ブラジル連邦共和国

スアッペ臨海工業開発計画

資料No. 2

VOLUME IV

PART 7 - FLOOD CONTROL SYSTEM

2

TOMO II
PARTE 1
VOLUME 5

PART 7 - FLOOD CONTROL SYSTEM

1.0 IPOJUCA SYSTEM

1.1 PRELIMINARIES

Because of its influence over the area of the CI, the problem of the floods of the River Ipojuca was given special consideration by the Consultant. Due to the extension of its hydrographic basin - over 3.500 km^2 - almost 1/5 of which is in the Littoral and Mata zones, where heavy rains predominate, the river Ipojuca shows a potential of flood discharges that demands the provision of control measure.

A study was made of various possibilities, combining the desirable solutions with the respective costs, in order to reach a viable solution.

The possibilities studied included studies of control dams, canals for the interconnection of basins, improvements of the waterways and an amortizing lake close to the CI, with the respective dams, spillways and flood-gates.

Besides the regulation of floods, through the limitation and the determination of the discharges into the feeder canals to the CI, the project also aimed at the retention of the sediments carried by the water, especially during floods, so as to protect the port and it navigable waterways from silting up because of river discharges.

1 - IPOJUCA SYSTEM - FLOOD CONTROL - POSSIBILITIES

1 - Conventions

2 - Canal axis

1.2 CHARACTERISTICS OF THE FLOOD CONSIDERED IN THE PROJECT

Through the use of synthetic methods, the hydrological studies led to discharges estimated at 1.072 m^3/sec. and 1.780 m^3/sec, respectively, for recurrence period of 100 to 1.000 years, at Engenho Maranhão, about 15km from the CI, no affluent of any importance existing between these points.

1.2.1 RAINS CONSIDERED IN THE PROJECT

The rains considered in the project were determined through the use of the Grimbel-method, and considering the upstream area, divided into 3 sub-basins, with the following characteristics

SECTION CONSIDERED	AREA KM^2	LENGTH OF BOTTOM LINE Km	ELEVATION SUP	ELEVATION INF	DIFFERENCE IN LEVEL Km	SLOPE %
I- Tacohbó	1.550	120	670	570	300	0,25
II-Riacho Vertonto	1.362	113	570	390	180	0,16
III-Engonho Maranhão	516	50	390	55	335	0,67
TOTAL	3.450	283	670	55	615	0,28

The series of maximum daily precipitations were measured at the Arcoverde Station for sub-basin I, at the Caruarú Station for sub-basin II and at the Escada Station for sub-basin III, the following results having been obtained:

STATION	SUB-BASIN	DAILY AVERAGE 100 YEAR PERIOD	DAILY AVERAGE 1.000 YEAR PERIOD
ARCOVERDE	I	15,5	23,1
CARUARU	II	12,5	18,7
ESCADA	III	22,5	33,6

In order to obtain a synthetic hydrogram, a uniform precipitation was assumed for the whole of the basin, its value being that of the weighted averageof the above value $\frac{(\varepsilon P \times A)}{\varepsilon A}$, that is:

Recurrence period of 100 years: 13,50 cm

Recurrence period of 1000 years: 18,30 cm

1.2.2 RAIN DISTRIBUTION CONSIDERED IN THE PROJECT

Using the recommended methodolgy for Hydrological Research, Jayme Taborga (1974) obtained the following distribution for the rains:

100 year period

Hours	% accum.	εP	εq	q
0 - 6	69	9.3	2.1	2.1
6 - 12	84	11.3	3.3	1.2
12 - 18	93	12.5	4.1	0.8
18 - 24	100	13.5	4.8	0.7
24 - 36	111	15.0	5.8	1.0
36 - 48	122	16.5	6.8	1.0

1000 year period

Hours	% accum.	εP	εq	q
0 - 6	69	12.6	4.1	4.1
6 - 12	84	15.4	6.1	2.0
12 - 18	93	17.0	7.3	1.2
18 - 24	100	18.3	8.3	1.0
24 - 36	111	20.3	9.3	1.5
36 - 48	122	22.3	11.5	1.7

1.2.3 ELEMENTS OF THE UNITARY HYDROGRAPH

The formulae used were:

$$Tc = 0,39 \left(\frac{L^2}{S}\right) 0,885$$

$$T_p = \frac{\Delta T}{2} + 0,6\ Tc$$

$$T_p = 2,67\ T_p$$

$$q_p = 2,08\ \frac{A}{T_p},\quad \text{where}$$

Tc - Concentration time, in hours

L - Length of the Bottom Line, in km

S - Slope, im %

Tp - Peak time, in hours

Δt - Interval considered (6 hours)

Tb - Basic time - in hours

qp - Discharge per cm of effective rain, in m^3/sec/cm

The following values were obtained:

$Tc = 49,5$ h

$T_p^6 = 33,0$ h

$T_p^{12} = 36,0$ h

$T_b^6 = 88,0$ h

$T_b^{12} = 96,0$ h

$q_p^6 = 217.$ m^3/seg/cm.

$q_p^{12} = 199$ m^3/seg/cm.

Thus, the following peak discharges were obtained, and used in the composition of the hydrograms:

Peak	100 years	1000 years
Qo - 6	456,00	889,70
Q6 - 12	260,00	434,00
Q12 - 18	174,00	260,00
Q18 - 24	152,00	217,00
Q24 - 36	217,00	298,00
Q36 - 48	217,00	338,00

The respective hydrograms are on the following drawings.

2 - IPOJUCA SYSTEM - DAM B1 - FLOOD HYDROGRAM - TR 100 years

1) - Cubic meters per second

2) - Hours

3 - IPOJUCA SYSTEM - ENG. MARANHÃO - FLOOD HYDROGRAM - TR 1.000 YEARS

1) - Cubic meters per second

2) - Hours

1.3 MAPPING AND TOPOGRAPHICAL CONDITIONS

The maps used were thoses of SUDENE, prepared by VASP, to the scale of 1:25.000, with a spacing of 10 meters between the contour curves.

Due to the contribution of some important affluents, in the Mata zone, it was determined that the control should be made downstream from the city of Escada, and eventual sites for a dam were investigated in the determined stretch. The nearness of the Ipojuca basin to the Mereque and Sibirõ basins in the egion was considered, and of the Sibirõ to the upper Serinhaem, with which it joins further downstream.

The inhabited elevations of the city of Escada, and those of highway BR-101 and the railroad limited the water level to elevation 75.

In the last phse of the work, aerial photographies taken by the FAB (Brazilian Air Force), in 1969 - 1970, to the scale of 1:30.000 were used, as well as others taken by Aerodata in 1974, to the scale of 1:5000.

1.4 POSSIBILITIES STUDIED

Possibilities were studied, based on dams at two wide points of the river Ipojuca: one, immediately upstream from Engenho Maranhão, and the other downstream from Engenho Crauassũ.

Thus, it was considered that possibilities A, A-1, B and B-1 deserved further analyses, and they are shown below:

POSSIBILITY A

In this case, the basis is a small dam, upstream from Engenho Maranhão, called Dam I; and the water level was considered at elevation 75, assumed to be a maximum, as menrioned in the preceding item. A canal, called Canal I, would divert the excess waters from the floods to an affluent of the Ipojuca itself; then, in sequence, Dam2 would dam this affluent, Canal 2 would divert the excess water from the floods to an affluent of the river Sibirõ, Dam 3 would dam this affluent, Canal 3 would carry the excess to the river Sibirõ that would in its tuern be dammed by Dam 4 and, finally, the excess waters would be diverted to the basin of the river Serinhaem. The characteristics of the areas, accumulated volumes, and the cost of the work mentioned are shown below.

Calculations of Areas and Volumes

Elevation m	Area $10^6 m^2$	Sum of Areas $\cdot 10^6 m^2$	Half-height m	Partial Volume $10^6 m^3$	ACCUMULATED VOLUME $10^6 m^3$
Basin 1					
80	6,65	9,56	5	47.800	68.050
70	2,91	3,48	5	17.400	20.250
60	0,57	0,57	5	2.850	2.850
Basin 2					
80	2,1	3,3	5	16.500	30.500
70	1,2	1,8	5	9.000	14.000
60	0,6	0,8	5	4.000	5.000
50	0,2	0,2	5	1.000	1.000
Basin 3					
80	3,57	5,74	5	28.700	64.550
70	2,17	3,48	5	17.400	35.850
60	1,31	2,09	5	10.450	18.450
50	0,78	1,19	5	5.950	8.000
40	0,11	0,41	5	2.050	2.050
Basin 4					
80	4,47	7,00	5	35.400	50.250
70	2,61	2,86	5	14.300	15.550
60	0,25	0,25	5	1.250	1.250

Quantitative

WORK	VOLUME OF EARTH Cu M		VOLUME OF STONE Cu M		DISPOSSESS ha.
	CUT	FILL	CUT	RIP-RAP- DRAIN	
Dam 1	-	277.900	-	6.347	665
Canal 1	196.067	-	94.029	-	-
Dam 2	-	628.260	-	19.500	336
Canal 2	964.290	-	413.267	-	-
Dam 3	-	1.913.490	-	70.560	357
Canal 3	41.874	-	17.946	-	-
Dam 4	-	256.380	-	12.180	447
Canal 4	58.881	-	25.234	-	-
T O T A L S	1.261.112	3.086.030	540.476	180.587	1.805

Cost

WORK	COST OF WORK	COST OF DISPOSSESSION	Total
Dam 1	1.884.000	997.500	2.881.500
Canal 1	1.927.600	-	1.927.600
Dam 2	4.351.000	504.000	4.885.000
Canal 2	9.480.000	-	9.480.000
Dam 3	13.500.000	535.500	14.035.500
Canal 3	411.700	-	411.700
Dam 4	1.850.500	670.500	2.522.000
Canal 4	579.000	-	579.000
SUMS	33.904.000	2.707.500	36.692.300

4 - IPOJUCA SYSTEM - DAM I - DIAGRAM OF AREAS AND VOLUMES

1) - Elevation in meters

5 - IPOJUCA SYSTEM . DAM 2 - DIAGRAM OF AREAS AND VOLUMES

1) - Elevation in meters

6 - IPOJUCA SYSTEM - DAM 3 - DIAGRAM OF AREAS AND VOLUMES

1) - Elevation in meters

7 - IPOJUCA SYSTEM - DAM 4 - DIAGRAM OF AREAS AND VOLUMES

1) - Elevation in meters

8- IPOJUCA SYSTEM - POSSIBILITIES A and A1

1) - Crest = 82,00

9 - IPOJUCA SYSTEM - POSSIBILITY A - DAM - 2

1) - Apex

10 - IPOJUCA SYSTEM - POSSIBILIDADE A E A1

1 - Apex

11- IPOJUCA SYSTEM - POSSIBILITY A and A1 - DAM 4

1) - Apex

12 - IPOJUCA SYSTEM - POSSIBILITY A and A1 - CANAL 1

LONGITUDINAL PROFILE

13 - IPOJUCA SYSTEM - POSSIBILITY A and A1 - CANAL 1

SECTION S'c1

14 - IPOJUCA SYSTEM - POSSIBILITY A and A1 - CANAL 1 - SECTION S"c1

15 - IPOJUCA SYSTEM - POSSIBILITY A and A1 - CANAL 1 - SECTION S''' c1

16 - IPOJUCA SYSTEM - POSSIBILITY A - CANAL 2 - LONGITUDINAL PROFILE

17 - IPOJUCA SYSTEM - POSSIBILITY A - CANAL 2 - Section S"c2

18 - IPOJUCA SYSTEM - POSSIBILTY A - CANAL 2 - SECTION S''' c2

19 - IPOJUCA SYSTEM - POSSIBILITY A - CANAL 2 - SECTION $S^{IV}c2$

20 - IPOJUCA SYSTEM - POSSIBILITY A -
CANAL 2 - SECTION $S^{V}c2$

21 - IPOJUCA SYSTEM - POSSIBILITY A and A1

CANAL 3 - LONGITUDINAL PROFILE

22 - IPOJUCA SYSTEM - POSSIBILITY A and A1

CANAL 3 - SECTION S'c3

23 - IPOJUCA SYSTEM - POSSIBILITY A and A1

CANAL 3 - SECTION S"c3

24 - IPOJUCA SYSTEM - POSSIBILITY A and A1

CANAL 3 - SECTION S'''c3

25 - IPOJUCA SYSTEM - POSSIBILITY A and A1 -
CANAL 4 - LONGITUDINAL PROFILE

26 - IPOJUCA SYSTEM - POSSIBILITY A and A1

CANAL 4 - SECTION S'c4

27 - IPOJUCA SYSTEM - POISSIBILITY A and A1
CANAL 4 - SECTION S"c4

28 - IPOJUCA SYSTEM - POSSIBILITY A and A1
CANAL 4 - SECTION S"'c4

POSSIBILITY A-1

The high cost of Canal 2 led to the study of this possibility, thát consists in damming the affluent of the Ipojuca further upstream with Dam 2 A-1: a smaller canal, designated as Canal 2 A-1 would deliver the excess water to the upper course of the river Merepe that would then be dammed by Dam 2 A-2; then a canal, alsosmall in size, called Canal 2 A-2, would overtake the watershed of the basin of the Sibirõ affluent, and from then on, the work would be the same as in possibility A. As we can see in the comparative table below, the replacement of Dam 2 and Canal 2 by dams 2 A-1 and 2 A-2 and canals 2 A-1 and 2 A-2 proves unfavourable.

Calculation of Areas and Volumes

Reservoirs 1, 3 and 4 are the same as in possibility A.

BASIN 2 A-1

Elevavation	Área $10^6 m^2$	Sums of Áreas $10^6 m^2$	Half-height m	Partial volume $10^6 m^3$	Accumulated volume $10^6 m^3$
80	3,35	5,55	5	27.750	61.300
70	2,20	3,52	5	17.600	33.650
60	1,32	1.97	5	9.850	16.000
50	0,65	0,94	5	4.700	6.150
40	0,29	0,29	5	1.450	1.450

BASIN 2 A-2

80	0,48	0,72	5	3.600	5.600
70	0,24	0,32	5	1.600	2.000
60	0,08	0,08	5	0,400	0,400

QUANTITIES

WORK	VOLUME OF EARTH m^3		VOLUME OF STONE m^3	
	CUT	FILL	CUT	RIP-RAP-DRAIN
Canal 2 Dam 2	964.290	628.260	413.267	19.500
Dam 2 A-1		1.565.512	-	45.120
Canal 2 A-1	357.815	-	153.349	-
Dam 2 A-2		320.920	-	12.500
Canal 2 A-2	194.187	-	83.223	-
Aux. Dam		21.540	-	1.260
Sum	552.022	1.907.972	236.572	59.180
Difference	-412.288	+1.279.712	-176.695	+39.680

COST

WORK	Cost in Cr$ 1,00
Canal 2	4.351.000
Dam 2	9.480.000
Sum	13.831.000
Canal 2 A-1	3.517.000
Dam 2 A-1	10.780.700
Canal 2 A-A	1.709.100
Dam 2 A-2	2.273.600
Aux. Dam	159.800
Sum	18.640.400
Difference	+4.809.400

29 . IPOJUCA SYSTEM - DAM IIA1 - DIAGRAM OF AREAS AND VOLUMES

1). - Elevation in meters

30 - IPOJUCA SYSTEM - DAM IIA2 - DIAGRAM OF AREAS AND VOLUMES

1) - Elevation in meters

31 - IPOJUCA SYSTEM - POSSIBILITY A1 - DAM 2A1 -

1) - Apex

32 - IPOJUCA SYSTEM - POSSIBILITY A1 - Dam 2 A2

1) - Apex

33 - IPOJUCA SYSTEM - POSSIBILITY A1 - CANAL 2A1 - Longitudinal Profile

34 - IPOJUCA SYSTEM - POSSIBILITY A1 - CANAL 2A1 - S'c2A1

35 - IPOJUCA SYSTEM - POSSIBILITY A1 - CANAL 2A1 - S"c2A1

36 - IPOJUCA SYSTEM - POSSIBILITY A1 - CANAL 2 A1 - S"' c2Á2

37 - IPOJUCA SYSTEM - POSSIBILITY A 1 - CANAL 2 A2 - Lonitudinal Profile

38 - IPOJUCA SYSTEM - POSSIBILITY A1 - CANAL 2 A2 - SECTION S'c2 A2

39 - IPOJUCA SYSTEM - POSSIBILITY A1 - CANAL 2 A2 - SECTION S'' c2 A2

40 - IPOJUCA SYSTEM - POSSIBILITY A1 - CANAL wA2 - SECTION "'c2 A2

POSSIBILITY B

Preliminarily, upon consideration of a dam downstream from Engenho Maranhão, on the site of Engenho Crauassú, the scope of this possibility is to replace by a single construction Dams 1 and 2 and Canal 1 of possibilility A. Surprisingly, even though the displacement downstream is small, a mere 4 km approximately, the intermediate basin proved quite receptive to an important water reserve. From the results of the water accumulation, it is shown that a reservoir created on this site, and operated with a reserve of about 20.000.000 m^3 at the beginning of a flood wave, even considering the the millenial flood, it couldcontain the whole vloume at elevation 75. Considering the high cost of construction, and assuming that comparatively high discharges could be allowed in the flume of the Ipojuca River, downstream from Dam B and a new control close to the CI (Amortizing Lake), possibility B-1 was studied.

POSSIBILITY B AND B-1

Calculation of Areas and Volumes

Ele-vation m	Aroa $10^6 m^2$	Sum of Areas $10^6 m^2$	Half-height	Partial volume $10^6 m^3$	Accumulated volume $10^6 m^3$
80	20,45	33,28	5	166.400	434.050
70	12,83	20,68	5	103.400	267.650
60	7,85	13,36	5	66.800	164.250
50	5,51	9,59	5	47.950	97.450
40	4,03	6,27	5	31.350	49.500
30	2,19	2,91	5	14.550	13.150
20	0,72	0,72	5	3.600	3.600

Possibility B

Quantities

WORK	FILL m^3	STONE m^3	CONCRETE m^3	DISPOSSESSION Ha.
Dam	8.789.107	264.000	-	2.045
Spillway	-	-	1.875	-

COST

Dam	Cr$	60.717.500,00
Spillway	Cr$	375.000,00
Dispossession	Cr$	3.067.500,00
TOTAL	Cr$	64.160.000,00

41 - IPOJUCA SYSTEM - POSSIBILITY B - DAM B

POSSIBILITY B-1

This possibility considered the allowableness of flows of about 500 m^3/sec in the flume downstream of Dam B and the ease of control of this flow before it enters the CI, at a low cost, through a lake formed by a dam (the future PE-9), spillway sills and sluices to regulate the flow of water. These works will be examined and analysed in another part of this study. Therefore, a trial was made, tentatively, to operate the reservoir, considering an open orifice that would flow constantly up to the predetermined discharge limit. Thus was reached the solution of a dam, whose water level would reach elevation 53.50 for the conditions of the flood hydrogram, 100 year recurrence period. The free-flow would be guaranteed by a pipe, 6 m in diameter, made of sheet iron covered concrete. In order to guarantee the flow of higher discharges, an emergency spillway was calculated, with its sill at elevation 53,50, capable of allowing the flow of the excess of the millenial flood with a water depth of 1.88 m. Correspondingly, steps were taken downstream to provide against the effects of the millenial flood.

Quantities

WORK	EARTH			STONE		DISPOSSESSION Ha.
	CUT	FILL	RECUP	CUT	RECUP	
Spillway	186.700	-	-	46.600	-	
Free descent	616.000	-	-	204.000	-	
Dam	-	1.317.034	802.000	-	250.000	785

Cost	
Excavating	7.368.300
Dam	9.236.500
Discharge Pipe	3.750.000
Dispossession	1.177.500
	21.532.500

42 - IPOJUCA SYSTEM - DAM B1 - PLAN AND SECTION

1) - Front view
2) - Apex
3) - Transverse Section AA_1
4) - Elevation

43 - IPOJUCA SYSTEM - POSSIBILITY B1 - DAM B1

1) - Longitudinal Profile

2) - Apex

3) - Transverse Section

44 - IPOJUCA SYSTEM - DAM B - DIAGRAM OF AREAS AND VOLUMES

1) - Elevation in meters

45 - IPOJUCA SYSTEM - DAM B1 - RIPPL'S DIAGRAM

A - Directing Floods to the Reservoir

Thw following procedure was adopted for the study:

The basic equation is:

$$\frac{I_1 + I_2}{2} - \frac{O_1 + O_2}{2} = \frac{V_2 - V_1}{\Delta t} \text{, where}$$

$I_m = \frac{I_1 + I_2}{2}$ average of affluent discharges in time inter- val. Δt.

O_1 - Affluent discharge through the funnel in the middle of the interval

O_2 - Same at the end of the interval

V_1 - Accumulated volume in the middle of the interval

V_2 - Same, at the end

The formula above can be written as follows:

$$I_m + \left(\frac{V_1}{\Delta t} - \frac{O_1}{2}\right) = \left(\frac{V_2}{\Delta t} + \frac{O_2}{2}\right)$$

This formula was used for the directing of floods.

The following were tabulated:

a) I_m em $10^6 m^3/h$, for every interval Δt = 5 hours

These values were obtained from the hydrogram of the project. Table nº 1 (Tr = 100years) Table nº 2 (Tr = 1.000 years.

b) $\frac{O}{2}$ em $10^6 m^3$/hour for each 1 m of elevation above the bottom of the tunnel

$$\frac{O}{2} = \frac{Qm^3/s \times 3.600}{1.000.000 \times 2} = 0,0018 \times Qm^3/s$$

The values of Qm^3/s are shown in table 1 of tunnel calibration

c) $\frac{V}{}$ em $10^6 m^3$/hour for every 1 m of the elevation of the reservoir above the floor of the tunnel. The values for V are shown in table 1

d) and e) - also tabulated:

$$\frac{V_1}{\Delta t} - \frac{O_1}{2} \quad e \quad \frac{V_1}{\Delta t} + \frac{O_2}{2}$$

In table these values are shown as well as the differences, to make interpolation easier.

The simulation of the operation was made assuming the reservoir was at elevation 30 at the start of the flood.

The time interval adopted was 5 hours.

Table shows the numerical operations for the simulation.

Drawing Nº 1 shows:

a) Project hydrogram: Tr = 100 years, D = 48 hours. The justification of this hydrogram is in an attached report.

b) Hydrogram of the discharges flowing out of the tunnel

c) Variation of the water level in the reservoir.

From the results, it can be noted that:

The maximum slevation obtained in the reservoir was 53.59

The maximum flow through the tunnel was 555,8 m^3/s at hours 75 and 80.

Note: $O = \frac{O_1 + O_2}{2}$

$$Q = \frac{O \times 1.000.000}{3.600} = \frac{O \times 10^4}{36}$$

B - Determination of the calibration curve of the Spillway Tunnel.

Axis Elevation: 30

Diameter: 6 m - elevation of floor 27 - elevation of top 33

Slope: 2,5%

a) - Height of water level to top of tunnel: $6 < h < D$

Critical discharges corresponding to various heights (Small Dams)

The original data were converted to metric units.

$Q = 0,555 \times c \times D^{5/2}$, where Q is in m^3/s

$Q = K\ D^{5/2}$ D in m

C of the table

The critical discharge for heights varyirgmeter by meter are shown in the table below:

$D = 6$ m; $D^{5/2} = 88,20$

Elevation	h	H/D	C	K	Q m^3/s
28	1	0,17	(Small Dams)	0,0957	8,44
29	2	0,33	" "	0,3488	30,76
30	3	0.50	" "	0,7745	68,31
31	4	0,67	" "	1,3577	119,75
32	5	0,83	" "	2,1124	186,31
33	5,64	0,95	" "	3,0624	245,01
	5,94	0,99	" "	4,0906	432,05

Note: Value H/D = 0,99 and the critical discharge were calculated only for the determination of the slope of the tunnel, to guarantee that all dischargeswith a partially filled pipe were processed in the supercritical regime, that is with control at the input, Since the formula above gives an infinite value for H/D = 1, a discharge corresponding to H/D = 0,04 was adopted for H = 6, a value compatible with those obtained for the pipe working under pressure.

b) - Calculation of the Slope

Manning's formula was used:

$$S = \left(\frac{nQ}{AR^{2/3}}\right)^2$$

$n = 0,17$
$Q = 432\ m^3/s$
$A = 0,7841 \times D^2 = 28,23$
$R = 0,2665 \times D = 1,60$
$R^{2/3} = 1,378$

$$S = \left(\frac{0,014 \times 432}{28,23 \times 1,378}\right)^2 = 0,0242$$

Where S = 0,025 or 2,5%

c) - Heights of water level above top of tunnel

For the calculation of discharges, the formula below, extracted from Small Dams, was used, with units converted into the metric system:

$$\frac{Q}{D^{5/2}} = 3,48 \left[\frac{H/D + L/D1 - 0,50}{1 + Ke + f\ \frac{L}{D}}\right]$$

with f = 0,025

Ke	= loss of load at entry	= 0,10
L	= length of tunnel	= 100 m
D	= diameter of tunnel	= 6 m
H	= height above floor of tunnel	

$$f = \frac{L}{D} \times i = \frac{100}{6} \times 0{,}025 = 0{,}42$$

The formula above can then be written as follows:

$$\frac{Q}{88{,}2} = 3{,}48 \left[\frac{(H/D + 0{,}42 - 0{,}50)}{(1 + 0{,}10 + 0{,}225)}\right]^{1/2} \quad \text{or}$$

$$Q = \frac{88{,}2 \times 3{,}48}{1{,}325^{1/2}} \left(H/D - 0{,}08\right)^{1/2}, \quad \text{or, finally}$$

$$Q = 266{,}72 \times (H/D - 0{,}08)^{1/2}$$

This formula was used to calculate the discharges of:

$H = 7$ m $H/D = 1{,}17$ (close to 1,20) - elevation 34 to

$H = 33$ m $H/D = 5{,}5$ - elevation 60

These calculations are shown in Table 1

Table 1

Calibration of the tunnel and accumulated volumes (differential in relation with volume of elevation 30 (1.82 10^6 m^3)

Elevation	H	H/D	(H/D-0,08)	$(H/D-0,08)^2$	Qm^3/s	Q/2	Remarks
30	3				60,3	0,12	$\frac{Q}{2}$ = 0,018 Qm^3/s
31	4				119,8	0,22	
32	5				186,3	0,34	
33	6				245,0	0,44	
34	7	1,17	1,09	1,044	278,5	0,50	
35	8	1,33	1,25	1,118	298,2	0,54	
36	9	1,50	1,42	1,192	317,9	0,57	
37	10	1,67	1,59	1,261	336,3	0,61	
38	11	1,83	1,75	1,323	352,9	0,64	
39	12	2,00	1,92	1,385	369,4	0,66	
40	13	2,17	2,09	1,446	385,7	0,69	
41	14	2,33	2,25	1,500	400,1	0,72	
42	15	2,50	2,42	1,556	415,0	0,75	
43	16	2,67	2,59	1,609	429,2	0,77	
44	17	2,83	2,75	1,658	442,2	0,80	
45	18	3,00	2,92	1,709	455,8	0,82	
46	19	3,17	3,08	1,758	468,9	0,84	
47	20	3,33	3,25	1,803	480,9	0,87	
48	21	3,50	3,42	1,849	493,2	0,87	
49	22	3,67	3,59	1,895	505,4	0,91	
50	23	3,83	3,75	1,936	516,4	0,93	
51	24	4,00	3,92	1,980	528,1	0,95	
52	25	4,17	4,09	2,022	539,3	0,97	
53	26	4,33	4,25	2,062	550,0	0,99	
54	27	4,50	4,42	2,102	560,6	1,01	
55	28	4,67	4,59	2,142	571,3	1,03	
56	29	4,83	4,75	2,198	586,2	1,05	
57	30	5,00	4,92	2,218	591,6	1,06	
58	31	5,16	5,08	2,254	601,2	1,08	
59	32	5,33	5,25	2,291	611,0	1,10	
60	33	5,50	5,42	2,320	620,9	1,12	

Assumption: Tunnel + Spillway of 250 m at elevation 53.50

ELE-VATION	Qm^3/s			$\div$ Q/2		
	TUNNEL	SPILLWAY	TOTAL	TUNNEL	SPILLWAY	TOTAL
53,5	555,3	0	555,3	1,00	0	1,00
54	560,6	156,6	717,2	1,01	0,28	1,29
54,5	565,9	442,5	1.008,4	1,02	0,80	1,82
55	571,3	812,8	1.364,1	1,03	1,46	2,49
55,5	578,7	1.251,4	1.830,1	1,04	2,25	3,29
56,0	586,2	1.348,8	2.335,0	1,05	3,15	4,20
	X	XX		X		

X - Interpolated values for fraction 0,5

XX - Values calculated through the use of formula: $Q = 1,77 \times 250\ h^{3/2}$

$h = 0,50,\quad h^{3/2} = 0,354 \qquad Q = 442,5 \times 0.354 = 156,6$

$h = 1,00,\quad h^{3/2} = 1,00 \qquad 1,000 = 442,5$

$h = 1,50,\quad h^{3/2} = 1,837 \qquad = 812,8$

$h = 2,00,\quad h^{3/2} = 2,828 \qquad = 1.251,4$

$h = 2,50,\quad h^{3/2} = 3,952 \qquad = 1.748,8$

TABLE FOR THE OPERATION OF DIVERSION OF FLOODS TO THE RESERVOIR

Δt = 5 hours

ELEVA-TIONS	H	$\frac{V}{\Delta t}$	$\frac{O}{2}$	DIF.	$\frac{V}{\Delta t} + \frac{O_2}{2}$	$\frac{V}{\Delta t} - \frac{O_1}{2}$	
30	3	0	-0,12		0,12	- 0,12	
31	4	0,46	0,22	0,10	0,68	0,24	0,14
32	5	0,98	0,34	0,12	1,32	0,64	0,10
33	6	1,54	0,44	0,10	1,98	1,10	0,46
34	7	2,12	0,50	0,06	2,62	1,62	0,52
35	8	2,74	0,54	0,04	3,28	2,20	0,58
36	9	3,38	0,57	0,03	3,95	2,81	0,61
37	10	4,06	0,61	0,04	4,67	3,45	0,64
38	11	4,78	0,64	0,03	5,42	4,14	0,69
39	12	5,52	0,66	0,02	6,18	4,86	0,72
40	13	6,30	0,69	0,03	5,99	5,61	0,75
41	14	7,12	0,72	0,03	7,84	6,40	0,79
42	15	7,96	0,75	0,03	8,71	7,21	0,81
43	16	8,84	0,77	0,02	9,61	8,07	0,86
44	17	9,76	0,80	0,02	10,56	8,96	0,89
45	18	10,70	0,82	0,02	11,52	9,88	0,92
46	19	11,68	0,84	0,02	12,52	10,84	0,96
47	20	12,70	0,87	0,03	13,57	11,83	0,99
48	21	13,74	0,89	0,02	14,63	12,85	1,02
49	22	14,82	0,91	0,02	15,73	13,91	1,06
50	23	15,92	0,93	0,02	16,85	14,99	1,08
51	24	17,06	0,95	0,02	18,01	16,11	1,12
52	25	18,24	0,97	0,02	19,21	17,27	1,16
53	26	19,46	0,99	0,02	20,45	18,47	1,20
53,5	26,5	20,09	1,00	0,01	21,09	19,09	0,62
54	27,0	20,72	1,29	0,29	22,01	19,43	0,34
54,5	27,5	21,37	1,82	0,53	23,19	19,55	0,12
55	28	22,02	2,49	0,67	24,51	19,53	-0,02
55,5	28,5	22,70	3,29	0,80	25,99	19,41	-0,12
56	28	23,38	4,20	0,91	27,58	19,18	-0,23

RIO IPOJUCA $I + \left(\frac{V_1}{t} - \frac{O_1}{2}\right) = \left(\frac{V_2}{t} + \frac{O_2}{2}\right)$ Tr = 100 years

HOUR	INITIAL ELEVATION	I $10^6 m^3/h$	$O_{1/2}$	$\frac{V_1}{\Delta t} - \frac{O_1}{2}$	$\frac{V_2}{\Delta t} + \frac{O_2}{2}$	FINAL ELEVATION	$O_{2/2}$	O	Q_o
0 - 5	30,00	0,13	0,12	- 0,12	0,01	30,00	0,12	0,24	66,7
5 - 10	30,00	0,44	0,12	- 0,12	0,36	30,43	0,16	0,28	77,8
10 - 15	30,00	0,84	0,16	0,03	0,87	31,30	0,26	0,42	116,7
15 - 20		1,32	0,26	0,36	1,68	32,55	0,38	0,64	177,8
20 - 25		1,90	0,38	0,89	2,79	34,26	0,51	0,89	247,2
25 - 30		2,54	0,51	1,77	4,31	36,50	0,55	1,10	305,6
30 - 35		3,12	0,59	3,13	6,25	39,09	0,61	1,20	333,3
35 - 40		3,54	0,61	4,93	8,47	41,72	0,74	1,35	375,0
40 - 45		3,77	0,74	6,95	10,75	44,20	0,80	1,54	427,8
45 - 50		3,86	0,80	9,14	13,00	46,45	0,85	1,65	458,3
50 - 55		3,81	0,85	11,30	15,11	48,44	0,90	1,75	486,1
55 - 60		3,70	0,90	13,32	17,02	50,15	0,91	1,81	502,8
60 - 65		3,48	0,91	15,16	18,64	51,53	0,96	1,87	519,4
65 - 70		3,10	0,96	16,72	19,90	52,56	0,98	1,94	536,3
70 - 75		2,84	0,98	17,94	20,78	53,26	1,00	1,48	552,7
75 - 80		2,41	1,00	18,79	21,20	53,59	1,00	2,00	555,6
80 - 85		1,94	1,00	19,20	21,14	53,54	1,00	2,00	555,3
85 - 90		1,49	1,00	19,14	20,63	53,14	0,99	1,99	551,4
90 - 95		1,14	0,99	18,04	19,78	52,46	0,98	1,97	547,2
95 - 100		0,87	0,98	17,82	18,69	51,57	0,96	1,94	538,9
100 - 105		0,67	0,96	16,77	17,44	50,51	0,96	1,92	533,3
105 - 110		0,50	0,96	15,56	16,06	49,29	0,92	1,88	522,2
110 - 115		0,36	0,92	14,22	14,58	47,95	0,89	1,81	502,8
115 - 120		0,24	0,89	12,80	13,04	46,50	0,86	1,75	485,1
120 - 125		0,14	0,86	11,34	11,49	44,96	0,82	1,68	465,7
125 - 130		0,07	0,82	9,84	9,91	43,32	0,78	1,60	444,4
130 - 135		0,01	0,78	8,35	8,35	43,32	0,76	1,56	433,3

Diversion of the project flood (Tr = 100 years) into the reservoir

46 - IPOJUCA SYSTEM - DAM B1 - OPERATION OF RESERVOIR - TR = 100 YEARS

1) - Eleveations in meters
2) - Variations in the level of the reservoir
3) - Cubic meters per second
4) - Affluent hydrogram
5) - Effluent hydrogram
6) - hours

47 - IPOJUCA SYSTEM - DAM B1 - OPERATION OF RESERVOIR - TR = 1.000 YEARS

1) - Elevations in meters

2) - Cubic meters per second

3) - Affluent hydrogram

4) - Effluent hydrogram

5) - Operation curve of the spillway tunnel

6) - Variation in the level of the reservoir

7) - Hours

1.5 ANALYSIS OF THE POSSIBILITIES

Possibilities A and A-1 were conceived to divert the flood discharges of the river Ipojuca to the river Serinhaem. As regards the river Ipojuca, total control could be obtained, at a comparatively low cost. However, this would only add the problems of the Ipojuca to those that already exist in the Serinhaem, since the reservoir studied allow only partial control of floods. There is no doubt that, by the confrontation of values and the importsnce of the areas under consideration, that it would be a partial solution worth studyying, in case no other more ptactical one could be found, especially if we admit the possibility of damming the Serinhaem itself further downstream, and so control its waters, to which is added the excess water of the Ipojuca.

On the other hand, possibility B appears as a rather interesting topographical and hydraulic solution, since it would be possible through only one construction to contain all the flow of the millenial flood, which would finally lead us to the conclusion that, conversely to the reservoirs of possibility A, whcih would have to be operated empty, reservoir B could be used for many purposes, taking advantage of its regulating capacity, and using its water for water supply on a large scale. The present opinion that regulated important flows are not neceesary for water supply, as well as the possibility that the flume downstream should allow a comparatively important flow, leads to the abandonment of possibility B, because of its high cost, in favour of possibility B-1, of reasonable cost, actually the lowest of all the possibilities studied. The choice of possibility B-1, and its execution, would not, in the future, if the generating conditions of the concepts here expressed should undergo alterations, prevent that this possibility, without prejudice to investments made, be evolved as far as possibility B, controlling the outgoing flow through a sluice, and increasing the height of the dam.

COST COMPARISON

POSSIBILITY	COST OF WORK CR$ 1,00	COST OF DISPOSSESSION CR$ 1,00	TOTAL COST CR$ 1.00
A	33.984.800	2.707.500	36.692.300
A-1	38.794.200	2.824.500	41.618.700
B	61.092.500	3.067.500	64.160.000
B-1	20.355.000	1.177.500	21.532.500

RIO IPOJUCA $I + \left(\frac{V_1}{t} - \frac{O_1}{2}\right) = \left(\frac{V_2}{t} + \frac{O_2}{2}\right)$ Tr = 1.000 years

HOUR	INITIAL ELEVATION	I $10^6 m^3/h$	$O_{1/2}$	$\frac{V_1}{\Delta t} - \frac{O_1}{2}$	$\frac{V_2}{\Delta t} + \frac{O_2}{2}$	FINAL ELEVATION	$O_{2/2}$	C	Q_s
0 - 5	30,00	0,24	0,12	- 0,12	0,12	30,00	0,12	0,24	66,7
5 - 10	30,00	0,81	0,12	- 0,12	0,69	31,02	0,22	0,34	94,4
10 - 15		1,55	0,22	0,25	1,80	32,73	0,41	0,53	175,0
15 - 20		2,41	0,41	0,98	3,39	35,16	0,54	0,95	263,9
20 - 25		3,37	0,54	2,30	5,67	38,33	0,65	1,19	330,6
25 - 30		4,45	0,65	4,38	8,83	42,13	0,75	1,40	388,9
30 - 35		5,38	0,75	7,32	12,70	46,17	0,85	1,60	444,4
35 - 40		5,95	0,85	11,01	16,96	50,09	0,93	1,78	494,4
40 - 45		6,24	0,93	15,09	21,33	53,69	1,11	2,04	566,7
45 - 50		6,30	1,11	19,33	25,63	55,38	3,10	4,21	1.165,4
50 - 55		6,18	3,10	19,44	25,62	55,38	3,10	6,20	1.722,2
55 - 60		5,93	3,10	19,44	25,37	55,29	2,95	6,05	1.680,6
60 - 65		5,53	2,95	19,46	24,99	55,16	2,75	5,70	1.583,4
65 - 70		5,03	2,75	19,49	24,52	55,01	2,51	5,25	1.461,1
70 - 75		4,42	2,51	19,53	23,95	54,79	2,21	4,72	1.311,1
75 - 80		3,64	2,21	19,54	23,18	54,50	1,82	4,03	1.119,4
80 - 85		2,91	1,82	19,55	22,46	54,19	1,49	3,31	919,4
85 - 90		2,25	1,49	19,48	21,73	53,85	1,20	2,69	747,2
90 - 95		1,68	1,20	19,33	21,01	53,44	1,00	2,20	611,1
95 - 100		1,29	1,00	19,02	20,31	52,89	0,99	1,99	552,8
100 - 105		0,98	0,99	18,34	19,32	52,09	0,97	1,96	544,4
105 - 110		0,73	0,97	17,38	18,11	51,08	0,95	1,92	533,3
110 - 115		0,53	0,95	16,20	16,73	49,69	0,93	1,88	522,2
115 - 120		0,34	0,93	14,87	15,21	48,53	0,90	1,83	508,3
120 - 125		0,19	0,90	13,41	13,60	47,03	0,87	1,77	491,7
125 - 130		0,09	0,87	11,86	11,95	45,43	0,82	1,69	469,4
130 - 135		0,01	0,82	10,29	10,30	43,73	0,79	1,61	447,2

Diversion of the flood studied in the project (Tr = 1.000 years) to the reservoir

46 - IPOJUCA SYSTEM - DAM B1 - RESERVOIR OPERATION - TR = 100 YEARS

1 - Elevation in meters

2 - Variation in the level of the reservoir

3 - Cubic Meters per second

4 - Affluent hydrogram

5- Effluent hydrogram

6 - Hours

47 - IPOJUCA SYSTEM - DAM B1 - RESERVOIR OPERATION - TR = 1.000 YEARS

1) - Elevation in meters
2) - Cubic meters per second
3) - Affluent hydrogram
4) - Effluent hydrogram
5) - Curve for the operation of the spillway tunnel
6) - Variation in the level of the reservoir
7) - Hours

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2. 0 MEREPE SYSTEM

The lowe, lagoonal waters of the river. Merepe have acted as an accumulation basin for the normal annual floods; in the case of exceptional floods, the level of the water in this area rises, and add to the Merepe discharges the supplementary discharges of the river Ipojuca which, from a certain level overflow the limits of its flume. These lagoonal areas extend toward the South, and at high levels go over the Merepe-Maracaipe divider (watershed), at Porto de Galinhas, thus turning continuous the connections Barra do Ipojuca -Barra do Merepe- Barra do Maracaipe. As a trial at disciplining this natural phenomenon, thus allowing safe occupation of the CI, traffic on the accessesPE-60 - Cupe, Cupe - Porto de Galinhas and Porto de Galinhas - PE-60, and various other accesses by road to the CI, finally disciplining the flows and the use of the flumes of the Ipojuca and Merepe within the CI, the idea was conceived of the Amortization Lake, in the study of which the control of the floods will be included, as far as the CI is concerned.

48 - MEREPE SYSTEM - FLOOD HYDROGRAM - TR = 100 YEARS

1) - Cubic meters per second

2) - Hours

49 - MEREPE BASIN - FLOOD DIAGRAM - TR = 1.000 YEARS

1) - Cubic meters per second

2) - Hours

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3 AMORTIZATION LAKE

3.1 FEATURES

The Amortization Lake is the result of the utilization of the low lagunal waters that create a continuity at the time of floods to the basins of the rivers Ipojuca, Merepe and Maracaipe, for the disciplining of the affluent and effluent flows. The lake can also be used for other purposes, such as recreation and fish breeding, making up, along with the remaining Atlantic forest, the coconut groves, the dunes and the beaches a harmonious touristic unit. The formation of the Lake is the result of the judicious construction of a dike-road (the future PE-9), with a control at 3 outlets: the present flume of the Ipojuca, the present flume of the Merepe and the dividing canal Merepe-Maracaipe (Porto de Galinhas PE-60 road). All these outlets would have inverts at elevation 1,80 (above high tide) and would be controlled through sluices for the maintenance of a constant level at elevation 3. The grade of the dike road vould be determine to elevation 4, at least, and, after paving, at elevation 4,50. The works of art would have the lower parts of their beams at elevations higher than 4.

3.2 AFFLUENT DISCHARGES

The dimensioning of the controls for the outlet of the lake, in order to maintain a constant water level at elevation 3, shall be made considering the peak discharges of the 100 year floods of the rivers Ipojuca and Merepe. As fro the river Ipojuca, we shall consider here the regulating effect of Reservoir B-1, the result of which is a discharge of 555 m^3/sec. The peak discharge of the river Merepe was calculated at 384 m^3/sec according to the hydrogram attached. The millenial floods would be considered, allowing the increase of the water level to elevation 3,80, and reducing the grade of the initial part of the Porto de Galinhas-PE-60 road, so as to make possible the complete flow of discharges estimated at 1.780 m^3/sec for the river Ipojuca and 621 m^3/sec for the river Merepe.

The conjunction of the 100 year flods of the Ipojuca Merepe with those of the Utinga de Baixo-Bita was not considered, since once dammed, they will spill into the Ipojuca system, because of its practically inexistent probability. The simultaneity of floods in these basins, considered only in the Mata and the Littoranean zone, led to lower values than those considered.

Th composition of the hydrograms for the 100 year and the millenial floods are shown in the illustrations.

50 - MEREPE LAKE - COMPOUND FLOOD HYDROGRAM - MEREPE BASIN AND EFFLUENCE OF RIVER IPOJUCA - TR = 100 YEARS

1) - Cubic meters per second

2 - Effluent hydrogram of river Ipojuca

3) - Hydrogram of the Merepe Basin

4) - Affluent hydrogram of Merepe Lake

5) - Hours

51 - MEREPE LAKE - COMPOUND FLOOD HYDROGRAM - MEREPE BASIN AND EFFLUENCE OF RIVER IPOJUCA - Tr = 1000 Years

1) - Cubic meters per second
2) - Effluent hydrogram of the river Ipojuca
3) - Hydrogram of the Merepe Basin
4) - Affluent hydrogram of the Merepe Lake
5) - hours

3.3 EFFLUENT DISCHARGES

As shown in item 2.4.1, the control of the level of the lake shall be effected through 3 outlets, utilizing the present flumes of the rivers Ipojuca and Merepe, and an already existing canal in the Merepe-Maracaipe watershed.

The maximum discharges to be handled sould be, according to the attached hydrograms, 555 m^3/sec and 1.780 m^3/sec respectively for the 100 year and 1.000 year floods.

It was accepted that control should be through flood-gates, with the following features.

Useful width	5.53 m
Width of the walls of the counterdoor	0,45 m
Offset to guarantee the flow of the outgoing water	0,15 m
Correction for the contraction of the outgoing flow	0,20 m
Height of the outgoing flow	2,00 m

The slopes were considered as free fall, made of thin walls, supported by buttresses and a foundation slab also used for the dissipation of energy (reinforced concrete).

The elevation of the slopes was fixed at 1.80 m and the level of the lake, constant to elevation 3,00, with discharges up to 555 m^3/sec (100 years), accepting, as of that value the increase in level up to elevation 3.80, for a discharge of 1.780 m^3/sec (1.000 year).

The elements of the floodgates were distributed as follows:

Sites	Merepe	Ipojuca	Maracaipe	Total
Element Nº	11	16	15	42
Discharge m^3/sec	145	198	210	553
Useful width m	60,83	88,48	82,95	232,26

The discharge per element, for a constant level at elevation 3.00 is q = 1.82 x 5,53 x 1.20 3/2, or q = 13.18 m^3/sec.

For the level at elevation 3,80, the discharge per element will be q = 1.82 x 5.53 x 1.20 3/2 = 28,50 m^3/sec, and the total discharge capacity will be 28.50 x 42 = 11.97 m^3/sec.

Assuming a damping of 10% of the load of the millenial peak, through the accumulation effect of the lake, we shall have

Q = 0,9 x 17.80 = 16.02 m^3/sec, and the remainder is therfore 16.02 - 11.97 = 405 m^3/sec to be evacuated.

By lowering the grade of the PE-9 highway, over an extension of 900 m, to elevation 3.40, we shall have a flow capacity, over the road, of:

Q = 1,77 x 900 x 0,4 3/2 = 403 m^3/sec, a sufficient amount to guarantee the system.

52 - MEREPE LAKE

1) - Conventions
2) - Site of the slopes
3) - Lake
4) - Canals
5) - Transverse section - A-B (FLOW UNIT)
6) - Maximum tide
7) - Plan (Flow unit)

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4.0 SMALL SYSTEMS

4.1 PRELIMINARIES

The small systems considered, contributors to the CI area, are the basins of the rivers Prego, Algodoais, Jasmim and Massangana.

The Massangana river is formed by the rivers Cangari and Tabatinga, that have part of their basins controlled by the reservoirs of Utinga de Baixo and Bica, the first named bleeding for the second, and the latter for the Ipojuca basin. Thus, the part of the Massangana to be considered corresponds to that located downstream from the above-mentioned reservoirs.

4.2 AFFLUENT DISCHARGES

According to the flood hydrograms exhibited,, the peak discharges to be considered are the following:

River Prego	85 m^3/sec
River Algodoais	100 m^3/sec
River Jasmim	40 m^3/sec
River Massangana	150 m^3/sec

.53 - SMALL SYSTEMS - FLOOD HYDROGRAM - TR = 100 YEARS

1) - Cubic Meters per second

54 - MAXIMUM AFFLUENT DISCHARGES IN THE COMPLEX

1) - 'Maximum discharge
2) - To dividing canal
3) - Reservoir
4) - Lake
5) - Cement
6) - Fertilizers
7) - Aluminium
8) - Railroad yard

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5.0 TENTATIVE GLOBAL ESTIMATE

5.1 GENERAL

The following works constitute the flood control system:

- Dam on the Ipojuca at Engenho Crauassũ. As we saw in the study of the water supply, it is a multi-purpose work, and represents an important component of the system.

- Dam of the Amortization Lake, with the respective bridges. These are also multi-purpose works, for they constitute the natural access to Cupe Point and to thh hamlet of Porco das Galinhas. Strictly speaking, only the inverts, the slopes and the sluices for the control of the water level are really works for the control of floods.

- Merepe, Ipojuca, Massangana, Jasmim, Algodoais and Prego collecting channels; they are natural mains that, once regulated, may serve various purposes; thus the channels above, excepting that of the Ipojuca, were considered navigable for barges.

- Cement collecting channel,so called because it supplies only the area reserved for the cement industry; it collects only the waters of the area of the CI, without outer contribution, and it is also navigable.

5.2 QUANTITATIVE ESTIMATE

As a first estimate, for quantitative purposes, the Merepe canal was pre-dimensioned, and its discharge capacity was calculated. A note at the end of this chapter shows the pre-dimensioning of the canal.

It is assumed that the canals of the rivers Prego, Algodoais, Jasmim and Massangana can have the same section as that of the Merepe.

A canal system was not studied for the river Ipojuca, since in the present conditions, it is navigable up to the site of the sluices. Only slight cleaning up operations are under consideration.

Since the canal, built of cement, receives no outer contribution, its section may well be rather restrictedwith no need of covering of the slopes.

As can be seen in the information concerning the pre-dimensioning of the Merepe canal, the section of the canal is 199,07 m^2 and it will not be open, but maintaines in the earthmoving work in the CI area. Therefore, the service to be considered will be the stone protection of 2 x 18 = 36 m^2/m.

The cement canal was pre-dimensioned with a section of 10 m at the base, a height of 6 m and a width (maximum) of 10 + 6 x 6 = 46 m, considering slopes of 3:1.

The services can be quantified as follows:

Work / Serviço	Canals: Merepe	Massangana, Jasmim, Algo-goais, Prego	Cement	Ipojuca
Cleaning				25.000 m^2
Earthmoving			150.000 m^3	
Sea wall	120.000 m^3	120.000 m^3		

Bridges, Inverts and Sluices

Work Service	Ipojuca	Merepe	Ipojuca
Bridge	115 m	80 m	105 m
Invert	101 m	76 m	100 m
Sluices	16 cm	11 cm	15 cm

Dike-Road (PE-9

Width: 7.20 m, unpaved - 15 km

Dispossession

Waterlogged or floodable land - 2 100 ha

5.3 ESTIMATE

Merepe Canal	2.400.000,00
Prego- Algodoais - Jasmim Massangana Canal	2.400.000,00
Ipojuca Canal	50.000,00
Cement Canal	600.000,00
Bridges on PE-9	3.750.000,00
Lake inverts and sluices	2.000,000,00
Dike-road (PE-9)	7.395.000,00
Dispossession in the Lake area	630.000,00
Damming of the Ipojuca	21.532.500,00
Total for Flood Control	40.757.500,00

Information about the pre-dimensioning of the Merepe Canal.

a) Geometrical elements

Elevation of invert: - 3,34 (2,00 m below minimum tide)

Slope of inevrt: horizontal

Width of bottom: 30 m

Maximum height . 2,68 (port platform) - (-3,34) = 6.02

Banks: 3 H : 1 V

Section of the Canal

Area = (15 + 3 x 6,02) x 6,02 = 199.02 m^2

Width of mouth: 50 + 6 x 6,02 = 66,12 m

Length: 3.000 m

b) Protection: large stones.

c) Determination of discharges.

The following assumptions were made for the determination of discharges:

Elevation at the head of the canal:

z_1 = + 2,38 - 30 cm below the port platform

z_2 = + 1,46 - maximum level of tide

Intermediate levels

Levels at the end of the canal close to the sea (port)

z_1 = + 1,46 - maximum level of tide
z_2 = 0 - average level of tides
z_3 = - 1,34 - minimum level of tides

Intermediate elevations

The calculations made and the formulae used are attached. The formulae are based on the assumption that in the downstream section, close to the port, the height of this section is the same as that of the tide, that acts, therefore, as a control of the draft in the canal.

The results are condensed in the table below.

Upstream Section		Downstream Section				
Elevation (m)	Height (m)	Elevation (m)	Height (m)	Area m^2	Discharge m^3/s	Deci. m/s
2,38	5,72	+ 1,46	4,80	213,12	474	2,22
2,38	5,72	+ 0,76	4,10	173,43	481	2,77
2,38	5,72	0,00	3,34	133,67	420	3,14
2,38	5,72	- 1,34	2,00	72,00	225	3,12
1,91	5,25	+ 1,46	4,80	213,12	323	1,52
1,91	5,25	+ 0,76	4,10	173,43	405	2,33
1,91	5,25	0,00	3,34	133,67	373	2,79
1,91	5,25	- 1,34	2,00	72,00	210	2,92
+ 1,66	5,00	+ 1,46	4,80	213,12	208	0,98
+ 1,66	5,00	+ 0,76	4,10	173,43	351	2,02
+ 1,66	5,00	0,00	3,34	133,67	344	2,57
+ 1,66	5,00	- 1,34	2,00	72,00	201	2,79

Upstrea Section		Downstream Section				
Elevation (m)	Height (m)	Elevation (m)	Height (m)	Area m2	Discharge m3/s	Decl. m/s
+ 1,46	4,80	+ 1,46	4,80	213,12	0	0
+ 1,46	4,80	+ 0,76	4,10	173,43	306	1,76
+ 1,46	4,80	0,00	3,34	133,67	319	2,39
+ 1,46	4,80	- 1,34	2,00	72,00	194	2,69

Th results show that:

- The speed registered show the need for lining the canal. A lining of heavy stones was considered.

- The discharges are very variable, for a same height upstream proportionately to the height downstream, showing that in the case of the flow of floods, the inlet of discharges into the canal must be controlled proportionately to the height of the water level at the point. (The canal must first be calibrated) .

In the case of the inket of discharges not previouly contracted, it seems prudent not to admit more than 200 m^3/s.