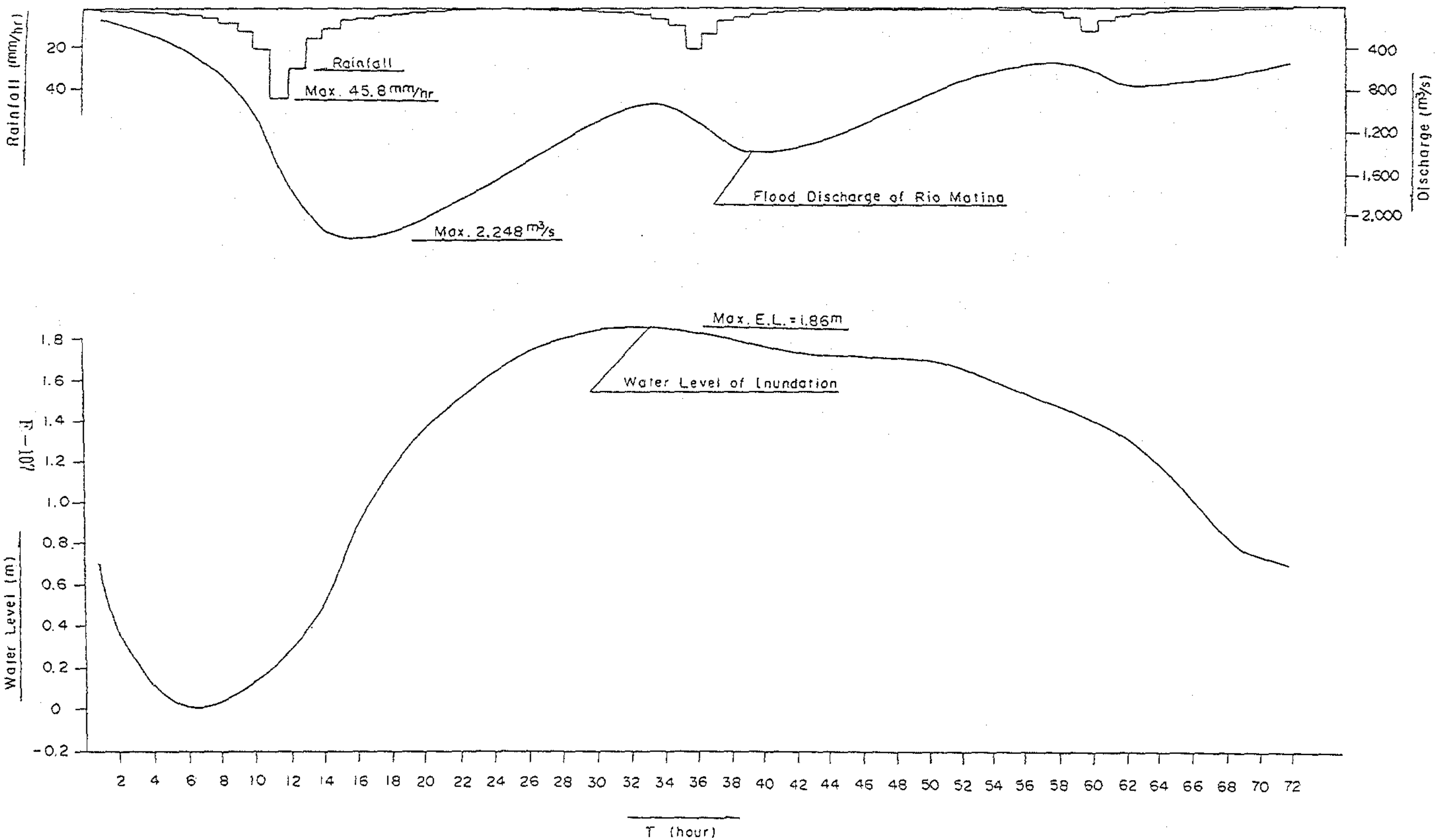


Fig. F.2.3. Inundations Analysis of the Swamp Area (Present P.=1/5)

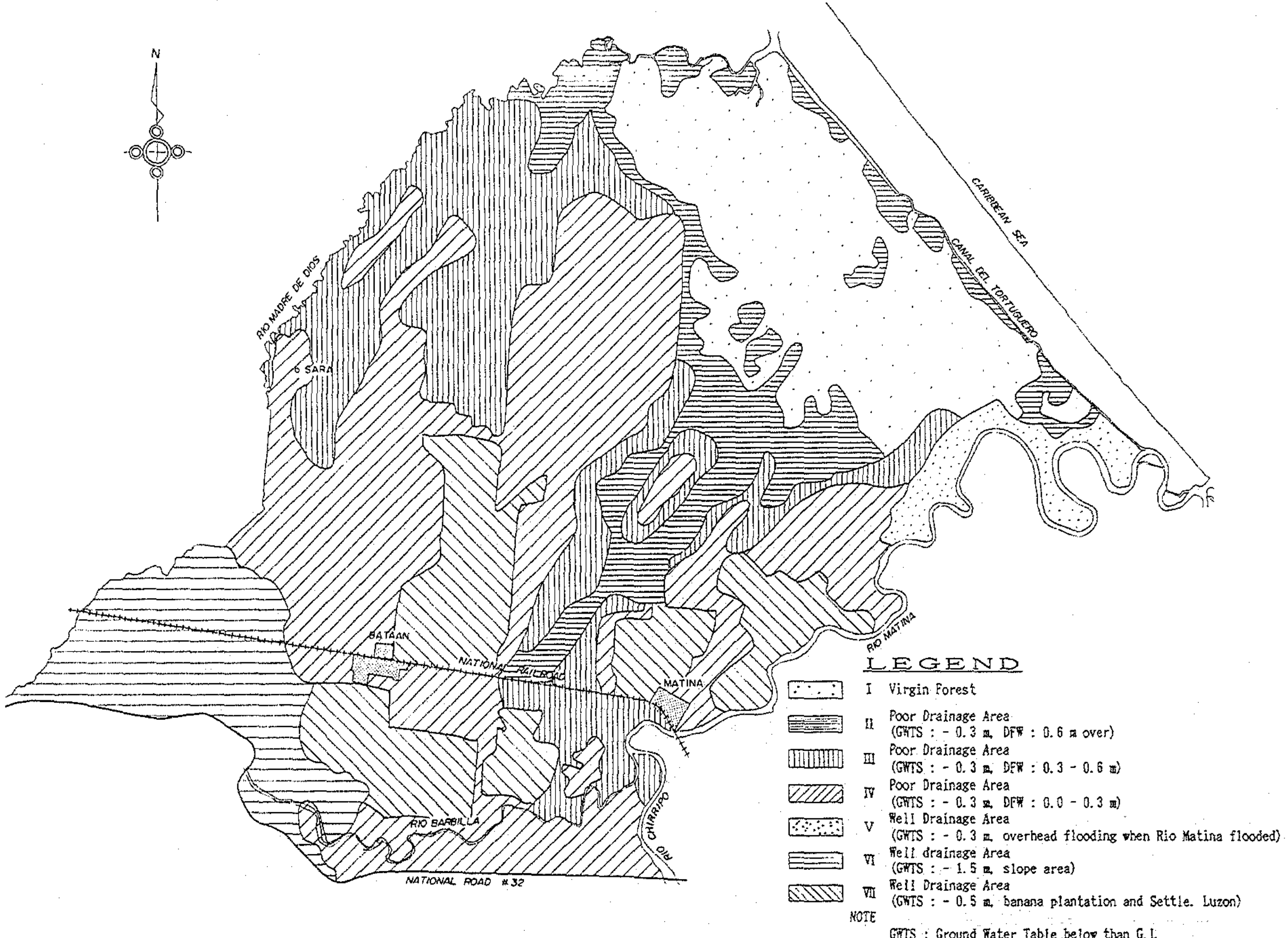


Fig. F.2.4 Present Drainage Condition

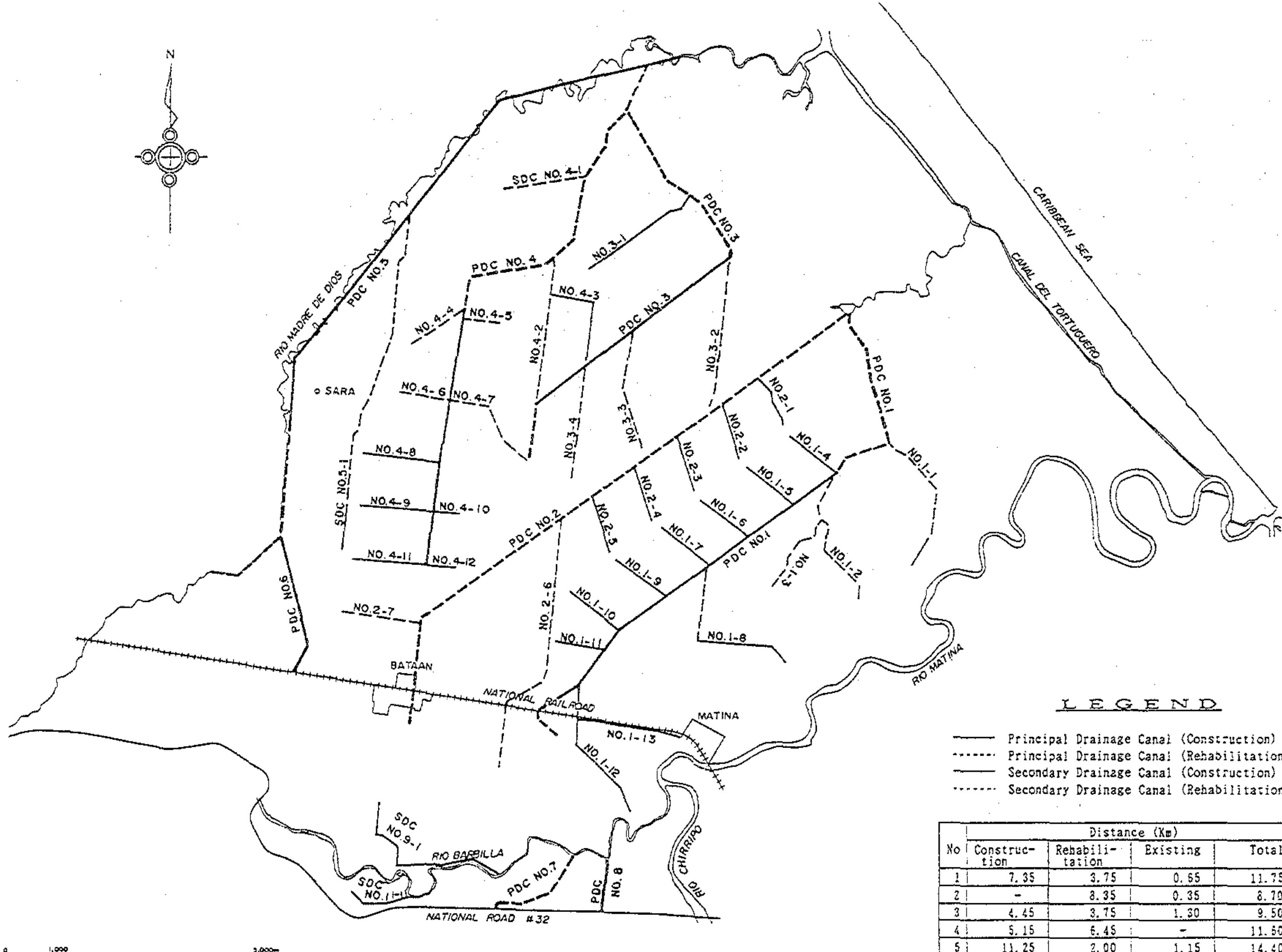
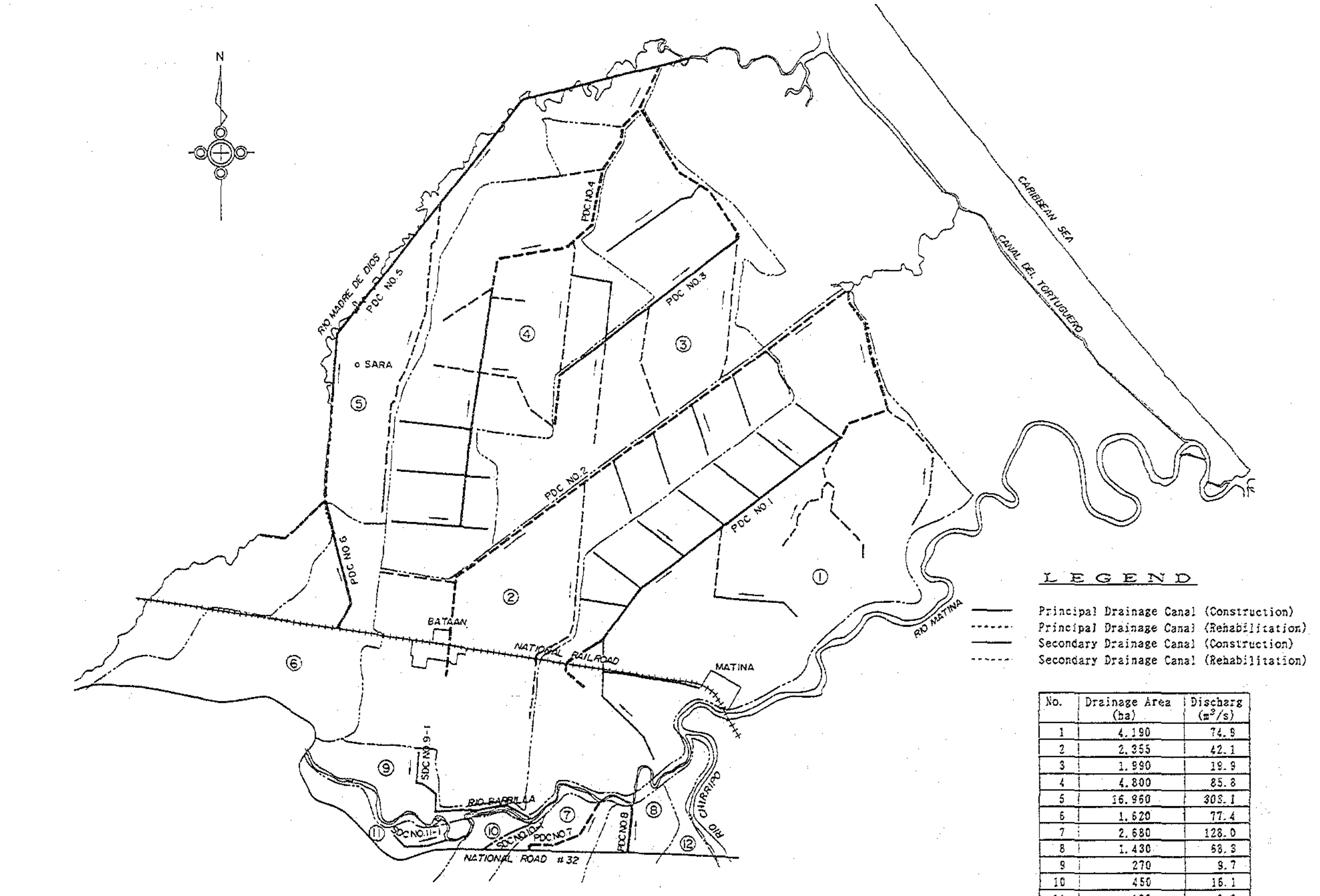


Fig. F.4.1 Proposed Drainage Canal Network

LEGEND

- ——— Principal Drainage Canal (Construction)
- - - - - Principal Drainage Canal (Rehabilitation)
- ——— Secondary Drainage Canal (Construction)
- - - - - Secondary Drainage Canal (Rehabilitation)

No	Distance (Km)			
	Construction	Rehabilitation	Existing	Total
1	7.35	3.75	0.65	11.75
2	-	8.35	0.35	8.70
3	4.45	3.75	1.30	9.50
4	5.15	6.45	-	11.60
5	11.25	2.00	1.15	14.40
6	2.65	-	-	2.65
7	0.25	1.65	-	1.90
8	1.00	-	-	1.00



No.	Drainage Area (ha)	Discharg (m^3/s)
1	4.190	74.9
2	2.355	42.1
3	1.990	19.9
4	4.800	85.8
5	16.960	303.1
6	1.620	77.4
7	2.680	128.0
8	1.430	68.3
9	270	9.7
10	450	16.1
11	100	3.6
12	200	7.2

Fig. F.4.2 Proposed Drainage System

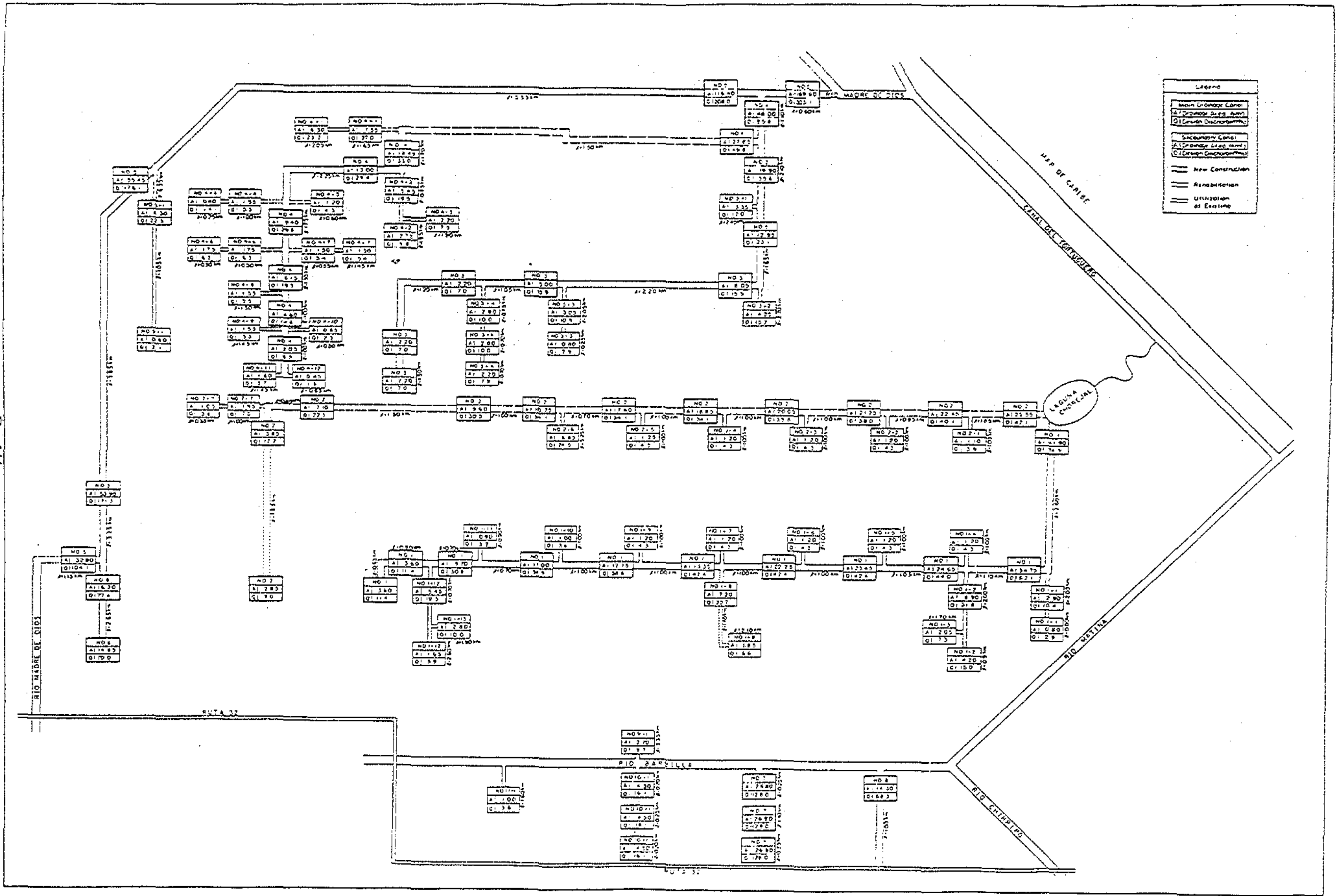


Fig. F.4.3 Proposed Drainage Diagram

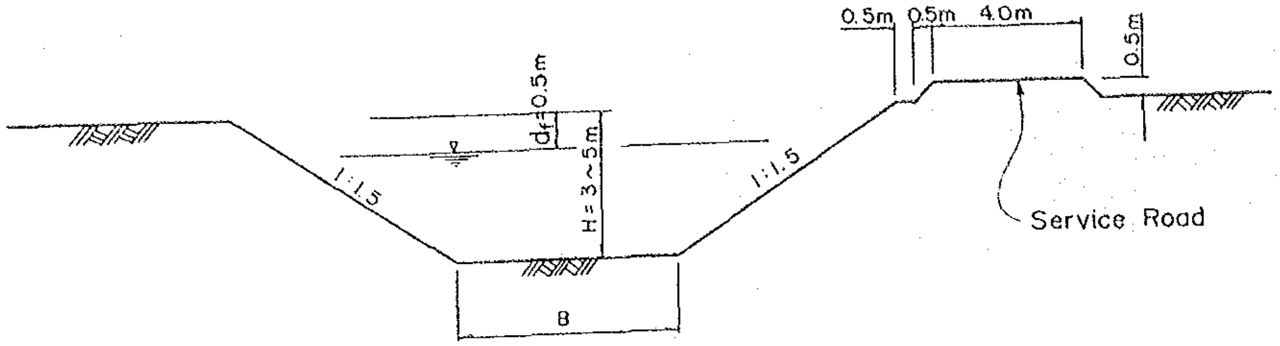
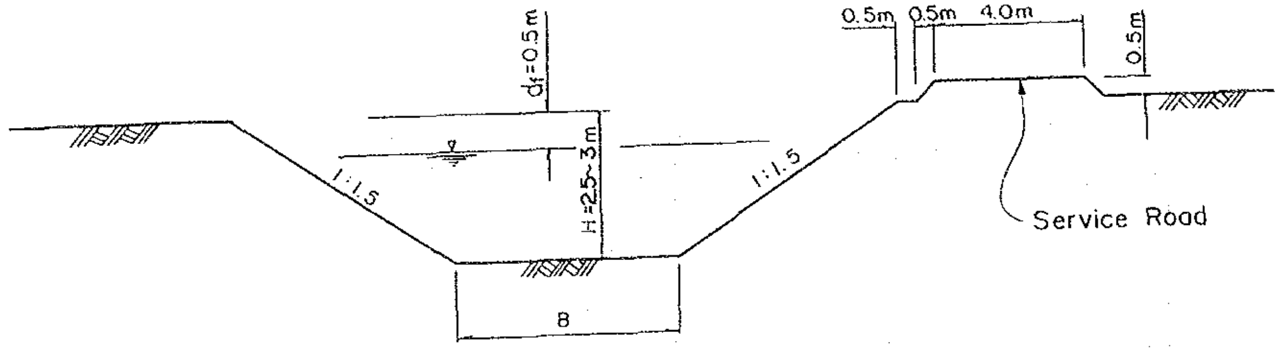


Fig. F.4.4 Proposed Cross Section of Principal Drainage Canal



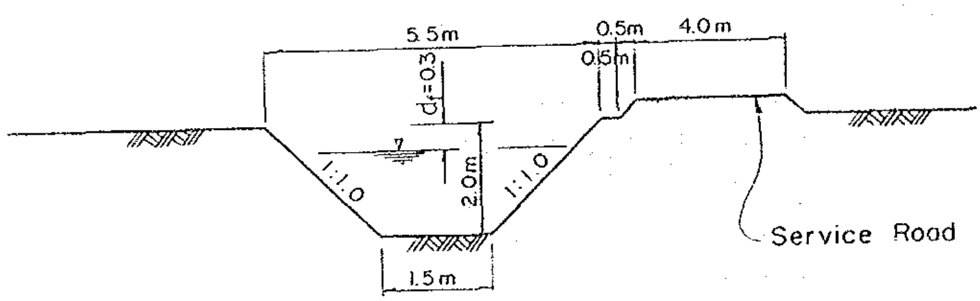


Fig. F.4.5 Proposed Cross Section of Secondary Drainage Canal

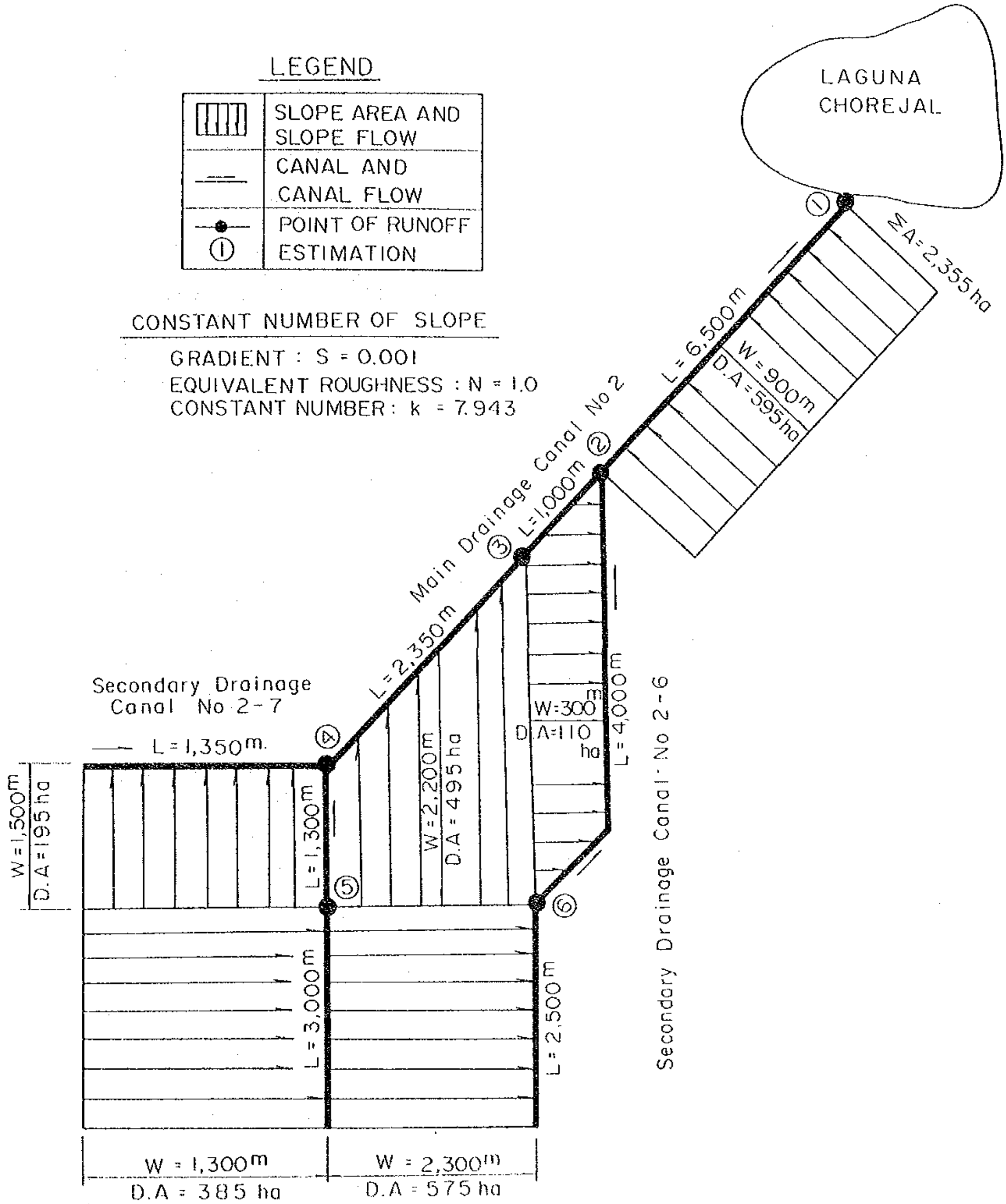


Fig. F.4.6. Runoff Model for Small and Medium River's Watershed

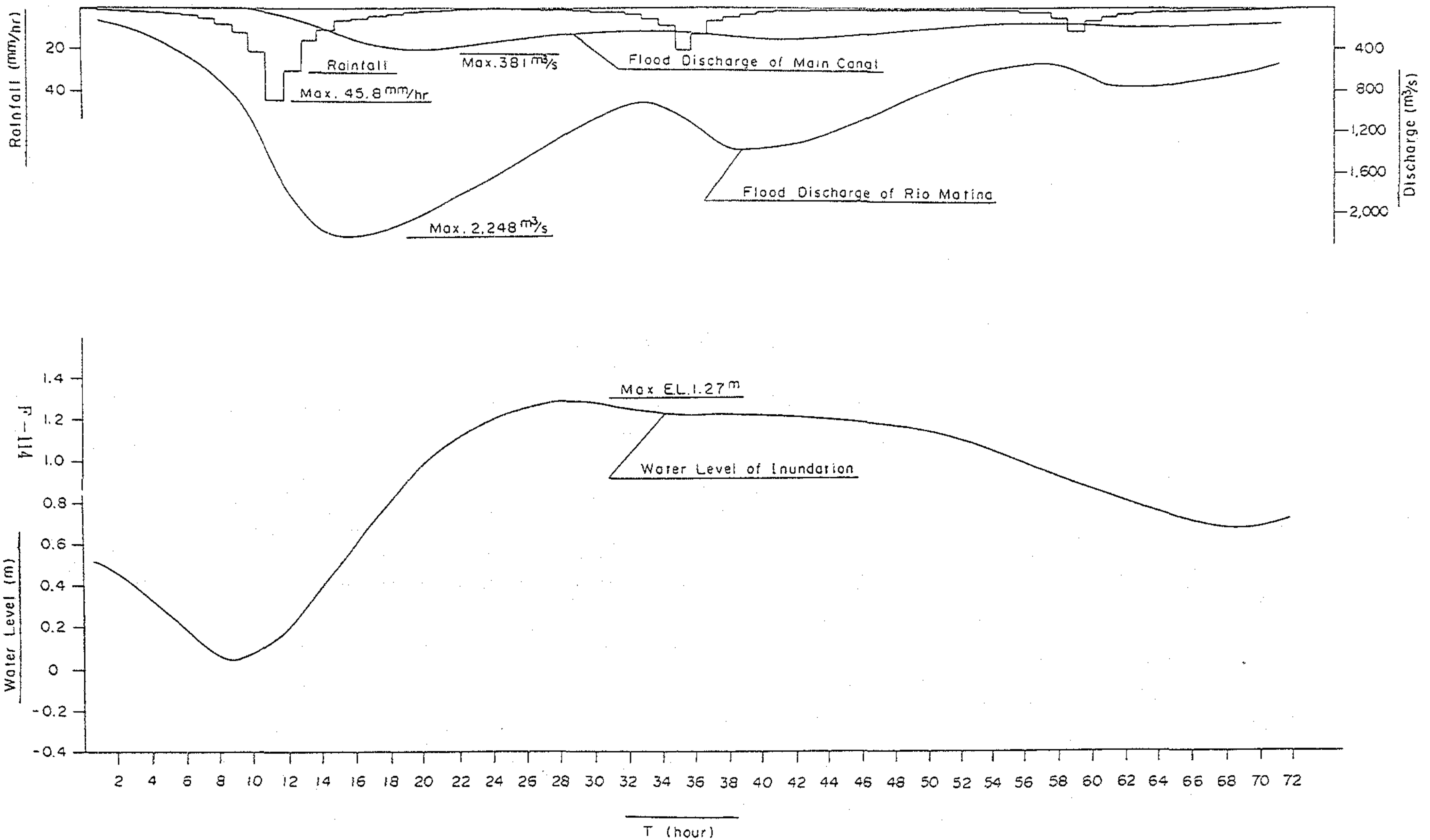
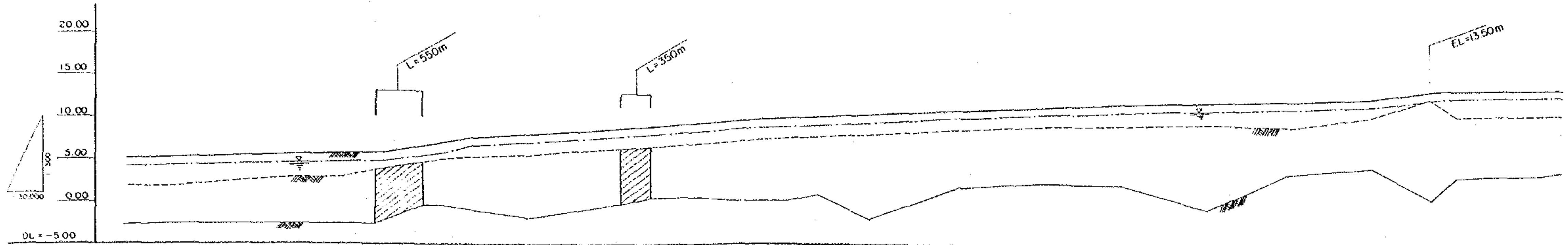


Fig. F.4.7. Inundations Analysis of the Swamp Area (Projection P.=1/5)

Figure F.5.1 CASE I Profile of Proposed Flood Protection Facilities
(Rio Marina)

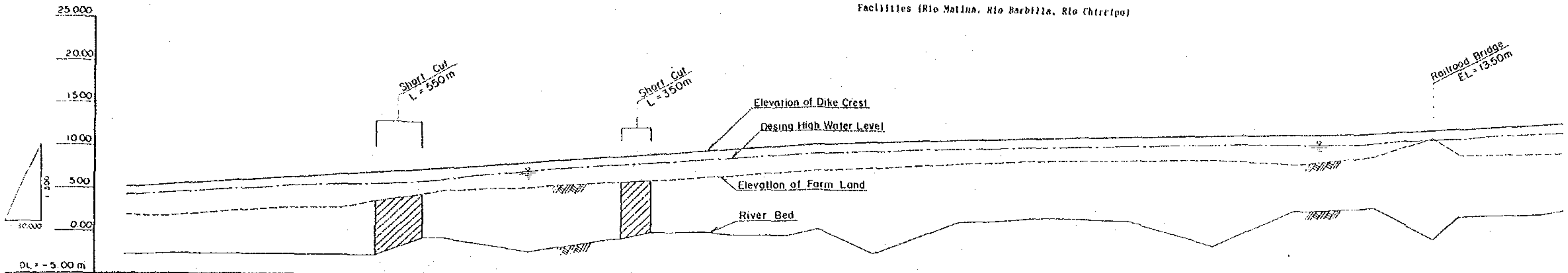


DESIGN RIVER LOCATION	1310	1310	1310	1250	1290	1290	1290	1320	1320	1300	1300	1300
LONGITUDINAL DISTANCE	L=2,500m / L=1,600m						L=1,300m / L= 600m		L=750m / L=2,550m			
HEIGHT OF DIKE	3.40	3.60	3.30	3.00	2.90	2.90	1.90	2.10	2.30	2.60	2.50	2.40
ELEVATION OF DIKE CREST	5.10	5.30	5.40	5.50	5.60	5.70	5.80	7.00	7.60	8.10	8.30	8.70
DESIGN HIGH WATER LEVEL	4.07	4.13	4.20	4.31	4.46	4.60	4.70	5.60	6.59	7.00	7.22	7.67
ELEVATION OF MAJOR BED	1.7	1.7	2.1	2.5	2.7	2.8	3.9	2.9	5.1	5.5	5.8	6.5
RIVER BED	-2.9	-2.7	-2.7	-2.7	-2.7	-2.7	1.391	-0.6	-1.1	-7.0	-1.4	-0.4
ACCUMULATION DISTANCE	0	0.50	1.00	1.50	2.00	2.50	3.00	3.65	4.00	4.65	5.00	5.65
DISTANCE	0	0.50	0.50	0.50	0.50	0.50	0.50	0.65	0.35	0.65	0.35	0.65
STATION	No.0	+0.5	No.1	+0.5	No.2	+0.5	No.3	+0.65	No.4	+0.65	No.5	+0.65

DESIGN RIVER LOCATION	1300	1300	1300	1300	1300	1330	1330	1330	1330	1330	1330	1310
LONGITUDINAL DISTANCE	L=2,050m / L=3,300m						L=2,850m / L= 0m				L=950m / L=1,150m	
HEIGHT OF DIKE	2.50	2.50	2.50	2.50	2.50	2.50	2.50	2.50	2.50	2.50	2.60	2.80
ELEVATION OF DIKE CREST	8.90	9.30	9.50	9.90	10.10	10.50	10.70	11.10	11.30	11.60	11.80	12.00
DESIGN HIGH WATER LEVEL	7.39	8.24	8.49	8.84	9.05	9.42	9.63	10.02	10.19	10.51	10.67	10.95
ELEVATION OF MAJOR BED	6.4	6.8	7.0	7.4	7.6	8.0	8.2	8.6	8.8	9.1	9.2	9.4
RIVER BED	16.41	0.4	0.1	0	0.7	2.1	0.8	1.7	1.9	2.4	2.4	2.3
ACCUMULATION DISTANCE	6.00	6.65	7.00	0.65	8.00	8.65	9.00	9.65	10.00	10.65	11.00	11.65
DISTANCE	0.35	0.65	0.35	0.65	0.35	0.65	0.35	0.65	0.35	0.65	0.35	0.65
STATION	No.6	+0.65	No.7	+0.65	No.8	+0.65	No.9	+0.65	No.10	+0.65	No.11	+0.65

DESIGN RIVER LOCATION	1310	1360	1360	1380	1380	1360	1360	600	1370	1370	1630	1630
LONGITUDINAL DISTANCE	L=2,600m / L=2,400m					L=850m / L=850m		L=500m / L=0m	L=0m / L=2,750m			
HEIGHT OF DIKE	2.60	2.60	2.80	3.10	2.80	2.20	1.60	0.90	3.10	3.20	3.20	3.20
ELEVATION OF DIKE CREST	12.10	12.40	12.50	12.70	12.80	13.00	13.30	13.90	14.10	14.20	14.30	14.30
DESIGN HIGH WATER LEVEL	11.10	11.37	11.47	11.65	11.70	11.99	12.41	12.81	13.06	13.15	13.22	13.25
ELEVATION OF MAJOR BED	9.5	9.8	9.7	9.6	10.0	10.6	11.9	13.0	11.0	11.0	11.1	11.1
RIVER BED	1.4	-0.4	1.1	4.0	4.2	4.7	2.6	0.9	3.6	3.7	3.9	4.5
ACCUMULATION DISTANCE	12.00	12.65	13.00	13.65	14.00	14.65	15.00	15.35	15.65	16.00	16.65	16.90
DISTANCE	0.35	0.65	0.35	0.65	0.35	0.65	0.35	0.35	0.30	0.35	0.65	0.25
STATION	No.12	+0.65	No.13	+0.65	No.14	+0.65	No.15	+0.35	+0.65	No.16	+0.65	+0.70

Figure F.5.2(1) CASE II Profile of Proposed Flood Protection Facilities (Rio Matina, Rio Barbilla, Rio Chirripo)



DESIGN RIVER LOCATION (m)	330	420	420	390	390	640	640	520	520	500	500	520
HEIGHT OF DIKE (m)	3.40	3.70	3.60	3.60	3.70	4.00	3.20	2.70	2.70	3.00	3.00	3.00
ELEVATION OF DIKE CREST (EL. m)	5.10	5.40	5.70	6.10	6.40	6.80	7.10	7.60	7.80	8.50	8.80	9.30
DESIGN HIGH WATER LEVEL (EL. m)	4.09	4.40	4.68	5.00	5.32	5.59	5.72	6.23	6.74	7.39	7.76	8.24
ELEVATION OF MAJOR BED (EL. m)	1.7	1.7	2.1	2.5	2.7	2.6	3.9	4.9	5.1	5.5	5.8	6.3
RIVER BED (EL. m)	-2.9	-2.7	-2.7	-2.7	-2.7	-2.7	(-3.9)	-0.6	-1.1	-2.0	-1.4	-0.4
ACCUMULATION DISTANCE (Km)	0	0.50	1.00	1.50	2.00	2.50	3.00	3.65	4.00	4.65	5.00	5.65
DISTANCE (Km)	0	0.50	0.50	0.50	0.50	0.50	0.50	0.65	0.35	0.65	0.35	0.65
STATION	No.0	+0.5	No.1	+0.5	No.2	+0.5	No.3	+0.65	No.4	+0.65	No.5	+0.65

DESIGN RIVER LOCATION (m)	520	510	510	510	510	530	530	540	540	540	540	520
HEIGHT OF DIKE (m)	3.20	3.10	3.20	3.10	3.10	3.00	2.90	2.80	2.70	2.70	2.70	2.60
ELEVATION OF DIKE CREST (EL. m)	9.50	9.90	10.20	10.30	10.70	11.00	11.10	11.40	11.50	11.80	11.90	12.20
DESIGN HIGH WATER LEVEL (EL. m)	8.47	8.87	9.11	9.45	9.61	9.89	10.05	10.32	10.46	10.71	10.86	11.11
ELEVATION OF MAJOR BED (EL. m)	6.4	6.6	7.0	7.4	7.6	8.0	8.2	8.6	8.8	9.1	9.2	9.4
RIVER BED (EL. m)	(6.4)	0.4	0.1	0	0.7	-2.1	-0.8	1.7	1.9	2.4	2.4	2.3
ACCUMULATION DISTANCE (Km)	6.00	6.65	7.00	7.65	8.00	8.65	9.00	9.65	10.00	10.65	11.00	11.65
DISTANCE (Km)	0.35	0.65	0.35	0.65	0.35	0.65	0.35	0.65	0.35	0.65	0.35	0.65
STATION	No.6	+0.65	No.7	+0.65	No.8	+0.65	No.9	+0.65	No.10	+0.65	No.11	+0.65

DESIGN RIVER LOCATION (m)	520	470	470	470	470	460	460	300	380	360	330	330
HEIGHT OF DIKE (m)	2.90	2.90	3.10	3.50	3.30	2.80	1.80	1.00	3.10	3.40	3.50	3.60
ELEVATION OF DIKE CREST (EL. m)	12.40	12.70	12.80	13.10	13.30	13.50	13.70	14.00	14.10	14.40	14.70	14.70
DESIGN HIGH WATER LEVEL (EL. m)	11.26	11.53	11.70	11.96	12.07	12.38	12.65	12.83	12.91	13.24	13.62	13.66
ELEVATION OF MAJOR BED (EL. m)	9.5	9.6	9.7	9.6	10.0	10.8	11.9	13.0	11.0	11.0	11.1	11.1
RIVER BED (EL. m)	1.4	-0.4	[illegible]	4.0	4.2	4.7	2.8	0.5	5.6	5.7	5.9	4.3
ACCUMULATION DISTANCE (Km)	12.00	12.65	13.00	13.65	14.00	14.65	15.00	15.35	15.65	16.00	16.65	16.90
DISTANCE (Km)	0.35	0.65	0.35	0.65	0.35	0.65	0.35	0.35	0.30	0.35	0.65	0.25
STATION	No.12	+0.65	No.13	+0.65	No.14	+0.65	No.15	+0.35	+0.65	No.16	+0.65	+0.90

LONGITUDINAL DISTANCE: L:900m / L:800m; L:1,200m / L:750m; L:2,000m / L:2,200m; L:650m / L:1,400m; L:1,150m / L:900m; L:350m / L:1,000m; L:600m / L:0m; L:750m / L:450m; L:200m / L:500m; L:500m / L:900m; L:1,650m / L:550m; L:1,350m / L:750m; L:250m / L:1,000m; L:3,500m / L:3,150m; L:1,450m / L:1,800m; L:100m / L:250m

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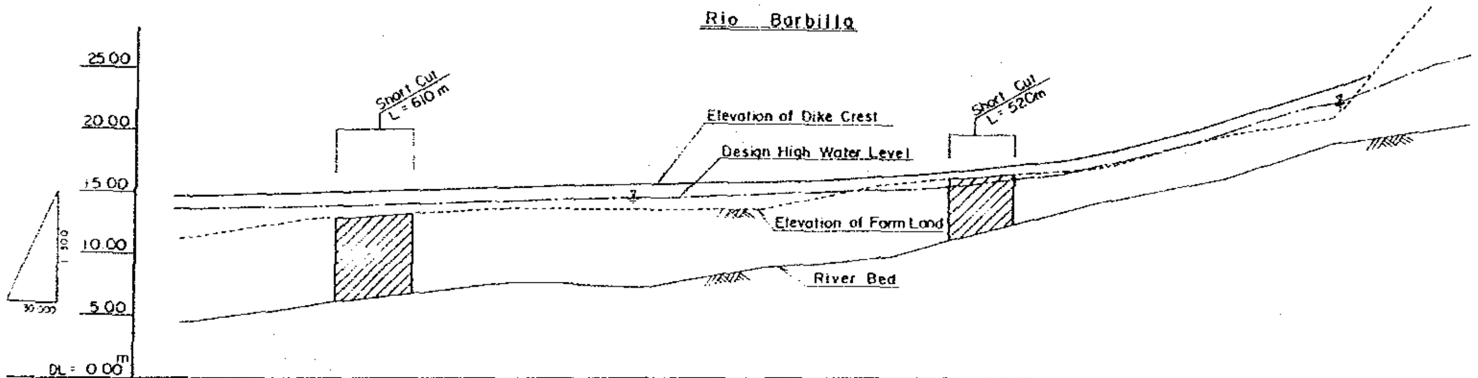
CONTENT	River Training Plan of Rio Matina
SCALE / DATE	No. 10

Figure F.5.2(2) CASE II Profile of Proposed Flood Protection Facilities (Rio Matina, Rio Barbilla, Rio Chirripo)

Rio Barbilla

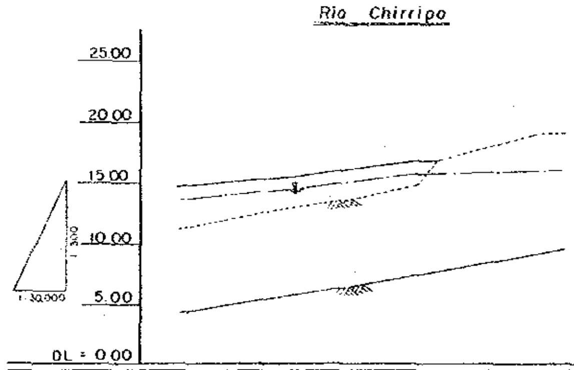


DESIGN RIVER LOCATION (m)	130	135	130	160	160	160	150	160	110	110	130
HEIGHT OF DIKE (m)	3.60	2.40	2.30	1.90	1.60	1.80	2.00	2.10	[illegible]	1.60	0.60
ELEVATION OF DIKE CREST (EL. m)	14.70	15.00	15.00	15.30	15.40	15.60	15.70	15.90	[illegible]	16.40	16.80
DESIGN HIGH WATER LEVEL (EL. m)	13.66	13.84	13.86	14.14	14.40	14.51	[illegible]	14.78	15.0	15.2	15.55
ELEVATION OF MAJOR BED (EL. m)	11.1	12.6	12.7	13.4	13.8	13.8	13.7	13.8	[illegible]	14.6	15.0
RIVER BED (EL. m)	4.3	5.6	5.8	7.0	7.8	7.7	7.4	8.0	[illegible]	9.4	10.0
ACCUMULATION DISTANCE (Km)	0.00	0.90	1.00	2.00	2.65	3.00	3.65	4.00	4.65	5.00	5.65
DISTANCE (Km)	0.00	0.90	0.10	1.00	0.65	0.35	0.65	0.35	0.65	0.35	0.65
STATION	No.0	+0.9	No.1	No.2	+0.65	No.3	+0.65	No.4	+0.65	No.5	+0.65

DESIGN RIVER LOCATION (m)	130	130	130	130	130	120	120	131	150	150
HEIGHT OF DIKE (m)	0.50	0.90	1.20	1.00	1.30	2.10	2.30	0	-	-
ELEVATION OF DIKE CREST (EL. m)	16.80	18.00	18.30	20.30	21.40	23.30	24.00	23.00	-	-
DESIGN HIGH WATER LEVEL (EL. m)	15.77	16.95	17.30	19.30	20.35	22.30	22.65	23.83	25.80	26.34
ELEVATION OF MAJOR BED (EL. m)	16.3	17.1	17.3	19.3	20.1	21.2	21.9	23.0	30.7	34.2
RIVER BED (EL. m)	11.0	13.7	14.3	16.0	16.6	18.8	19.4	19.8	20.5	20.9
ACCUMULATION DISTANCE (Km)	6.00	7.00	7.30	8.00	8.30	9.00	9.20	9.50	10.00	10.30
DISTANCE (Km)	0.35	1.00	0.30	0.70	0.30	0.70	0.20	0.30	0.50	0.30
STATION	No.6	No.7	+0.3	No.8	+0.3	No.9	+0.2	+0.5	No.10	+0.3

LONGITUDINAL DISTANCE: L:2,800m / L:2,550m; L:1,000m / L:1,050m; L:1,400m / L:1,300m; L:950m / L:1,000m; L:1,050m / L:950m; L:1,000m / L:1,050m; L:1,000m / L:1,050m; L:500m / L:500m

Rio Chirripo



DESIGN RIVER LOCATION (m)	130	130	130	130	165	360	360
HEIGHT OF DIKE (m)	3.60	2.60	2.50	2.00	0	-	-
ELEVATION OF DIKE CREST (EL. m)	14.70	15.50	16.20	16.60	16.60	-	-
DESIGN HIGH WATER LEVEL (EL. m)	13.66	14.53	15.10	15.73	15.74	15.94	16.01
ELEVATION OF MAJOR BED (EL. m)	11.1	13.0	13.7	14.8	16.0	19.0	19.0
RIVER BED (EL. m)	4.3	5.6	6.5	7.4	7.7	9.1	9.5
ACCUMULATION DISTANCE (Km)	0	1.00	1.50	2.00	2.15	3.00	3.20
DISTANCE (Km)	0	1.00	0.50	0.50	0.15	0.85	0.20
STATION	No.0	No.1	+0.5	No.2	+0.15	No.3	+0.2

LONGITUDINAL DISTANCE: L:1,050m / L:950m; L:1,050m / L:1,050m

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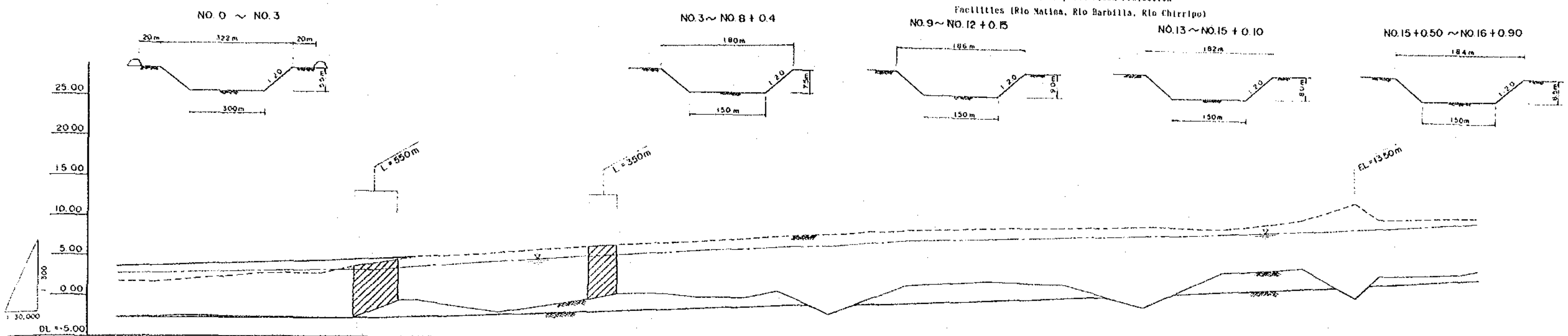
CONTENT	River Training Plan of Rio Barbilla and Rio Chirripo
SCALE / DATE	No. 11

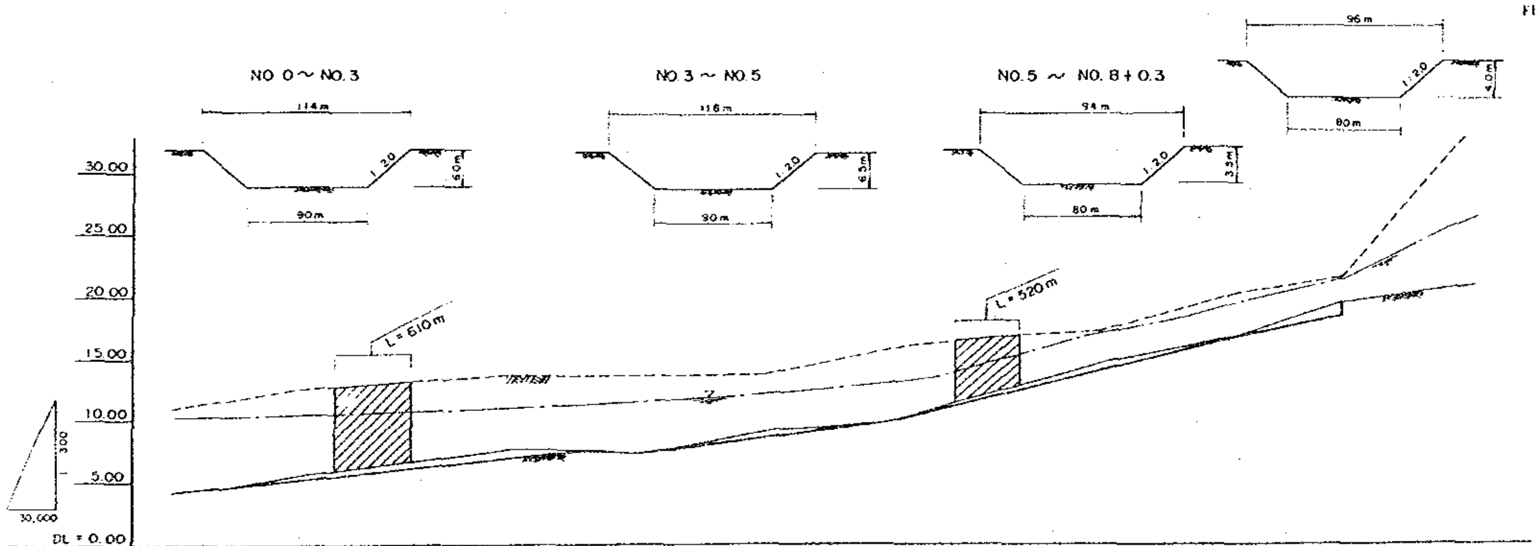
Figure F.5.3(1) CASE II Profile of Proposed Flood Protection Facilities (Rio Matina, Rio Barbilla, Rio Chirripo)



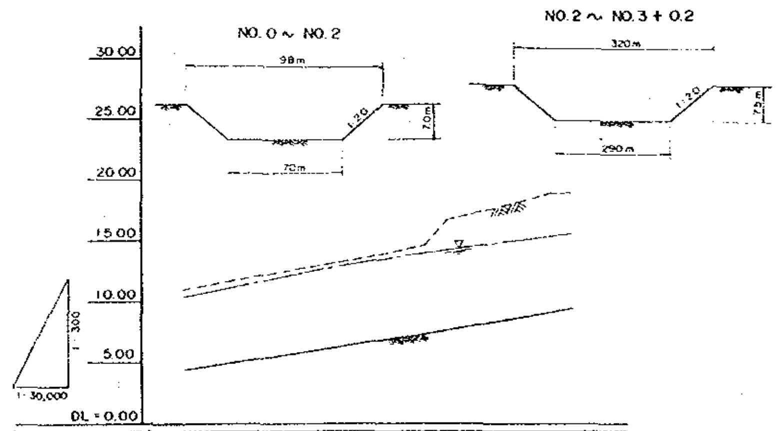
	No.0	+0.5	No.1	+0.5	No.2	+0.5	No.3	+0.65	No.4	+0.65	No.5	+0.65	No.6	+0.65	No.7	+0.65	No.8	+0.40	+0.65	No.9
LONGITUDINAL SLOPE	I=1/15,000							I=1/3,000												
DESIGN HIGH WATER LEVEL	2.56	2.71	2.85	2.98	3.11	3.23	3.31	3.86	4.12	4.55	4.77	5.14	5.33	5.66	5.83	6.14	6.30	6.47	-2.10	6.80
DESIGN RIVER BED	-2.90	-2.87	-2.83	-2.80	-2.77	-2.73	-2.70	-2.48	-2.37	-2.15	-2.03	-1.82	-1.70	-1.48	-1.37	-1.15	-1.03	-0.90	-2.10	-0.80
GROUND LEVEL	1.7	1.7	2.1	2.5	2.7	2.8	3.9	4.9	5.1	5.5	5.6	6.3	6.4	6.8	7.0	7.4	7.6	7.8	8.0	8.2
RIVER BED	2.9	-2.7	-2.7	-2.7	-2.7	-2.7	(3.9)	-0.6	-1.1	-2.0	-1.4	-0.4	(6.4)	0.4	0.1	0	0.7	-0.9	-2.1	0.8
ACCUMULATED DISTANCE	0	0.50	1.00	1.50	2.00	2.50	3.00	3.65	4.00	4.65	5.00	5.65	6.00	6.65	7.00	7.65	8.00	8.40	8.65	9.00
DISTANCE	0	0.50	0.50	0.50	0.50	0.50	0.50	0.65	0.35	0.65	0.35	0.65	0.35	0.65	0.35	0.65	0.35	0.40	0.25	0.35
STATION	No. 0	+0.5	No. 1	+0.5	No. 2	+0.5	No. 3	+0.65	No. 4	+0.65	No. 5	+0.65	No. 6	+0.65	No. 7	+0.65	No. 8	+0.40	+0.65	No. 9

	+0.65	No.10	+0.65	No.11	+0.65	No.12	+0.15	+0.65	No.13	+0.65	No.14	+0.65	No.15	+0.10	+0.35	+0.50	+0.65	No.16	+0.65	+0.90
LONGITUDINAL SLOPE	I=1/2,000								I=1/2,000							I=1/2,000				
DESIGN HIGH WATER LEVEL	7.38	7.50	7.74	7.87	8.12	8.26	8.32	8.51	8.56	8.67	9.02	9.31	9.47	9.68	9.64	9.68	9.72	9.89	10.20	10.32
DESIGN RIVER BED	-0.48	-0.30	0.03	0.20	0.53	0.70	0.78	-0.40	1.20	1.53	1.70	2.03	2.20	2.45	0.90	2.45	2.53	2.70	3.03	3.15
GROUND LEVEL	8.6	8.8	9.1	9.2	9.4	9.5	9.6	9.8	9.7	9.6	10.0	10.8	11.9	12.2	13.0	12.0	11.0	11.0	11.1	11.1
RIVER BED	1.7	1.9	2.4	2.4	2.3	1.4	0.8	-0.4	1.3	4.0	4.2	4.7	2.8	2.4	0.9	2.5	3.6	3.7	3.9	4.3
ACCUMULATED DISTANCE	9.65	10.00	10.65	11.00	11.65	12.00	12.15	12.65	13.00	13.65	14.00	14.65	15.00	15.10	15.35	15.50	15.65	16.00	16.65	16.90
DISTANCE	0.65	0.35	0.65	0.35	0.65	0.35	0.15	0.50	0.35	0.65	0.35	0.65	0.35	0.10	0.25	0.15	0.15	0.35	0.65	0.25
STATION	+0.65	No. 10	+0.65	No. 11	+0.65	No. 12	+0.15	+0.65	No. 13	+0.65	No. 14	+0.65	No. 15	+0.10	+0.35	+0.50	+0.65	No. 16	+0.65	+0.90

Figure F.5.3(2) CASE II Profile of Proposed Flood Protection Facilities (Rio Matina, Rio Barbilla, Rio Chirripo)



	No.0	+0.9	No.1	No.2	+0.65	No.3	+0.20	+0.65	No.4	+0.65	No.5	+0.65	No.6	No.7	+0.30	No.8	+0.3	No.9	+0.2	No.10	+0.3
LONGITUDINAL SLOPE	I=1/1,000							I=1/770						I=1/400			I=1/500				
DESIGN HIGH WATER LEVEL	10.32	10.46	10.49	10.75	11.02	11.19	11.30	11.54	11.71	12.16	12.47	13.15	13.60	16.40	16.94	18.57	19.32	20.92	21.31	25.20	26.52
DESIGN RIVER BED	4.30	5.20	5.30	6.30	6.95	7.30	7.50	7.40	7.85	8.70	9.15	10.00	10.88	13.38	14.15	15.88	16.63	18.00	18.40	20.30	20.90
GROUND LEVEL	11.1	12.6	12.7	13.4	13.8	13.8	13.4	13.7	13.8	13.9	14.6	16.0	16.3	17.1	17.3	19.3	20.1	21.2	21.5	30.7	34.2
RIVER BED	4.3	5.6	5.8	7.0	7.8	7.7	7.5	7.4	8.0	9.1	9.4	10.0	11.0	13.7	14.5	16.0	16.6	18.8	19.4	20.8	20.9
ACCUMULATED DISTANCE	0	0.90	1.00	2.00	2.65	3.00	3.20	3.65	4.00	4.65	5.00	5.65	6.00	7.00	7.30	8.00	8.30	9.00	9.20	10.00	10.30
DISTANCE	0	0.90	0.10	1.00	0.65	0.35	0.20	0.45	0.35	0.65	0.35	0.65	0.35	1.00	0.30	0.70	0.30	0.70	0.20	0.80	0.30
STATION	No. 0	+0.9	No. 1	No. 2	+0.65	No. 3	+0.20	+0.65	No. 4	+0.65	No. 5	+0.65	No. 6	No. 7	+0.30	No. 8	+0.3	No. 9	+0.2	No. 10	+0.3



	No.0	No.1	+0.5	No.2	No.3	+0.2
LONGITUDINAL SLOPE	I=1/650					
DESIGN HIGH WATER LEVEL	10.32	12.44	13.28	14.09	15.53	15.63
DESIGN RIVER BED	4.30	5.80	6.60	7.40	9.10	9.30
GROUND LEVEL	11.1	13.0	13.7	14.8	19.0	19.0
RIVER BED	4.3	5.8	6.6	7.4	9.1	9.5
ACCUMULATED DISTANCE	0	1.00	1.50	2.00	3.00	3.20
DISTANCE	0	1.00	0.50	0.50	0.85	0.20
STATION	No. 0	No. 1	+0.5	No. 2	No. 3	+0.2

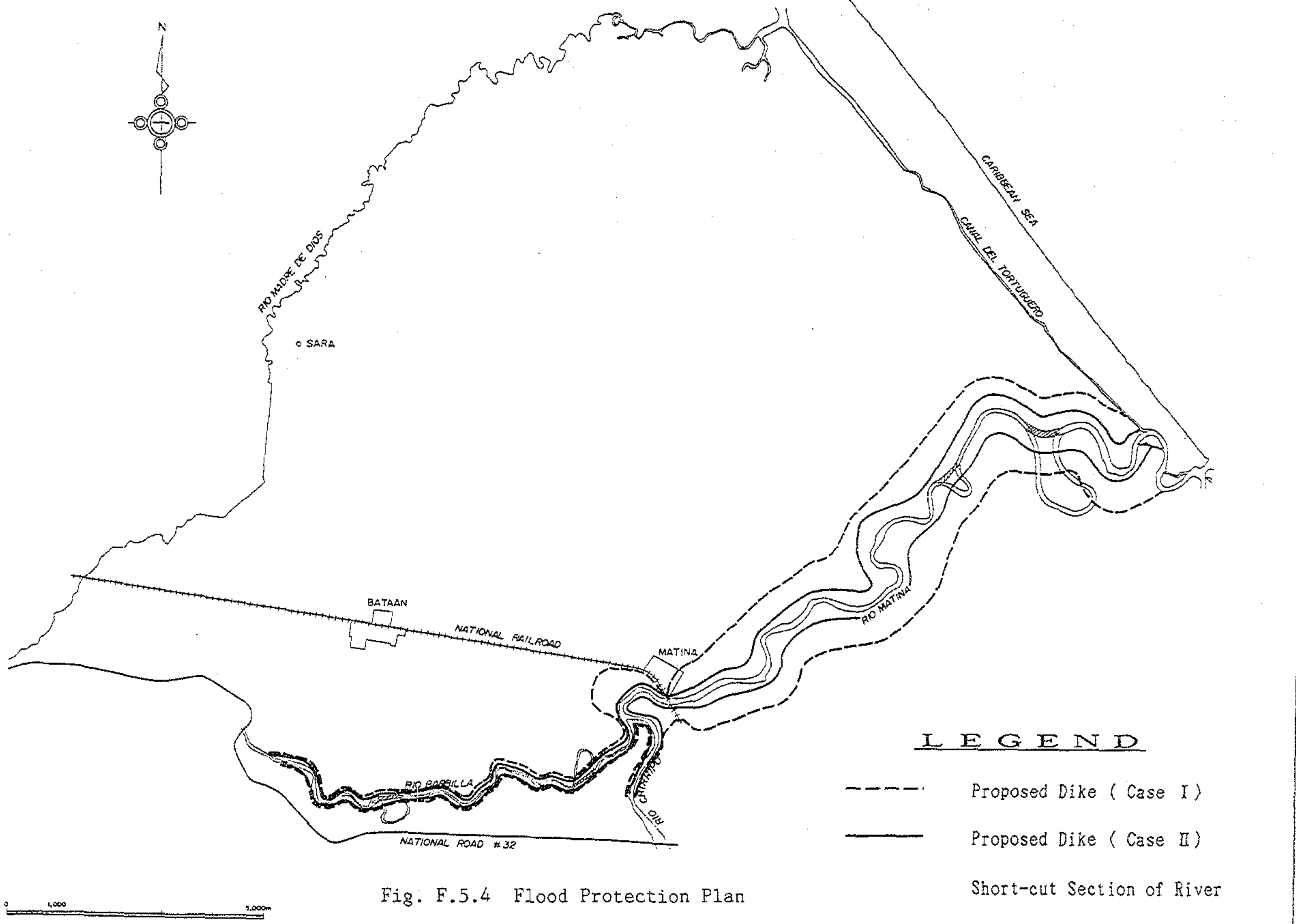
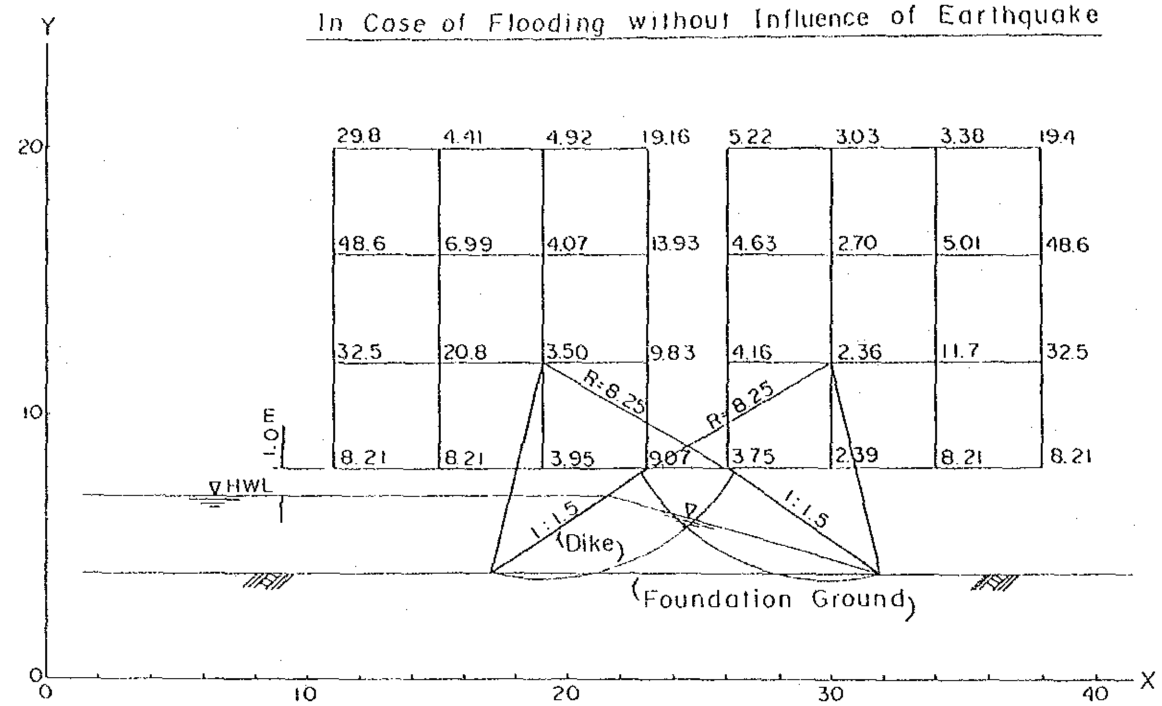


Fig. F.5.4 Flood Protection Plan



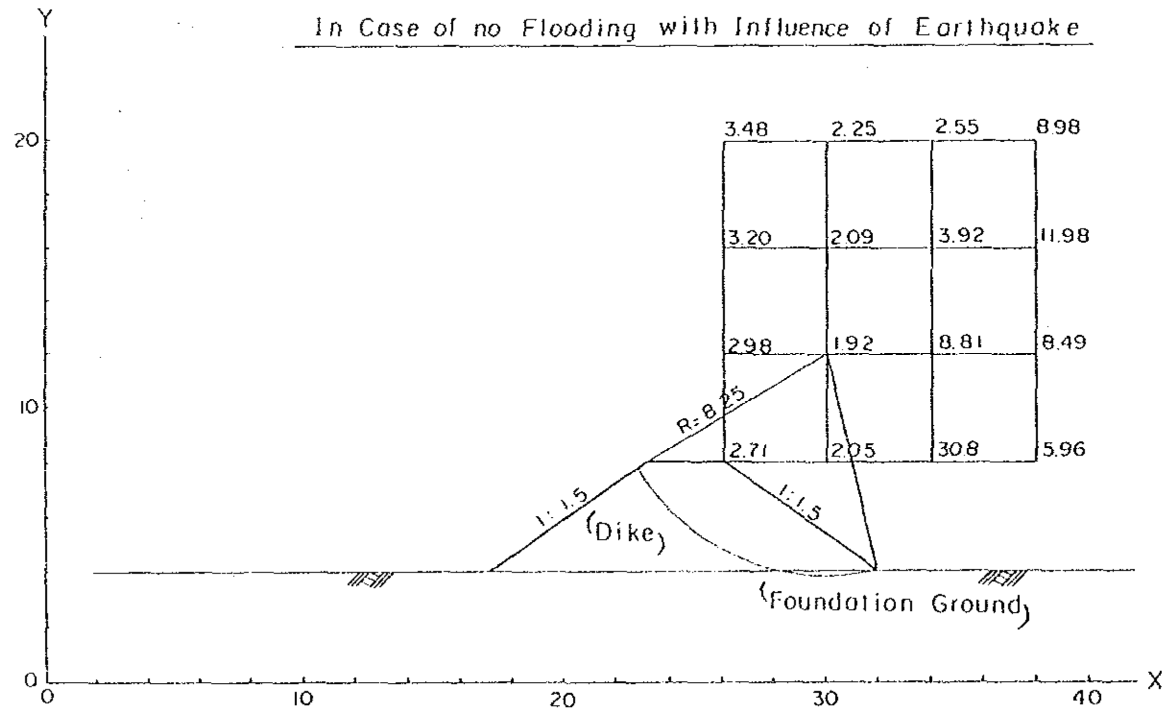


Fig. F.5.5. Stability Analysis for the Dike

Annex G Agricultural Land Consolidation

Annex G. Agricultural Land Consolidation

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G.1 Land Consolidation

G.1.1 General

The land consolidation is performed mainly for the main roads and trunk roads. In an area for which drainage facility is poor, though it has main roads or trunk roads, the land consolidation has not been performed. This area is used just as natural grassland or forestry land.

The land consolidation consists of two parts: Reconsolidation of a land consolidated as a banana plantation, excluding an existing banana plantation, and consolidation of an unconsolidated area located on the lower banana plantation.

G.1.2 Present Situation

1) This area has many agricultural land arranged from banana plantations around company or association banana plantations. Typical examples are viewed in Sara district and Good Hope Sur district.

(1) Sara district

The district has been improved comparatively well. Most of land have the shape of a rectangle. Almost land, excluding some blocked places, are provided with agricultural roads. The land currently used is positioned in an area drains wells. It accounts for approx. 40% of this district. The remaining approx. 60% is not cultivated and covered with natural grass due to poor drainage condition. Such area is often found in the southern district. Grass grows in this area because a natural drainage system is changed artificially accompanying the installation of the main roads. Reconsolidation of the drainage system including roads is needed.

(2) Good Hope Sur District

The district is located between the company banana plantation and association banana plantation. The land in third district has been consolidated in the same way as Sara district, but the drainage system is incomplete. Few part is used for a rice crop. Most of the district is covered with natural grass land.

There are many settlers employed by banana plantations. Since an income from a banana plantation is higher than that from a rice crop, they do not want to plant agricultural products.

Since the district is suitable for a banana plantation from a viewpoint of soil properties and its location, consolidation to a banana plantation is needed.

2) Generally, approx. 1,000 to 2,000 ha is provided with only one trunk road. Since a drainage of this area is dependent on natural rivers, if a flood occurs, this area is damaged by water flown from the higher banana plantation area. Agricultural land is not consolidated at all. Fruit cultivation and rice crop are performed in an area drains well along a trunk road. Most of low swamp along a trunk road are covered with natural grass land or forestry land. Example is viewed in North Bataan district (including Goschen, Santa Marta, and Damasco.)

(1) North Bataan District (including Goschen, Santa Marta and Damasco.)

North Bataan district is located on the lower company banana plantation. The land is not adjusted and each plot is not uniform. Generally, a plot that has poor drainage is large. Distribution of the land to settlers is completed.

Since each plot is not adjusted at all, almost all distributed land, excluding such plot that is connected to a main or trunk road, has no admission passage. Therefore, many settlers leave their distributed land as natural grassland or forestry.

The main and trunk roads are installed in this district. The land along these roads are consolidation to the some extent. However, it is limited to places that drain well.

Idea of land consolidation, centering on the arrangement of plots to which disposal of replace is applied because of the following two reasons, does not meet the actual conditions of land consolidation: First reason is that a plot is not arranged at all, and the second reason is that the land estimation standard is hard to set. Land consolidation, including the installation of agricultural roads to connect each plot and using a road side ditch for improving drainage, centering on the existing plotting is needed.

G.2 Land Consolidation Plan

G.2.1 General

The agricultural land continuity is lost at the end of the land by warm-eaten use of the land. It makes it hard to install agricultural roads needed for fields. This resulted from that the drainage system is not established.

No substantial difference is viewed between a consolidated land and a land that is not consolidated in the above condition.

First of all, an emphasis was laid on the arrangement of a drainage canal. Arrangement of an agricultural road was limited to irreducible minimum roads needed for a settler to enter his own distributed land easily.

G.2.2 Design Criteria

1) Drainage canal improvement area

a) Tertiary and lateral canal

Drainage canals are sorted and criteria is determined according to the scale of farmland.

Type 1 : $A \leq 0.5$ Km^2
Type 2 : 0.5 Km^2 $< A \leq 1.0$ Km^2
Type 3 : 1.0 Km^2 $< A \leq 1.5$ Km^2
Type 4 : 1.5 Km^2 $< A \leq 2.0$ Km^2
Type 5 : 2.0 Km^2 $< A \leq 2.5$ Km^2
Type 6 : 2.5 Km^2 $< A \leq 3.0$ Km^2
Note : A is a drainage area

The above types apply to the teritary and lateral canals.

Drainage canals list is as follows;

Table G.2.1 List of Drainage Canal

Type	Drainage area (km^2)	Bottom width (m)	Top width (m)	Depth of canal (m)	Water depth (l)	Canal slope (m3/sec)	Drainage Capacity (m^3/s)	Runoff Qs discharge (m3/ of surface sec)	Remarks
1	0.5	1.2	4.20	1.50	1.20	1/1,500	2.2	1.8	* C.A = 4.05 S.F = 4.23
2	1.0	1.5	5.5	2.0	1.70	1/1,500	3.7	3.6	* C.A = 7.00 S.F = 5.64
3	1.5	2.2	6.2	2.0	1.70	1/1,500	5.5	5.4	* C.A = 8.60 S.F = 5.64
4	2.0	2.5	6.9	2.2	1.90	1/1,500	7.5	7.2	* C.A = 10.36 S.F = 6.20
5	2.5	2.7	7.5	2.4	2.10	1/1,500	9.6	9.0	* C.A = 12.24 S.F = 6.77
6	3.0	3.0	8.0	2.5	2.2	1/1,500	11.3	10.8	* C.A = 13.75 S.F = 7.05

* C.A: Excavation (m^3/m)
S.F: Slope length (m /m)

b) Drainage ditch

Drainage ditch is classiffied bu the scale of the farmland, as shown follow;

Type 1: $A \leqq 0.10\ km^2$

Type 2: $0.10\ km^2 < A \leqq 0.20\ km^2$

Type 3: $0.20\ km^2 < A \leqq 0.30\ km^2$

Type 4: $0.30\ km^2 < A \leqq 0.40\ km^2$

Note : A is a drainage area

Table G 2.2 Drainage Ditch

Type	Drainage area (km^2)	Bottom width (m)	Top width (m)	Depth of canal (m)	Water depth (l)	Canal slope (m3/sec)	Drainage Capacity (m^3/s)	Runoff Qs discharge (m3/ of surface sec)	Remarks
1	0.1	0.6	2.6	1.0	0.80	$\frac{1}{2,000}$	0.45	0.4	* C.A : 3.06 S.F : 3.53
2	0.2	0.9	3.1	1.1	0.90	$\frac{1}{2,000}$	0.83	0.8	* C.A : 1.60 S.F : 2.82
3	0.3	1.2	3.7	1.25	1.05	$\frac{1}{2,000}$	1.22	1.2	* C.A : 2.20 S.F : 3.10

* C.A: Excavation (m^3/m)
S.F: Slope length (m /m)

2) Agricultural road improvement standard

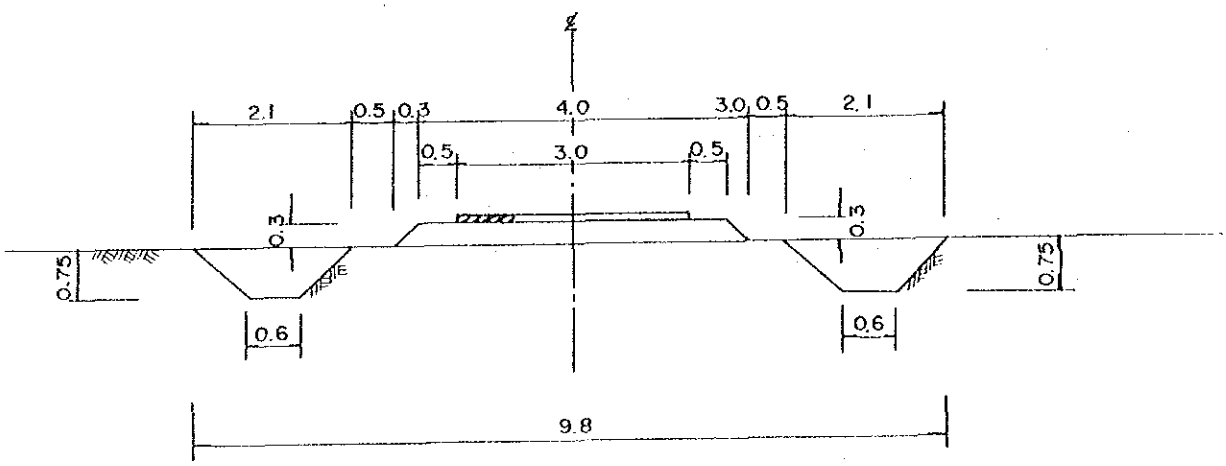
a) Road Structure

The road face is paved by sandy gravel. The effective width and reclamation width are 3.0 m and 4.0 m. The road face is higher than the land by 0.3 m. Material generated on the spot is road bed for the roadbed materials in principle.

b) Road side ditch

0.3 to 0.5 m is enough for the road side ditch depth functionally. However, if the agricultural roads are crossed, the culvert work covered with soil is little and it may be destroyed. To avoid destruction and to improve drainage around there, the road side ditch depth and bottom width must be 0.75 m and 0.60 m.

The end of the road side ditch is connected to the existing canal or drainage canal.



Banking Unit Quantity	:	1.29 m3/m
Side Ditch Excavation	:	2.02 m3/m
Slope Finishing Work	:	4.23 m3/m
Gravel Work	:	0.90 m3/m (two times of gravel work)

Fig. G.2.1 Standard Layout of Farm Road

3) <u>Land consolidation level</u>

(1) Drainage canal improvement level

The drainage density in an area where a former banana plantation is used as an agricultural land is approx. 45 km/1000 ha. Therefore, the target drainage canal density is determined to be 40 to 50 km/1,000 ha for the drainage canal improvement level.

(2) Road improvement level

The ideal road improvement level is that the road density is 60 km/1000 ha whether an area is consolidated or not.

The extension of exsisting road in already consolidate area goes approx. 60 km/1,000 ha. However, it is hard to upgrade the road improvement level unless the drainage canal improvement level is upgraded. Therefore, the target road improvement level is assumed to be 20 to 30 km/1,000 ha.

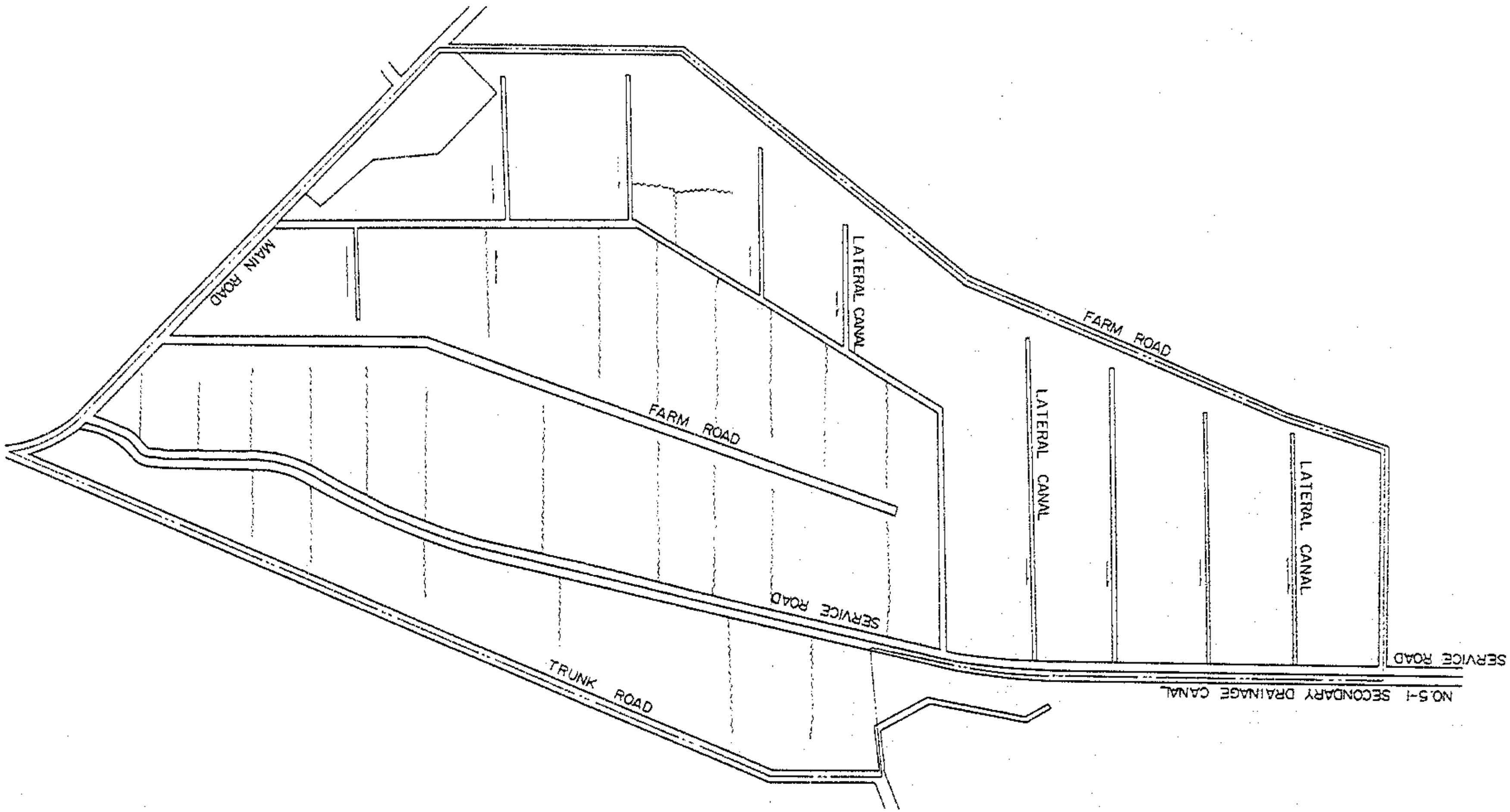


Fig. G.2.2(1) Land Consolidation Plan - Sara Area

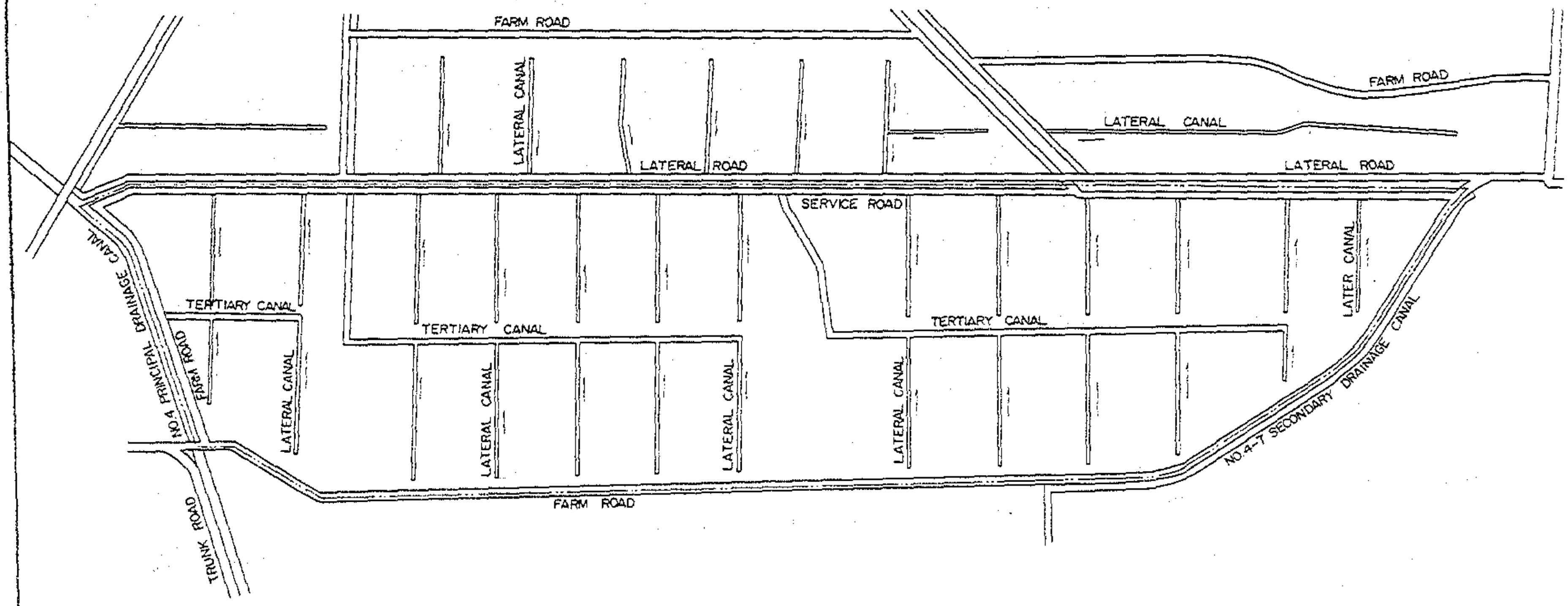


Fig. G.2.2(2) Land Consolidation Plan - Bataan Norte Area

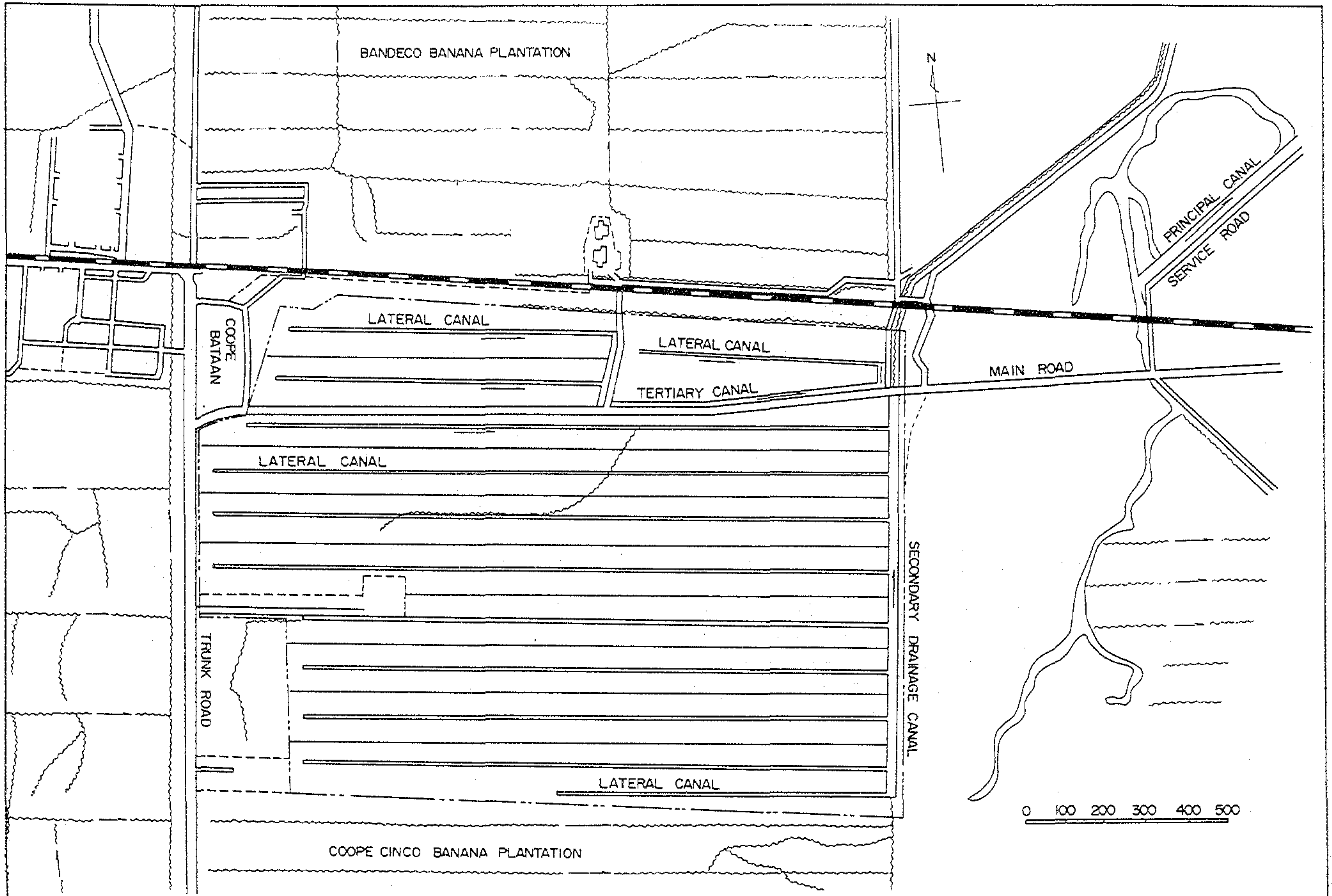


Fig. G.2.2(3) Land Consolidation Plan - Good Hope Sur Area

Annex H Agricultural Economy and Marketing

Annex H. Agricultural Economy and Marketing

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H.1 Demand and Supply of Agricultural Products

FAO has forecasted the foodstuffs production and supply on the estimated growth rate of population and GNP. The world food demand was calculated at 2.5 percent growth rate in 1970's and 1980's.

The foodstuff demand growth rate was estimated at 1.6 percent in developing countries, while 3.3 percent in developed countries per annum. The supply growth rate was estimated at 2.4 percent in developing countries, while 2.6 percent in developed countries per annum. The analysis by the FAO for 1970's shows the trend of demand and supply, but the estimated factor will be changed largely by the change of the premise in family income level and population growth rate. Especially, farm products other than grain as staple food, such as tiqisque, plantain, etc. are taken over to a substitution crops in case of shortage in production and supply to the market. Therefore, future demand including the international market demand would be quite difficult to forecast. Future supply also can not be known for certain tubercrops and vegetables and fruits except for staple foods like rice and wheat. The agricultural production prices for the future level would be most uncertain in respect of social and economic situation.

Meanwhile, due to the chronical poverty and unemployment in the world, FAO distinguishes the world food demand from the problem for supplementary food needed to improve the nutritional condition of those who do not benefit from economic growth.

Actual grain consumption and future demand for 1970-1990 estimated by FAO is shown in Table H.2.8.11; while commodity consumption per capita for Costa Rica in 1985 is shown in Table.H.2.8.12.

H.2 Export Trends of Cacao and Tubercrops

According to the census of 1983-86, the Costa Rican cacao export quantity has gained about 3.5 times of 2,596 metric tons in 1986 compared with 736 metric tons in 1983. In France, there was a sudden rise in export of cacao since 1985 to 1986, while the export market for cacao to West Germany remained constant. Japan, on the other hand, has the biggest export market of cacao since 1983. (See Table.H.2.8.13)

Export quantity of tubercrops was increased up to 10,500 metric tons in 1986 from 7,800 metric tons in 1983 or about 34 percent. The highest export country is to Puerto Rico which is remarkable export of 312 percent, following England, 61 percent. A large number of export is occupied by the United States, with 75 percent of the total export amount, followed by England, Holland, 5-6% each. (See Table H 2.8.7).

According to the above data, the future potential demand of export market shows an increase in European countries, however, the quantity of export will be largely occupied by U.S. market, particularly in Miami and New York markets. The world cacao and tubercrops actual production and consumption data is shown in Table H.2.8.14 and 2.8.15

H.3 Costa Rican Export Agricultural Production

The production, domestic consumption and export quantities in 1985 are shown as follows:

(Unit: metric tons)

	Production	Domestic Consumption	Exported	Exported Ratio (%)
Cacao	4,451	3,184	1,267	28.5
Tubercrops	72,006	63,800	8,206	11.4
Banana	1,007,889	151,889	856,000	84.9

Source: Cifras de produccion Agropecuaria de Costa Rica (Cacao, Banana), and Tabulados de Exportacion y PIMA (tubercrops).

In the Study area, major export crops production of banana was 52,600 metric tons or 2.9 million boxes and tubercrops including vegetables fruits of 2,000 metric tons, and most of cacao were exported or in use to produce finished chocolate. The ratio of those products in the Study area were estimated at about 12 percent (cacao), 24 percent (tubercrops and vegetable fruits) and 6 percent (banana).

H.4 Marketing Season Demand and Quality Standard in the United States

The peak season of demand of the agricultural production in the United States is fixed yearly. The amount of supply should be more than the quantity of demand, however, marketing of agricultural products is limited during harvest season in each producing district. The production season for tubercrops and vegetable fruits in the United States is from July to September in summer at the western state of California, and all the year-round production in the Miami State. The Miami State has generally produce tubercrops, and vegetable fruit on a small quantity. Mainly produce such being the case in United States, the wholesale market prices fluctuates in winter season off-crop season in September to late spring season in June. A fluctuating graph is shown in Figures. H.3.8.5 and 3.8.6.

The handling goods of agricultural production in Miami and New York market are importing constantly from Caribbean countries, middle and southern American countries and also domestic production state. There is no space in receiving of goods, but sometimes handling a small quantity of products of other cereals. The other cereal crops production such as tubercrops and vegetable fruits can be changed to other crops, if the production is inadequate.

The farmers in Costa Rica should learn the producing technology of good quality because the products to be marketed in the modern U.S. market system are required to meet the U.S. quality standards. In general Costa Rican exporters should pack commodities based on the U.S. grades and standards. These standards are available not only in the U.S. but also in the market which produces benefit such as European countries.

H.5 Price Analysis of Different Market Trends

In the price analysis of the different market trends, the data on the wholesale prices at New York and Boston were used. Generally, trading in Boston market is higher than in New York market as shown in the following table and the graph in Figure H.3.8.6 and 3.6.7.

Different Wholesale Prices

(Unit: Average US$/tons)

	Boston	Difference(%) (between Boston and New York)	New York Market
Chayote	10.5	(50)	7.5
Banana	12.0	(20)	10.0 (5-9 months)
Coconut (Fresh)	15.0	(20)	12.5
Yucca	20.0	(0)	20.0

The prices of yucca in Boston and New York markets are the same, but much fluctuation curve line are indicated on a graph every month in New York market. Miami and New York are mainly imported of Costa Rican's yucca. In the future, however, Boston market will become the major export of yucca.

European markets export only some plantain. Tubercrops and vegetable fruits are supplied from eastern Europe, northern Africa and EC products itself. Costa Rican profitable and marketable export places are located in United States such as Miami, New York and Boston, which are centers of the north-eastern American and Canada.

H.6 Post-Harvest Cost (F.O.B)

Generally, post-harvest cost consists of the production cost, post-harvest expenses and transportation charges. However, in this section, post-harvest cost and domestic transportation charges were only included.

The post-harvest cost for banana was calculated based on the JAPDEVA 1986 data as shown below. The processing capacity is 600,000 boxes per year. Therefore, the estimated processing and packing cost is about 62 colones per box (18.14 kg). The domestic transportation cost to Moin port is 7 colones per box. The total post-harvest cost require 69 colones per box in 1987.

Item	Annual Cost (1,000 colones)	Cost/box (18.14 kg) (colones)	Cost/kg (colones)
1. Capital repayment	812	1.35	0.07
2. Employee wage	3,420	5.70	0.31
3. Processing and packing cost	25,200	42.00	2.32
4. Direct expenses	1,500	2.45	0.14
5. Administration cost	1,000	1.70	0.09
Total	31,932	53.20	2.93

Source: JAPDEVA, December 1986

As indicated above, the December 1986 data was used, however, a price escalation of 16.43% has been considered during the study period in December 1987. The following table shows the post-harvest cost:

Unit: colones/box (18.14 kg)

Item	1986	1987/*
1. Post-harvest cost (Processing, packing)	53.20	61.94
2. Transportation to Moin Seaport	7.00	7.00
Total	60.20	68.94

/* .. including 5% damage loss.

Banana export price in FOB Moin was US$3.90 per box (18.14 kg) in 1987; thus, the post-harvest cost was 268 colones per box at an exchange rate of US$1.00 to colones 68.75. Finally, the post-harvest cost ratio accounted to 25.7 percent based on F.O.B. Moin prices. The detailed post-harvest ration in 1987 is shown below:

Cost Item	Expenses (Colones/Box)	Cost Ratio (%)
FOB Moin (US$3.90/box)	268	100
Producer's price	199.06	74.3
Post-harvest Cost (included transportation)	68.94	25.7

Cacao post-harvest cost was also analyzed in the same manner as the post-harvest cost of banana. The harvested cacao seed fermentation and drying cost is 10.8 colones per kilogram. The farm gate price of the dry-up cacao seed are about 90-100 colones per kg, therefore, the post-harvest cost ratio of the dried seed is 10.8 percent.

The post harvest cost of the tubercrops and vegetable fruits go to expenses 4.3 colones per kilogram, that is 3.3 colones for processing and 1.0 colones for transportation. Showing table H.2.8.6 are wholesale price, that price indicated 25 colones per kilogram. (It is equal to export price and wholesale price). That is 17.2 percent of post-harvest cost ratio.

H.7 Supporting Services for the Farmer (Roles of the INFOCOOP)

The INFOCOOP conducted to the farmer's supporting of the cooperatives organization. Therefore, the activities are operation and guidance to the cooperatives groups, improvement of the group management technics, guidance activities according to the cooperatives low, cooperate with the parties or groups and other related concerned. The INFOCOOP has not works to grouping or establishment in direct. In case of the establishment to the new cooperatives, INFOCOOP made for following supervision take the position of operation and guidance.

1) Cooperatives Establishment

The new establishment of cooperatives will depene on the Government policy, social and economic development in the area, the refore, the cooperative establishment planner of the rural society, cooperative and other organizations. The following sectors of the promoter were divided:

- The government authorities concerned

Establish the cooperatives based on the government policies.

- The company incorporate

In case of the both sides, employer and employees wants to be incorporated.

- Rural group

The rural farmer's group who wants to initiate a cooperative.

- Others

Upon requested by the other parties or groups.

2) The INFOCOOP's Inquiry

The requesting group should submit the feasibility report of the new cooperatives. INFOCOOP will, however, study the feasibility report which was prepared by the requesting group.

They will also collect the data and information through a questionnaire form. At the same time, the request will be confirmed, confirmation of information at the Ministry of Social Labor (Census and Statistic Dept.), IDA, IMAS, and make a field survey, carry out an inquiry, and confirm the following items:

- What kind of cooperatives?

- Which type of the organization, and what kind of services would be made?

- INFOCOOP duty itself of the central cooperatives organization management.

- Check a possibility of the cooperative.

3) Inspection and Field Survey

When cooperatives are established, INFOCOOP should inspect and conduct field survey of the following items:

- Proposed cooperatives which are based on the national and/or regional plan, and give impact of new cooperatives on society as regional.

- Feasibility report and Survey Team's opinion.

- Based upon the provincial policy and maintain harmony between related concerns and new cooperatives.

- Type and member of the cooperatives.

- Proper scale of the business.

- Adequate capital funds, etc.

4) Necessary Data for the Cooperatives Establishment

Submit new cooperatives plans to INFOCOOP of the following:

- The articles of the incorporation.

- The minutes of the cooperatives' meeting.

- Feasibility report of the organization and new cooperatives utility survey report.

- Report of the initial fund (more than 255 assurance of the total fund).

- Record of the selected members for the cooperatives member, managing staffs and other concerned members.

- Membership list, a membership fee, and share fund of the new cooperatives.

- Registration of the new cooperatives with the Ministry of Insurance and Labor.

H.8 Workshop Equipment

H.8.1. Chassis Repair Equipment

Item	Description	Quantity
1.	Rigid rack	4 sets
2.	Chassis lubricator	1 unit
3.	High pressure grease pump	1 unit
4.	Oil bucket pump	1 unit
5.	Drum carrier	1 unit
6.	Gas cutting tool & regulator set	1 unit
7.	Hand truck	1 unit
8.	Tool cabinet	3 units
9.	Hand tool set (without box)	3 units
10.	Work bench with vice	3 units
11.	Wire rope set	2 sets
12.	Hydraulic garage jack	1 unit
13.	Cylinder carrier	1 set
14.	Parts washing stand	1 unit

Item	Description	Quantity
15.	Sling chain set (single type)	1 set
16.	Bench drilling machine	1 unit
17.	Electric grinder	1 set
18.	Tool rack	1 set
19.	Straight shank drill set	5 sets
20.	Taper shank twist drill set	1 set
21.	Cut-off machine	1 unit
22.	Portable gantry crane	1 unit
23.	Air impact wrench (1 sq.)	1 set
24.	Hydraulic test gauge	1 set
25.	Volume pump	1 set
26.	Grease gun	3 pcs
27.	Micro hose, 340 mm chuck type	3 pcs
28.	Drum pump	2 pcs
29.	Drum opening spanner	1 pc
30.	Oil Measure, 2 liter	3 pcs
31.	Oil measure, 4 liter	3 pcs

32.	Funnel, 200 mm	2 pcs
33.	Portable fuel can	3 pcs
34.	Electric cord reel	2 sets
35.	Cleaning pan	10
36.	Portable hydraulic jack	2
37.	Portable hydraulic jack	2
38.	Hand operated pump	1 unit
39.	Cylinder, 100 ton	1 unit
40.	Cylinder, 70 ton	1 unit
41.	Puller, 50 ton	1 unit
42.	Bearing & Gear Puller Set for construction machinery	1 set
43.	Adjustable spanner wrench	1 pc
44.	Adjustable wrench, giant type	1 pc
45.	Adjustable wrench, giant type	1 pc
46.	Master pin service tool	1 pc
47.	Sprocket remover & installer	1 unit

H.8.2 Engineer Repair Equipment

Item	Description	Quantity
1.	Engine stand	1 set
2.	Diesel timing tachometer	1 set
3.	Diesel engine vacuum tester	1 set
4.	Stop watch	2 pcs
5.	Compression gauge (diesel)	1 set
6.	Thermometer	5 pcs
7.	Timing light	1 set
8.	Compression gauge (gasoline)	1 set
9.	Plug cleaner tester	1 set
10.	Air blow gun	3 pcs
11.	Engine cleaning gun	3 pcs
12.	Solderless terminal kit	2 sets
13.	Battery filler, 4 liter	1 set
14.	Valve Revacer	1 set
15.	Eccentric valve seat grinder	1 set
16.	Valve lifter & compressor	1 pc

17.	Valve spring tester	1 set
18.	Piston ring tool	1 pc
19.	Hand valve lapper 30 mm dia	10 pcs
20.	Hand valve lapper 25 mm dia	10 pcs
21.	Hand valve lapper 20 mm dia	10 pcs
22.	Piston ring compressor	1 pc

H.8.3. BATTERY SERVICE EQUIPMENT

Item	Description	Quantity
1.	Battery hydrometer set	2 sets
2.	Battery tester	1 unit
3.	Quick and normal (silicon)	1 unit
4.	Booster cable	1 pc
5.	Charging cable	3 pcs
6.	Electric soldering iron (100 w)	2 pcs
7.	Solder (1 kg/roll)	3 pcs
8.	Circuit tester	1 unit
9.	Work bench with vice 5 inch	1 set

10.	Parts rack	1 set
11.	Tool cabinet	1 unit
12.	Sling chain set (single type)	1 set
13.	Sound scope	1 pc
14.	Air hose reel (8 m hose)	1 unit
15.	Hand too set (without box)	1 set
16.	Torque wrench (general type)	1 set
17.	Diesel nozzle tester	1 set
18.	Cleaning pun: 300 x 400 mm	10 pcs
19.	Plug wrench set	1 set

H.8.4. <u>Tire Service Equipment</u>

Item	Description	Quantity
1.	Air hose reel	1 unit
2.	Tire pressure gauge (bar type)	2 pcs
3.	Hydraulic tire removing tool	1 set
4.	Tire service tool set	1 set
5.	Tube vulcanizer	1 unit
6.	Tire bead breaker	1 unit

7.	Automatic tire inflator	1 unit
8.	Wheel dolly	1 unit
9.	Air compressor	1 unit
10.	High power wrench	1 set
11.	Brake booster tester	1 set
12.	Brake adjusting wrench (4 pcs/set)	1 set
13.	Air impact wrench (1/2 sq.)	1 set
14.	Tire pressure gauge	2 pcs
15.	Tire pressure gauge	2 pcs
16.	Air chuck	2 pcs
17.	Air chuck (Jambo valve type)	2 pcs
18.	Hot patch (Round ∅ 23)	10 sets
19.	Hot patch (diamond 43 x 33)	10 sets
20.	Hot patch (oval 47 x 23)	10 sets
21.	Double face sledge hammer	3 pcs
22.	Valve tool	5 pcs
23.	Valve repair tool	5 pcs
24.	Valve repair tool for jambo valve	5 pcs

25.	Brake drum gauge	1 set
26.	Brake compression test gauge	1 set
27.	Brake shoe adjusting tool	1 set
28.	Brake spring plier, 700mm	1 set
29.	Brake spring plier, 515mm	1 set
30.	Brake spring plier, 330mm	1 set

H.8.5. Welding & Fabrication Equipment

Item	Description	Quantity
1.	Gas cutting tool & regulator set	2 set
2.	Electric grinder (bench type)	1 unit
3.	Arc welder	1 unit
4.	Hydraulic press	1 unit
5.	Cast iron anvil	1 unit
6.	Disc grinder (Wheel dia 100 mm)	3 units
7.	Disc grinder (Wheel dia 180 mm)	3 units
8.	Body fender tool set	1 set
9.	Body puller with accessories	1 set
10.	Spray gun & cup with hose 10m	2 sets

11.	Torch lamp	1 set
12.	Double-face sledge hammer	2 pcs
13.	Surface plat	1 set

H.8.6. Washing Equipment

Item	Description	Quantity
1.	Steam cleaner	1 unit
2.	High pressure water washer	1 unit

H.8.7. General Tools

Item	Description	Quantity
1.	Torque multipliers	1 pc
2.	Torque wrench (torque pre set ratchet type)	1 set
3.	Outside micrometer caliper	1 pc
4.	Vernier caliper	1 set
5.	Magnetic base (reference: support angle fixed)	1 pc
6.	Dial indicator	1 pc
7.	Square (flat type)	1 set
8.	V-block (B-type)	1 set

9.	Firm-joint caliper (for inside)	1 set
10.	Firm-joint caliper (for outside)	1 set
11.	Surface gauge	1 set
12.	Stainless steel rule (straight type)	1 set
13.	Steel tape measure	1 set
14.	Iron bench level	1 set
15.	Cylinder gauge (carl-mahr type)	1 set
16.	Hand tachometer (contact type)	1 pc
17.	Socket wrench set (inch size)	1 set
18.	Double offset box wrench set	1 set
19.	Double offset wrench	1 set
20.	Open end wrench set	1 set
21.	Single end wrench	1 set
22.	Hexagon wrench set	1 set
23.	Adjustable wrench	1 set
24.	Cross driver	1 set
25.	Stubby crew driver	1 set
26.	Ball peen hammer set	1 set

27.	Chisel set	1 set
28.	Engineers files set	2 sets
29.	Hacksaw frame-blade (12 pcs/set)	10 sets
30.	Hand taps (3 pcs/set)	2 sets
31.	Pipe taps set	2 sets
32.	"T" type wrench set	1 set
33.	Gear puller, effective thickness of max.	1 set
34.	Electric soldering iron	1 pc
35.	Solder (for electric)	1 pc
36.	Portable hydraulic jack	2 units
37.	Grease gun	2 pcs
38.	Lever block	1 unit
39.	Chain block	1 unit
40.	Bearing & gear puller set	1 unit
41.	Adjustable spanner wrench set	2 sets
42.	Steel shelf	3 sets

H.9 Cacao Fermentation and Drying Facilities Plan

H.9.1 Cacao Drying House

1) Function

To prevent the cacao dryer equipment from damage by rain, wind and other climatical conditions.

2) Features

1. Heavy construction for log durability.

2. Constructed by structural steel and anti-crrosive corrugated steel sheet..

3. Inside floor level is higher than the ground level to prevent rain from intruding.

4. Equipped with approach or access at the door for loading material easily.

5. Equipped with windows at all sides for environmental hygiene.

6. Inside floor is made by minium 150 mm thickness concrete and is durable for installing dryer equipment.

H.9.2 Cacao Fermentation Box

1) Function

To hold cacao seed temporarily and ferment it inside of the box.

2) Features

1. All steel made for long durability.

2. Consists of three stage-fermentation boxes, one for initial fermentation and others for next fermentation.

3. Equipped with discharge spouts for next process.

4. Discharge spouts have guides to prevent material from scattering during transporting to the next process.

5. Equipped with stairs and platforms with enough width to load material easily.

6. Inside of the boxes are done by anti-corrosive treatment.

H.9.3 Cacao Dryer

1) Function

To dry up the cacao seed safely by heated air and strong wind.

2) Features

(1) Drying box

a) All steel construction for long durability.

b) Be able to hold more than 2 metric ton seed.

c) Drying chamber is considered for enough air-tight.

d) Material of inside tray is made of performated aluminum sheet for anti-corrosion.

e) Equipped with a round thermometer indicating the inside temperature of drying chamber.

f) Equipped with discharge spout to unload easily.

(2). Burner

a) Steel made for long durability

b) Having more than 100,000 Kcal/hr combustion performance.

c) Burner is a fixed jet pressure fuel atomizing type.

d) Equipped with automatic ignition device and regulator of fuel supply at range of 12-16 litre/hr.

e) Be able to stop combustion within 2 second after switching off.

(3) Fan

a) Steel made for long durability.

b) Centrifugal limit load type and considered for low noise and vibration.

c) Having more than 5 m^3/s air volume at 100 mm Aq static pressure.

d) Bearing are treated for temperature not rising high.

e) Furnished with safety cover on the rotating portion.

f) Driven by engine and V-belts.

(4) Diesel Engine

a) Steel construction for long durability.

b) Water-cooled 4-cycle diesel engine.

c) Having more than 22 HP continuous output.

d) Equipped with electric starting device by cell starter and with clutch for starting easily.

e) Coupled to the blower by V-belts.

f) Furnished with 2 KVA generator for supplying electricity to the burner. Generator has two connector, one for burner and other for lighting of house inside.

g) Equipped with safety cover for rotating portion and high temperature portion.

(5) Moisture Tester

a) The moisture tester has a built-in scale for weighing 150 grain samples.

b) Testing is simply operated by push-button (calibrations for up to seven grains, commodities or seeds to be tested can be programmed).

c) The current moisture percentage automatically appears in lighted figures.

d) Specifications: 390 mm high x 310 mm wide x 160 mm deep power 115 Volt AC current 60 cycle. Also available for optional battery operation.

Table H.2.8.1 Banana Production and Shipment in the Study Area (1986 and 1987)

Name of Farm	Plantation area (ha)	Exported Boxes (1986)	Exported Boxes (1987)	Harvested Boxes/Ha (1986)	Harvest Boxes/Ha (1987)
Bananita	236	694.700	663.700	2.944	2.813
Monte Libano	313	755.400	844.300	2.472	2.682
Milca	136	219.700	253.300	1.620	1.643
Coope Bataan	286	500.600	500.500	1.944	1.983
Coope Cinco	155	252.000	306.200	1.792	1.982
Luzon	204	476.600	519.100	2.343	2.542
TOTAL	1.330	2.899.000	3.147.100	2.180	2.366

Source: JICA Study Team

Table H.2.8.2 Banana New York Market Price(1980-1985)

Unit: U. S. Dollar/Box (18.14kg)

Year / Month	1980	1981	1982	1983	1984	1985
1	9.38	9.22	10.28	9.48	9.93	10.55
2	9.37	10.25	10.75	10.24	11.14	11.38
3	10.41	10.96	11.25	11.11	11.47	12.50
4	10.45	11.10	11.25	12.92	11.36	13.11
5	10.84	11.67	10.88	14.50	10.81	12.30
6	9.32	10.54	10.75	14.06	11.35	11.43
7	9.10	9.09	9.00	13.17	11.44	10.37
8	9.04	9.00	n.d.	12.05	9.98	10.08
9	9.40	9.90	n.d.	12.17	10.7	10.03
10	8.92	10.00	n.d.	11.14	9.99	9.49
11	9.44	n.d.	n.d.	9.47	8.50	9.46
12	9.32	n.d.	9.37	9.20	8.96	9.12
Average	9.58	10.17	10.44	11.63	10.45	10.82

Source: New York Fruit and Vegetable Report, U. S. Department of Agriculture

Table H.2.8.3 Banana Export and Domestic Consumption (1976-1985)

Item / Year	Export				Consumption				Total			
	Volume (M. Ton)	Ratio (%)	Unit Price (Colones)	Value (Million Colones)	Volume (M. Ton)	Ratio (%)	Unit Price (Colones)	Value (Million Colones)	Volume (M. Ton)	Ratio (%)	Unit Price (Colones)	Value (Million Colones)
1976	1,068,513	90.0	1,201	1,283.7	118,634	10.0	237	28.1	1,187,147	100	1,093	1,297
1977	1,002,916	89.2	1,280	1,283.7	121,775	10.8	234	28.5	1,124,691	100	1,167	1,312
1978	1,058,022	89.4	1,371	1,450.7	124,940	10.6	274	34.2	1,182,962	100	1,255	1,484
1979	1,024,670	88.8	1,588	1,626.0	129,655	11.2	284	36.8	1,154,325	100	1,441	1,663
1980	973,190	87.9	1,938	1,886.5	134,328	12.1	293	39.4	1,107,518	100	1,739	1,925
1981	1,002,000	87.9	4,631	4,640.3	139,290	12.2	344	47.9	1,141,290	100	4,108	2,688
1982	1,013,000	87.8	8,274	8,381.9	140,305	12.2	694	97.4	1,153,305	100	7,352	8,479
1983	1,012,000	87.6	9,485	9,599.0	143,355	12.4	987	141.5	1,155,355	100	8,431	9,740
1984	1,020,000	87.3	10,847	11,064.2	148,623	12.7	1,063	158.0	1,168,623	100	9,603	11,222
1985	856,000	84.0	12,487	10,688.5	151,899	15.1	1,155	175.4	1,007,889	100	10,779	10,863

Source: Cifras de Produccion Agropecuaria de Costa Rica 1976-1985

Table H.2.8.4 Cacao New York Market Price (1977/78-1984/85)

Unit : U. S. Dollar/46kg

Month \ Year	1977-78	1978-79	1970-80	1980-81	1981-82	1982-83	1983-84	1984-85
10	176.82	171.86	132.62	98.03	98.50	67.43	90.19	110.49
11	181.92	179.37	127.73	90.61	93.60	61.58	95.58	111.00
12	148.72	174.63	136.74	89.14	95.70	67.25	-	105.30
1	139.68	161.90	137.81	95.00	95.40	76.15	-	105.40
2	135.39	153.91	142.28	92.60	94.70	82.62	-	106.52
3	154.51	147.15	136.64	95.90	88.30	78.06	124.32	109.85
4	156.64	140.63	127.85	95.50	79.20	79.50	121.89	113.33
5	141.81	147.27	112.21	87.70	77.80	88.61	128.58	110.34
6	137.92	148.18	106.57	75.30	71.20	97.65	117.35	109.17
7	144.74	136.77	105.35	91.90	70.60	96.96	106.43	111.68
8	153.84	132.00	95.99	101.80	71.00	98.13	110.72	114.56
9	169.00	140.94	101.58	106.30	76.40	92.28	116.71	116.84
Average	153.42	152.80	121.95	93.32	84.37	82.19	112.42	110.37
(per kg.)	(3.34)	(3.32)	(2.65)	(2.03)	(1.83)	(1.79)	(2.44)	(2.40)

Source: Organizacion del Estado Americana/The Wall Street Journal

Table H.2.8.5 Cacao Export and Domestic Consumption (1976-1985)

Item	Export				Consumption				Total			
Year	Volume (M. Ton)	Ratio (%)	Unit Price (Colones)	Value (Million Colones)	Volume (M. Ton)	Ratio (%)	Unit Price (Colones)	Value (Million Colones)	Volume (M. Ton)	Ratio (%)	Unit Price (Colones)	Value (Million Colones)
1976	4,180	71.0	14,211	59.4	1,675	29.0	13,612	22.8	5,855	100	14,039	82.2
1977	5,627	73.0	26,000	146.3	2,067	27.0	27,334	56.5	7,694	100	26,358	202.8
1978	5,842	56.0	2,030	138.7	4,529	44.0	24,124	109.5	10,381	100	22,946	238.2
1979	4,235	41.0	19,599	83.0	6,130	59.0	21,958	134.6	10,365	100	20,994	217.6
1980	2,165	41.0	17,644	38.2	3,101	59.0	20,574	63.8	5,266	100	19,370	102.0
1981	2,024	40.0	35,25	71.7	3,025	60.0	3,504	107.4	5,049	100	35,453	179.0
1982	2,024	57.0	44,022	89.1	1,522	43.0	59,593	90.7	3,546	100	50,677	179.7
1983	708	33.0	55,367	39.2	1,453	67.0	82,657	120.1	2,161	100	73,716	159.3
1984	843	20.0	80,902	68.2	3,296	80.0	105,006	346.1	4,139	100	100,072	414.2
1985	1,267	28.5	85,083	107.8	3,184	71.5	98,210	312.7	4,451	100	94,473	420.5

Source: Cifras de Produccion Agropecuaria de Costa Rica 1976-1985

Table H.2.8.6 Received Tiquisque and Whole Sales Prices at San Jose Market (1981-1986)

Handling : Metric Tons/Month
Prices : 1,000 Colones/Ton

Year	Month	1	2	3	4	5	6	7	8	9	10	11	12	Total	Growth Rate
1981	Handling at Market	-	-	-	-	19.78	21.76	25.21	31.14	36.57	38.02	27.04	29.30	228.81	100
	Standard Price	-	-	-	-	2.6	2.6	2.5	2.4	2.6	2.6	2.7	2.85	2.6	100
1982	Handling at Market	34.91	26.86	17.62	12.83	18.81	19.87	12.47	20.47	13.71	10.49	28.38	22.79	239.22	104
	Standard Price	3.05	3.25	3.25	4.35	4.35	4.9	8.7	8.7	13.05	12.5	13.05	10.85	3.25	125
1983	Handling at Market	20.29	2.25	6.16	13.94	17.62	18.31	8.83	9.52	17.39	17.25	17.76	10.53	159.85	069
	Standard Price	11.95	9.8	10.0	9.8	9.8	9.8	15.0	15.0	18.0	19.55	20.0	18.0	9.80	376
1984	Handling at Market	22.47	16.64	21.97	31.37	19.96	27.60	17.49	33.40	31.50	36.91	40.48	33.60	333.39	146
	Standard Price	18.0	18.0	14.0	12.0	12.0	12.0	12.0	10.0	10.0	13.0	15.0	15.0	12.0	461
1985	Handling at Market	42.96	24.61	30.95	45.54	38.92	23.37	15.24	16.09	20.06	9.34	20.97	9.10	297.15	130
	Standard Price	15.0	15.0	15.0	15.0	15.0	15.0	15.0	20.0	20.0	20.0	20.0	25.0	15.0	576
1986	Handling at Market	13.01	11.39	10.49	19.36	15.00	17.07	20.29	17.15	23.78	24.11	21.88	23.34	216.85	094
	Standard Price	20.0	20.0	20.0	20.0	20.0	20.0	25.0	25.0	25.0	25.0	25.0	25.0	25.0	961

Source: Departmento Tecnico PIMA