[III] DRAINAGE

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(1) OROPOUCHE (I) July 1967

1, Apr.	The outline of the Oropouche Area was explained by Mr. Antonio, Drainage Superintendent (South).
4, Apr.	Inspected with Mr. C. Taylor and Trinidad Sluice and the main drain made in 1963.
6, June	Visited the branch office at Siparia and met Mr. M. Antonio to discuss his problems.
22, June	Visited again with Mr. C. Taylor.

Now that the main drain that was suggested by the F.A.O. has been already constructed a method of discharge of the flood water from the watershed of Trinidad and Gucharon river at Trinidad Sluice Gate will have to be considered. This problem and the capacity of pumps and the scale of the retarding basin received my attention.

To arrive at the curve of the relation between flooding level and area, the flooding volume and area. First the contoured map drawn to a scale of one by ten thousands were made. Next, the flooding area was measured with a planimeter and the flooding volume was calculated as the following chart.

(Chart 1)

Elevation	Depth	Area		Mean Area		Volume	Total Volume	
ft		acres	ha	acres	acre-ft	acre-ft	ft^3	m^3
96.0	0	0.0	0.0	-	-	-	-	-
96.5	0.5	11.0	4.4	5.5	2.8	2.8	120,000	3,400
97.0	1.0	743.0	300.0	377.0	188.5	191.3	8,350,000	236,200
97.5	1.5	991.0	401.0	867.0	433.5	624.8	27,000,000	771,000
98.0	2.0	1,234.0	498.0	1,112.5	556.3	1,181.1	51,500,000	1,457,000
98.5	2.5	1,386.0	561.0	1,310.0	655.0	1,836.1	80,000,000	2,264,000

The lowest field level is 96.0 ft. However, as there are few areas below 96.6 ft., the basic field level is able to be regarded as 96.6 ft. (cf. Fig. 1).

To calculate the discharge through Trinidad Sluice, the downstream water level curve corresponding with upstream water level each 0.5 ft from El. 97.0 ft to 99.0 ft (cf. Chart 3 and Fig. 2) was made.

To calculate the flood discharge from the watershed of Trinidad river and Gucharan River, I measured the catchment area was measured with a planimeter. Then the lengths of the rivers and the differences of the river bed between the highest and Trinidad gate sill were measured, too.

(Chart 2)

Name of River	Location		Difference (H)		Length (l)		Catchment Area (a)	
Trinidad River	148 ft	94 ft	54 ft	15.3 m	4.61miles	7,420m	2,800 acres	$11.3km^2$
Gucharon River	168	94	74	22.3	8.05	12,930	6,000	24.3

BY RZIHA'S FORMULA:-

(1) Trinidad River

Velocity of river flow $V = 72(\frac{H}{l})\ 0.6$ km/hr $= 72\frac{(15.3)}{7,420} = 72\ (0.00206)^{0.6} = 1.76$ km/hr

Arrival time of flood $T = \frac{l}{V} = \frac{7.42}{1.76} = 4.2$ hr

Daily rainfall once in five years on an average

R = 4.15 inches = 105.3mm (by F.A.O. Report)

Mean hourly rainfall intensity tell flood arrival

$$r = \frac{R}{24}\ (\frac{24}{T})^{2/3} = \frac{105.3}{24}\ \frac{(24)^{2/3}}{4.2} = 4.39 \times (5.72)^{2/3} = 14.1 \text{ mm}$$

Run-off coefficient at flood peak $f = 0.5$

maximum flood discharge.

$$Q = \frac{1}{3.6}\ fr = \frac{1}{3.6} \times 0.5 \times 14.1 \times 11.3 = 22.1 m^3/sec = 781 \text{ cusecs}$$

Discharging time after flood peak

$\frac{1}{2}(1 + \lambda)\ T.\ 360\ Q = 1.00\ Rf'$ Total runoff coefficient which is 0.7

$$\lambda = \frac{Rf'A}{1.8TQ} - 1 = \frac{105.3 \times 0.7 \times 11.3}{1.8 \times 4.2 \times 22.1} - 1 = 4.0$$

$\lambda T = 4.0 \times 4.2 = 16.8$ hr

Total flood discharge $\Sigma Q = 100\ Rf'A = 100 \times 105.3 \times 0.7 \times 11.3 = 833,000m^3$

(2) Gucharon River

$$V = 72\ (\frac{H}{l})\ 0.6 \text{ km/hr} = 72\ \frac{(22.3)^{0.6}}{12,930} = 72\ (0.00172)^{0.6} = 1.58 \text{ km/hr}$$

$$T = \frac{l}{V} = \frac{12.93}{1.58} = 8.2 \text{ hr}$$

R = 4.15 inches = 105.3 mm

$$r = \frac{R}{24}\ \frac{(24)^{2/3}}{T} = \frac{105.3}{24}\ \frac{(24)^{2/3}}{8.2} = 4.39 \times (2.93)^{2/3} = 9.0 \text{ mm}$$

$f = 0.4$

$$Q = \frac{1}{3.6}\ frA = \frac{1}{3.6} \times 0.4 \times 9.0 \times 24.3 = 24.1\ m^3/sec = 852\ \text{cusecs}$$

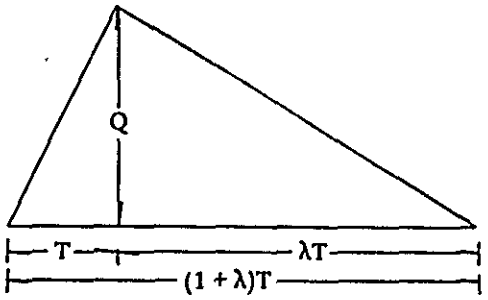
$$f' = 0.6$$

$$\tfrac{1}{2}(1+\lambda)\,T.\,3{,}600\ \ Q = 1{,}000\ Rf'A$$

$$\lambda = Rf'A - 1 = \frac{105.3 \times 0.6 \times 24.3}{1.8 \times 8.2 \times 24.1} - 1 = 3.33$$

$$\therefore\ T = 3.33 \times 8.2 = 27.3\ hr$$

$$\Sigma Q = 1{,}000\ Rf'A = 1{,}000 \times 105.3 \times 0.6 \times 24.3 = 1{,}533{,}000 m^3$$



(cf. Fig. 3)

About the external water level, the records in 1938, 1959, 1960 and 1961 were checked.

As the result, the data were taken as follows.

H.H.W.L. = 101.4 ft
L.L.W.L. = 95.0 ft

According to the report by F.A.O. the records in 1948, 1949 and 1955 showed the following figures.

H.H.W.L. = 99.5 ft
L.L.W.L. = 94.3 ft

It means that the data this time is more dangerous than the ones in the report by F.A.O. From the records of the spring tide (H.H.W.L.), and the neap tide (H.W.L.) the Trinidad Sluice cannot be used for discharge by gravity at all. Figure 4 shows that for the periods of discharge pumps must therefore be used. The internal water level and the capacity of the pumps, were calculated. (cf. Fig. 4)

First the internal water level with no pumps, that is on the present condition was checked. The maximum flooding water level showed 98.38 ft. and the flood continued, more than 2 days. Next, it with 4 propeller pumps with suction pipe in diameter of 48 inches was checked. The maximum flooding level went down and showed 97.65 ft and the flood ended completely after 43 hours. The flood over 97 ft only continued for 32.5 hours. (cf. Fig. 4)

CONCLUSION

As the capacity of pumps, more than 4 propeller pumps in diameter of 48 inches are needed. As mentioned above, if the basic field level is regarded as 96.5 ft, the flooding time over the depth of 0.5 inch (This depth is supposed to be allowable to rice) will take less than one day and a half.

If people want to build the retarding basin, they will have to consider the flooding volume after 24 hours that is about 9,000,000m³ (735 acre-feet). Of course they cannot build the basin storing all of the flooding volume. They will have to consider to reduce the flooding level by storing a part of the flood volume.

(Chart 3) Discharge of Trinidad Sluice Gates

$$q = Nb(H - h)\sqrt{2gh}$$

Up-stream W.L.	H	Down stream W.L.		h	H-h		$\sqrt{h}$	$\sqrt{2gh}$	q	Remarks
ft	ft	ft	m	ft	ft	m			m^3/s	
97.0	6.7	97.0	0	0	6.7	2.03	0	0	0	q: Quantity of flow in m^3/sec
		96.8	0.2	0.06	6.5	1.97	0.245	1.085	3.80	
97.5	7.2	97.5	0	0	7.2	2.18	0	0	0	H: upstream water head in ft.
		97.3	0.2	0.06	7.0	2.12	0.245	1.085	4.08	
		97.1	0.4	0.12	6.8	2.06	0.346	1.532	5.61	h: difference in upstream and
		97.0	0.5	0.15	6.7	2.03	0.387	1.713	6.17	downstream water head in
		96.8	0.7	0.21	6.5	1.97	0.461	2.040	7.14	ft and m
98.0	7.7	98.0	0	0	7.7	2.33	0	0	0	
		97.8	0.2	0.06	7.5	2.27	0.245	1.085	4.38	b: width of sluice which is 84
		97.6	0.4	0.12	7.3	2.21	0.346	1.532	6.02	ft (25.4m)
		97.5	0.5	0.15	7.2	2.18	0.387	1.713	6.64	
		97.0	1.0	0.30	6.7	2.03	0.548	2.425	8.74	N: discharge which coefficient
		96.8	1.2	0.36	6.5	1.97	0.600	2.655	9.29	which is 0.7
98.5	8.2	98.5	0	0	8.2	2.48	0	0	0	
		98.3	0.2	0.06	8.0	2.42	0.245	1.085	4.66	g: acceleration due to gravity
		98.1	0.4	0.12	7.8	2.36	0.346	1.532	6.43	which is $9.8m/sec^2$
		97.9	0.6	0.18	7.6	2.30	0.424	1.880	7.69	
		97.7	0.8	0.24	7.4	2.24	0.490	2.172	8.64	Sill level is 90.30 ft.
		97.5	1.0	0.30	7.2	2.18	0.548	2.424	9.39	
		97.0	1.5	0.45	6.7	2.03	0.671	2.970	10.71	
		96.8	1.7	0.51	6.5	1.97	0.714	3.160	11.05	
99.0	8.7	99.0	0	0	8.7	2.64	0	0	0	
		98.8	0.2	0.06	8.5	2.58	0.245	1.085	4.97	
		98.6	0.4	0.12	8.3	2.51	0.346	1.532	6.83	
		98.4	0.6	0.18	8.1	2.46	0.424	1.879	8.20	
		98.2	0.8	0.24	7.9	2.39	0.490	2.169	9.21	
		98.0	1.0	0.30	7.7	2.33	0.548	2.425	10.03	
		97.8	1.2	0.36	7.5	2.27	0.600	2.655	10.70	
		97.6	1.4	0.42	7.3	2.21	0.648	2.869	11.26	
		97.5	1.5	0.45	7.2	2.18	0.671	2.971	11.50	
		97.0	2.0	0.60	6.7	2.03	0.775	3.430	12.37	
		96.8	2.2	0.66	6.5	1.97	0.812	3.595	12.57	

(Chart 4) Flooding Level on Present Condition

Hour	Outside W.L.	Flood Discharge	Mean Flood	Discharge	Total	Discharge through sluice gates		Discharge with pumps	Flooding volume	Flooding depth	Inside W.L.	Remarks
0	97.1	$0m^3/s$	- m^3/s	$\cdot m^3$	$\cdot m^3/s$	$\cdot m^3/s$	m^3	$\cdot m^3$	$\cdot m^3$	·ft	96.00 ft	
1	97.0	8.0	14.0	14,400	14,400	-	-	-	14,400	0.62	96.62	
2	97.2	16.2	12.1	43,500	57,900	-	-	-	57,900	0.79	96.79	
3	97.9	24.3	20.3	73,200	131,100	-	-	-	131,100	0.88	96.88	
4	98.8	32.7	28.5	102,600	203,700	-	-	-	203,700	1.00	97.00	
5	99.4	35.4	34.1	122,800	356,500	-	-	-	356,500	1.10	97.10	
6	99.9	37.0	36.2	130,300	486,800	-	-	-	486,800	1.22	97.22	
7	100.1	38.5	37.8	136,000	622,800	-	-	-	622,800	1.34	97.34	
8	100.1	40.0	39.3	141,400	764,200	-	-	-	764,200	1.50	97.50	
9	99.9	38.6	39.3	141,400	905,600	-	-	-	905,600	1.62	97.62	
10	99.4	36.5	37.6	135,200	1,040,800	-	-	-	1,040,800	1.73	97.73	
11	98.1	34.3	35.4	127,400	1,168,200	-	-	-	1,168,200	1.83	97.83	
12	97.8	32.1	33.2	119,500	1,287,700	1.1	3,900	-	1,283,800	1.90	97.90	
13	97.2	30.0	31.1	111,900	1,395,700	7.4	26,600	-	1,369,100	1.96	97.96	
14	97.0	27.8	28.9	104,000	1,473,100	8.6	31,000	-	1,442,100	2.01	98.81	
15	97.2	25.7	26.8	96,500	1,538,600	8.0	28,800	-	1,509,800	2.05	98.05	
16	97.4	23.6	24.7	88,900	1,598,700	7.9	28,400	-	1,570,300	2.10	98.10	
17	97.9	21.4	22.5	81,000	1,651,300	4.4	15,800	-	1,635,500	2.13	98.13	
18	98.7	19.2	20.3	73,100	1,708,600	-	-	-	1,708,600	2.18	98.18	
19	99.5	17.1	18.2	65,500	1,774,100	-	-	-	1,774,100	2.23	98.23	
20	99.9	15.0	16.1	58,000	1,832,100	-	-	-	1,832,100	2.25	98.25	
21	99.6	12.9	14.0	56,400	1,882,500	-	-	-	1,882,500	2.28	98.28	
22	98.9	12.1	12.5	45,000	1,927,500	-	-	-	1,927,500	2.31	98.31	
23	97.8	11.2	11.7	42,100	1,969,600	7.0	25,200	-	1,944,400	2.32	98.32	
24	97.2	10.3	10.8	38,900	1,983,300	9.5	34,200	-	1,949,100	2.33	98.33	
25	96.8	9.4	9.9	34,700	1,983,800	10.6	38,200	-	1,945,600	2.32	98.32	
26	96.8	8.5	9.0	32,400	1,978,000	10.5	37,800	-	1,940,200	2.32	98.32	
27	97.3	7.6	8.1	29,200	1,969,400	9.1	32,800	-	1,936,600	2.31	98.31	
28	98.3	6.7	7.2	25,900	1,962,500	0.5	1,800	-	1,960,700	2.33	98.33	
29	99.0	5.8	6.3	22,700	1,983,400	-	-	-	1,983,400	2.34	98.34	
30	99.5	4.9	5.4	19,400	2,002,800	-	-	-	2,002,800	2.35	98.35	Max. 98.38
31	100.0	4.0	4.5	16,200	2,019,000	-	-	-	2,019,000	2.36	98.36	
32	100.3	3.1	3.6	13,000	2,032,000	-	-	-	2,032,000	2.37	98.37	
33	100.1	2.2	2.7	9,700	2,041,700	-	-	-	2,041,700	2.37	98.37	
34	97.7	1.3	1.8	6,500	2,048,200	-	-	-	2,048,200	2.38	98.38	
35	98.8	0.4	0.9	3,200	2,051,400	-	-	-	2,051,400	2.38	98.38	Max.

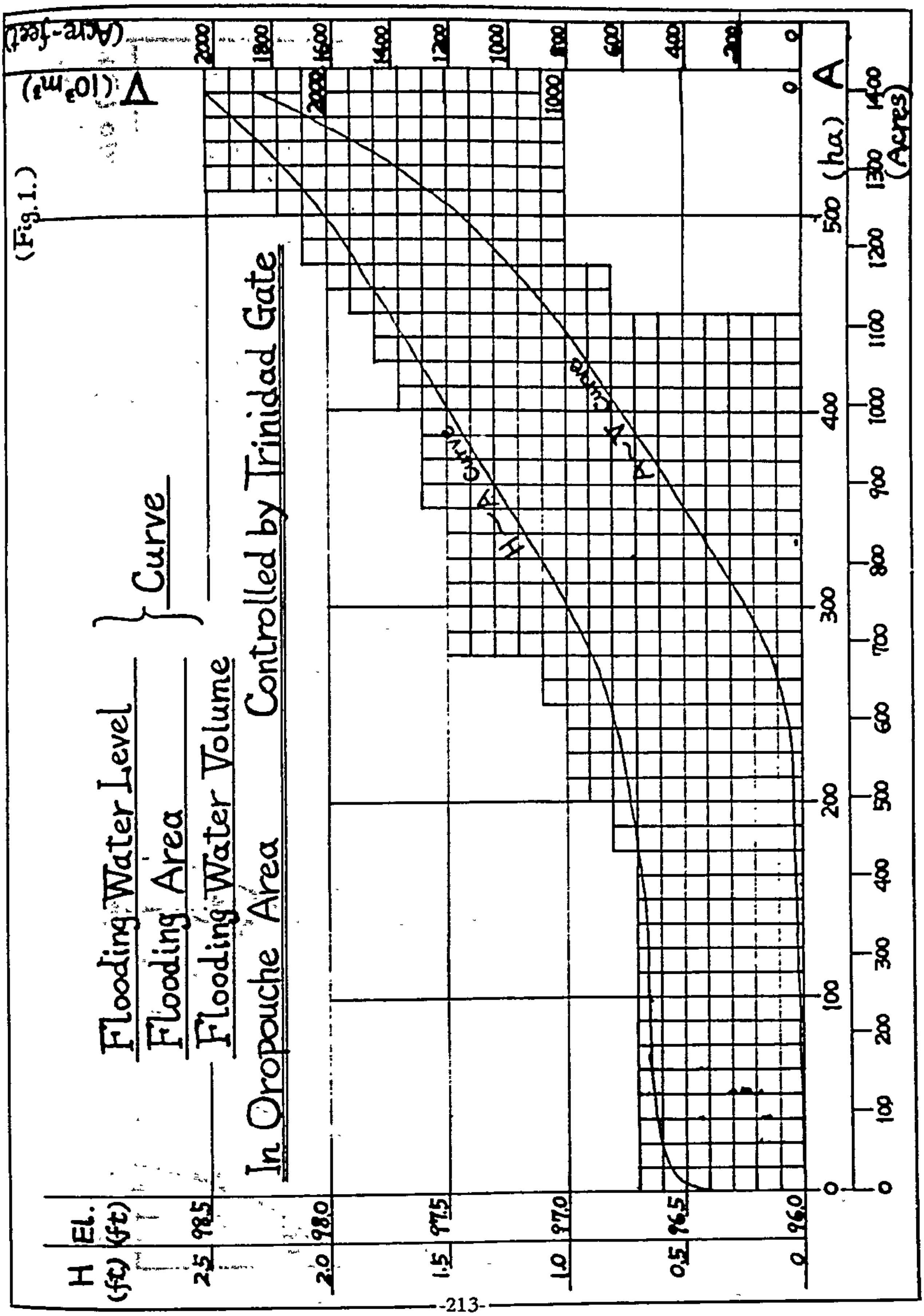
Hour	Outside W.L.	Flood Discharge	Mean Flood	Discharge	Total	Discharge through sluice gates		Discharge with pumps	Flooding volume	Flooding depth	Inside W.L.	Remarks
36	97.3	0	0.2	700	2,052,100	9.5	34,200	-	2,017,900	2.36	98.36	
37	96.4	0	0	0	2,017,900	12.4	44,600	-	1,973,300	2.33	98.33	
38	95.6	0	0	0	1,973,300	13.5	48,600	-	1,924,700	2.31	98.31	
39	95.6	0	0	0	1,924,700	13.5	48,600	-	1,876,100	2.27	98.27	
40	96.4	0	0	0	1,876,100	12.3	44,300	-	1,831,800	2.25	98.25	
41	97.5	0	0	0	1,831,800	8.2	29,500	-	1,802,300	2.24	98.24	
42	98.6	0	0	0	1,831,800	-	-	-	1,802,300	2.24	98.24	
43	99.2	0	0	0	1,831,800	-	-	-	1,802,300	2.24	98.24	
44	99.6	0	0	0	1,831,800	-	-	-	1,802,300	2.24	98.24	
45	99.6	0	0	0	1,831,800	-	-	-	1,802,300	2.24	98.24	
46	99.3	0	0	0	1,831,800	-	-	-	1,802,300	2.24	98.24	
47	98.5	0	0	0	1,831,800	-	-	-	1,802,300	2.24	98.24	
48	97.5	0	0	0	1,802,300	8.2	29,500	-	1,772,800	2.23	98.23	

(Chart 5) Flooding Level after Using Pumps

Hour	Outside W.L.	Flood Discharge	Mean Flood	Discharge	Total	Discharge through sluice gates		Discharge with pumps	Flooding volume	Flooding depth	Inside W.L.	Remarks
0	97.0ft	0m^3/s	-m^3/s	-m^3	-m^3	m^3/s	-m^3	-m^3	-m^3	-ft	96.00ft	Pump ϕ 1200
1	97.0	8.0	4.0	14,400	14,400	-	-	14,400	-	-	96.00	mm x 4 sets
2	97.2	16.2	12.1	43,500	43,500	-	-	43,500	-	-	96.00	capacity.
3	97.9	24.3	20.3	73,200	73,200	-	-	49,200	24,000	0.66	96.66	Q = 205m^3/min.
4	98.8	32.7	28.5	102,600	126,600	-	-	49,200	77,400	0.82	96.82	
5	99.4	35.4	34.1	122,800	200,200	-	-	49,200	151,000	0.90	96.90	
6	99.9	37.0	36.2	130,300	281,300	-	-	49,200	232,100	1.00	97.00	
7	100.1	38.5	37.8	136,000	368,100	-	-	49,200	318,900	1.08	97.08	
8	100.1	40.0	39.3	141,400	460,300	-	-	49,200	411,100	1.15	97.15	
9	99.9	38.6	39.3	141,400	552,500	-	-	49,200	503,300	1.24	97.24	
10	99.4	36.5	37.6	135,200	638,500	-	-	49,200	589,300	1.32	97.32	
11	98.1	34.3	35.4	127,400	716,700	-	-	49,200	667,500	1.38	97.39	
12	97.8	32.1	33.2	119,500	787,000	-	-	49,200	737,800	1.46	97.46	
13	97.2	30.0	31.1	111,900	849,700	4.5	16,200	49,200	784,300	1.48	97.48	
14	97.0	27.8	28.9	104,000	888,300	6.1	22,000	49,200	817,100	1.53	97.53	
15	97.2	25.7	26.8	96,500	913,600	5.3	19,100	49,200	845,300	1.56	97.56	
16	97.4	23.6	24.7	88,900	934,200	3.6	13,000	49,200	872,000	1.57	97.57	
17	97.9	21.4	22.5	81,000	953,000	-	-	49,200	803,800	1.60	97.60	
18	98.7	19.2	20.3	73,100	976,900	-	-	49,200	927,700	1.64	97.64	
19	99.5	17.1	18.2	65,500	993,200	-	-	49,200	944,000	1.65	97.65	
20	99.9	15.0	16.1	58,000	1,002,000	-	-	49,200	952,800	1.65	97.65	
21	99.6	12.9	14.0	50,400	1,003,200	-	-	49,200	954,000	1.65	97.65	
22	98.9	12.1	12.5	45,000	999,000	-	-	49,200	949,800	1.65	97.65	
23	97.8	11.2	11.7	42,100	991,900	-	-	49,200	942,700	1.65	97.65	
24	97.2	10.3	10.8	38,900	981,600	6.1	22,000	49,200	910,400	1.61	97.61	
25	96.8	9.4	9.9	34,700	945,100	7.6	27,400	49,200	868,500	1.50	97.58	
26	96.8	8.5	9.0	32,400	900,900	7.6	27,400	49,200	824,300	1.54	97.54	
27	97.3	7.6	8.1	29,200	853,500	4.4	15,800	49,200	788,500	1.51	97.51	
28	98.3	6.7	7.2	25,900	814,400	-	-	49,200	765,200	1.48	97.48	
29	99.0	5.8	6.3	22,700	787,900	-	-	49,200	738,700	1.46	97.46	
30	99.5	4.9	5.4	19,400	758,100	-	-	49,200	708,900	1.43	97.43	
31	100.0	4.0	4.5	16,200	725,100	-	-	49,200	675,900	1.41	97.41	
32	100.3	3.1	3.6	13,000	688,900	-	-	49,200	639,700	1.37	97.37	
33	100.1	2.2	2.7	9,700	649,400	-	-	49,200	600,200	1.33	97.33	
34	99.7	1.3	1.8	6,500	606,700	-	-	49,200	557,500	1.29	97.29	
35	98.8	0.4	0.9	3,200	560,700	-	-	49,200	511,500	1.24	97.24	

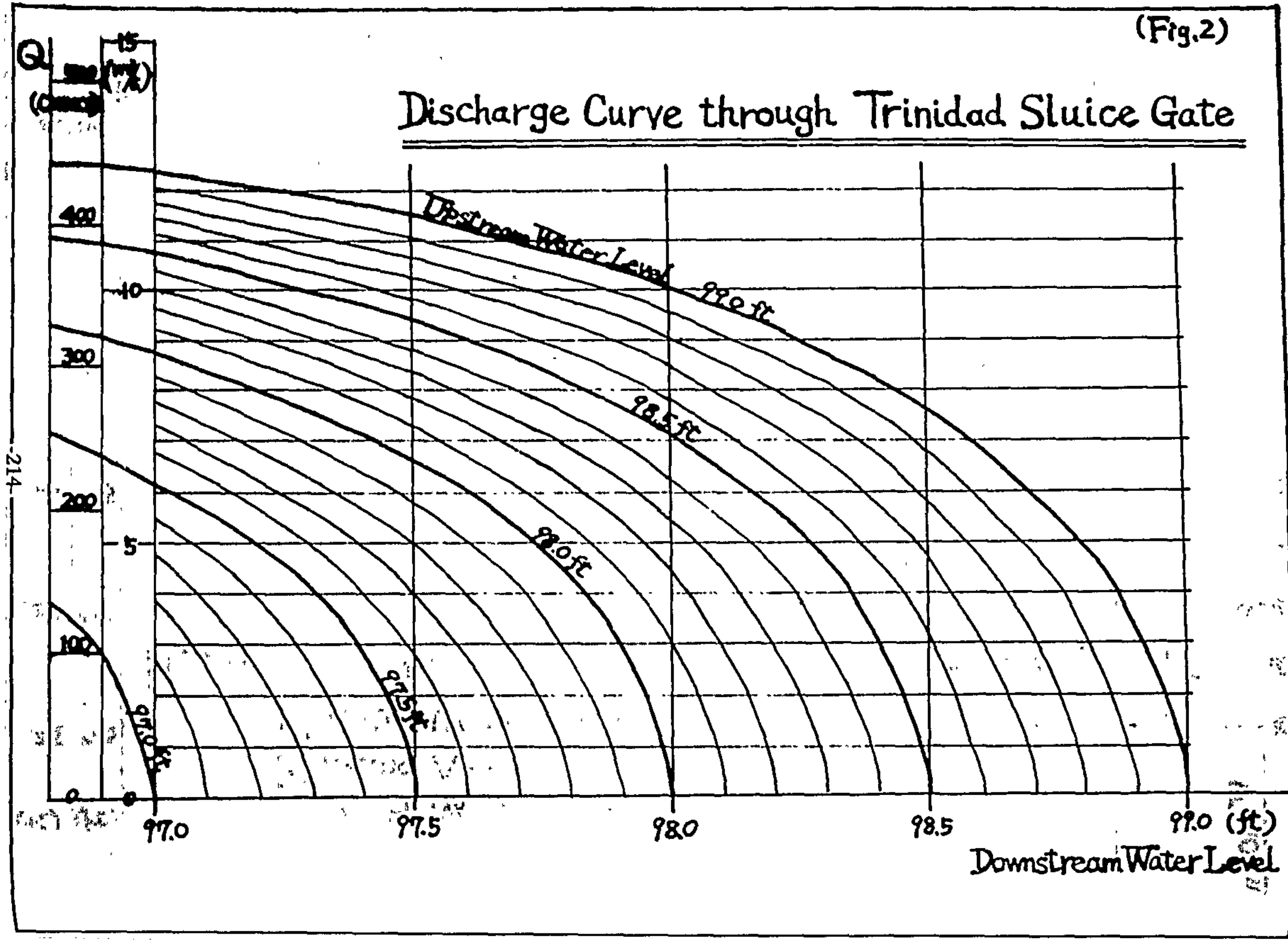
Hour	Outside W.L.	Flood Dis-charge	Mean Flood	Discharge	Total	Discharge through sluice gates		Discharge with pumps	Flooding volume	Flooding depth	Inside W.L.	Remarks
36	97.3	0	0.2	700	512,200	-	-	49,200	463,000	1.21	97.21	
37	96.4	0	0	0	463,000	8.0	28,800	49,200	385,000	1.14	97.14	
38	95.6	0	0	0	385,000	10.9	39,200	49,200	296,600	1.04	97.04	
39	95.6	0	0	0	296,600	10.5	36,800	49,200	210,600	0.97	96.97	
40	96.4	0	0	0	210,600	6.7	24,100	49,200	137,300	0.90	96.90	
41	97.5	0	0	0	137,300	-	-	49,200	88,100	0.83	96.83	
42	98.6	0	0	0	88,100	-	-	49,200	38,900	0.69	96.69	
43	99.2	0	0	0	38,900	-	-	38,900	0	0.69	96.69	
44	99.6	0	0	0								
45	99.6	0	0	0								
46	99.3	0	0	0								
47	98.5	0	0									
48	97.5	0	0									

Junichi Kitamura


(Fig.1.)
Flooding Water Level
Flooding Area
Flooding Water Volume
Curve
In Oropouche Area
Controlled by Trinidad Gate
H-A Curve
A-V Curve
H (ft)
EL. (ft)
V ($10^3 m^3$)
(Acre-feet)
A (ha)
(Acres)
0
0.5
1.0
1.5
2.0
2.5
96.0
96.5
97.0
97.5
98.0
98.5
100
200
300
400
500
1000
2000
1200
1400
1600
1800
600
800
900
1100
1300

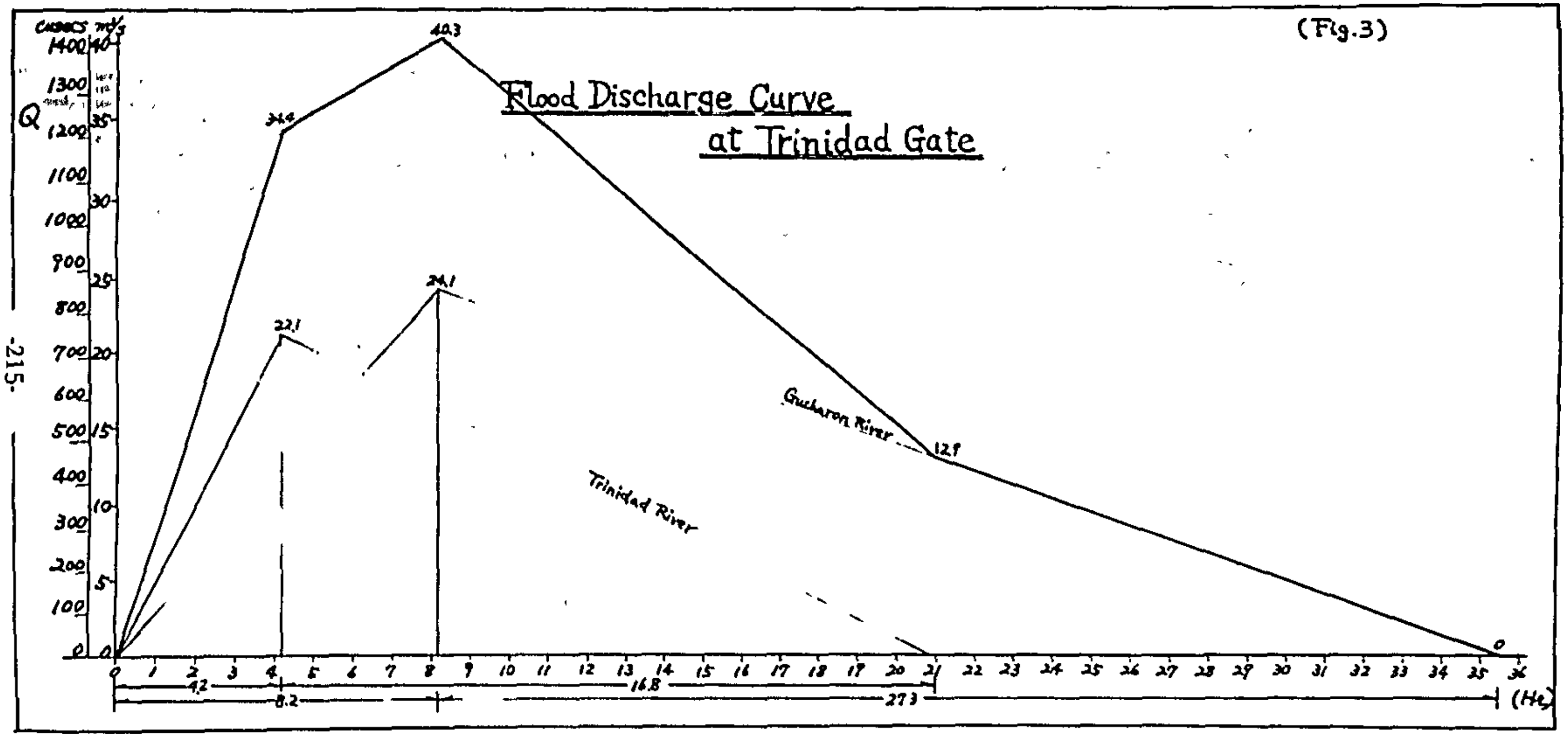
(Fig.2)

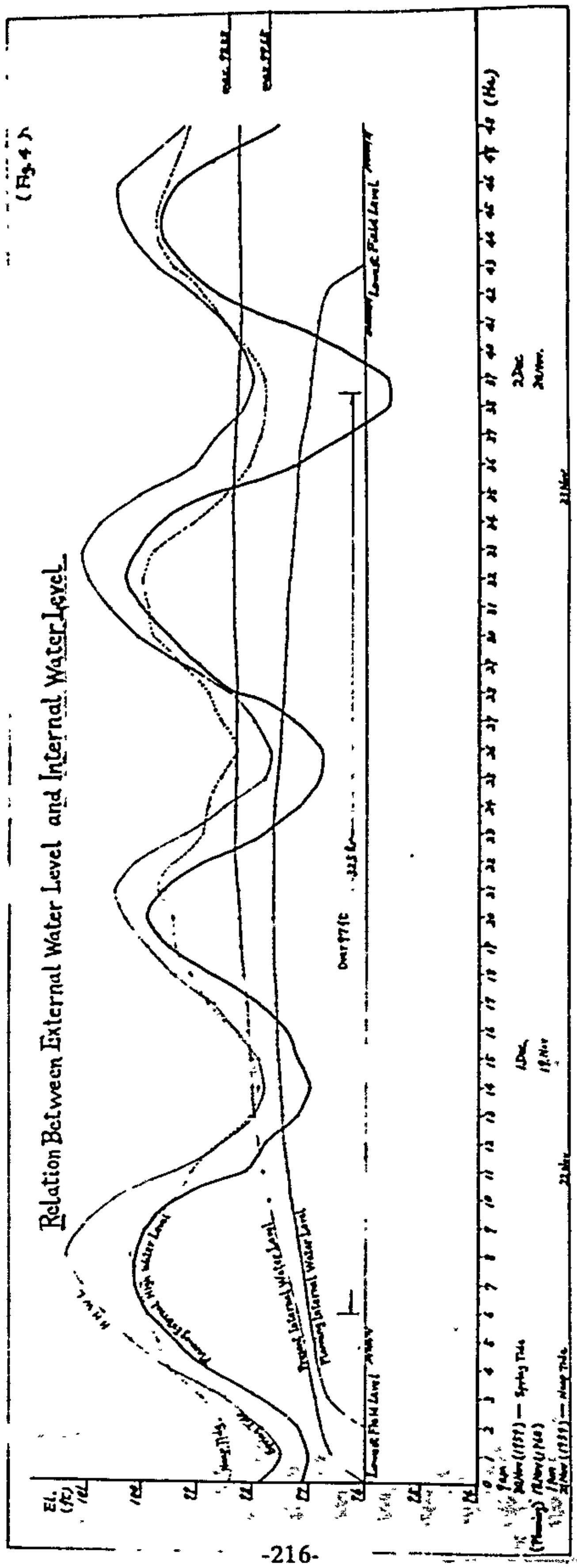
Discharge Curve through Trinidad Sluice Gate



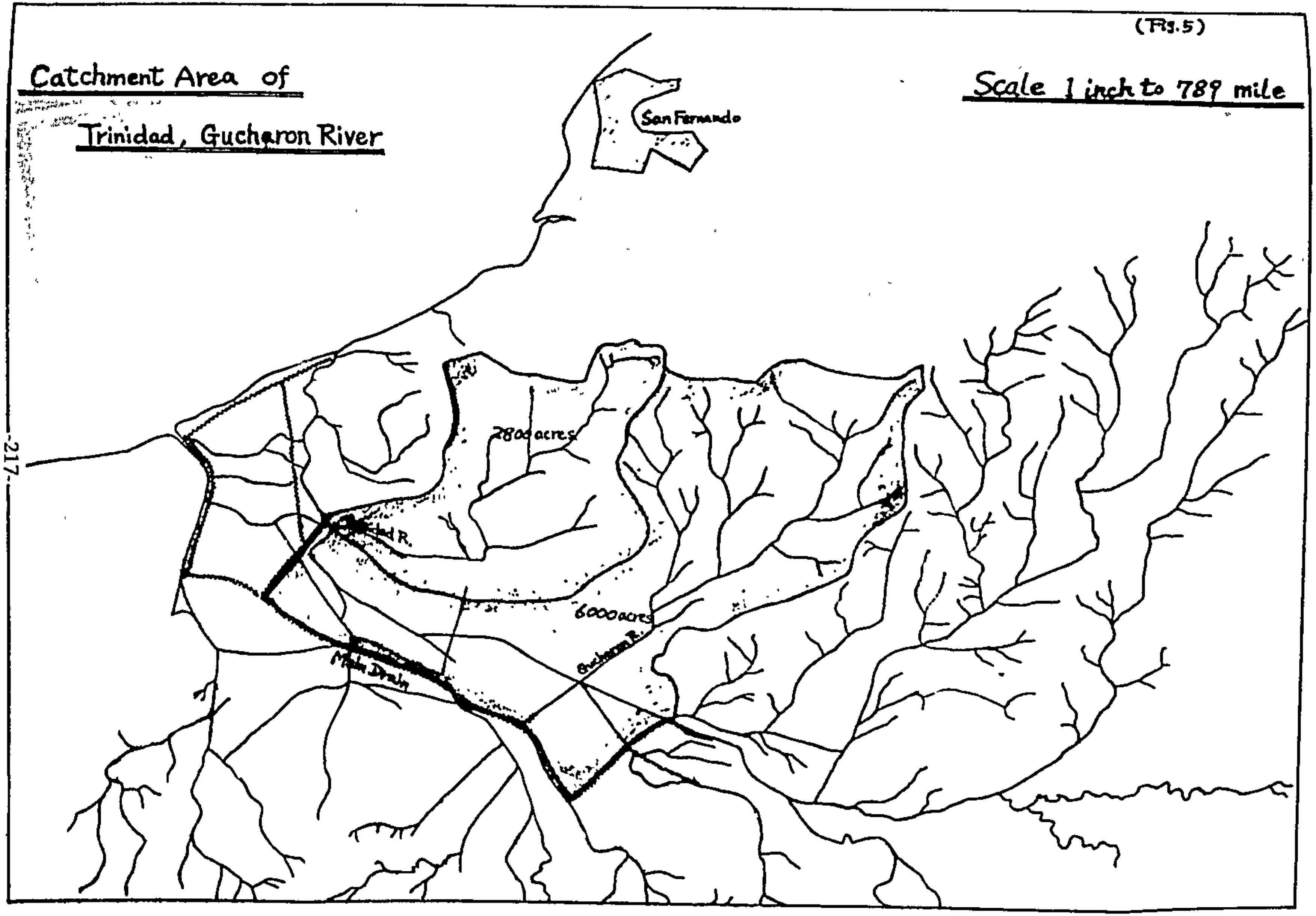
(Fig.3)

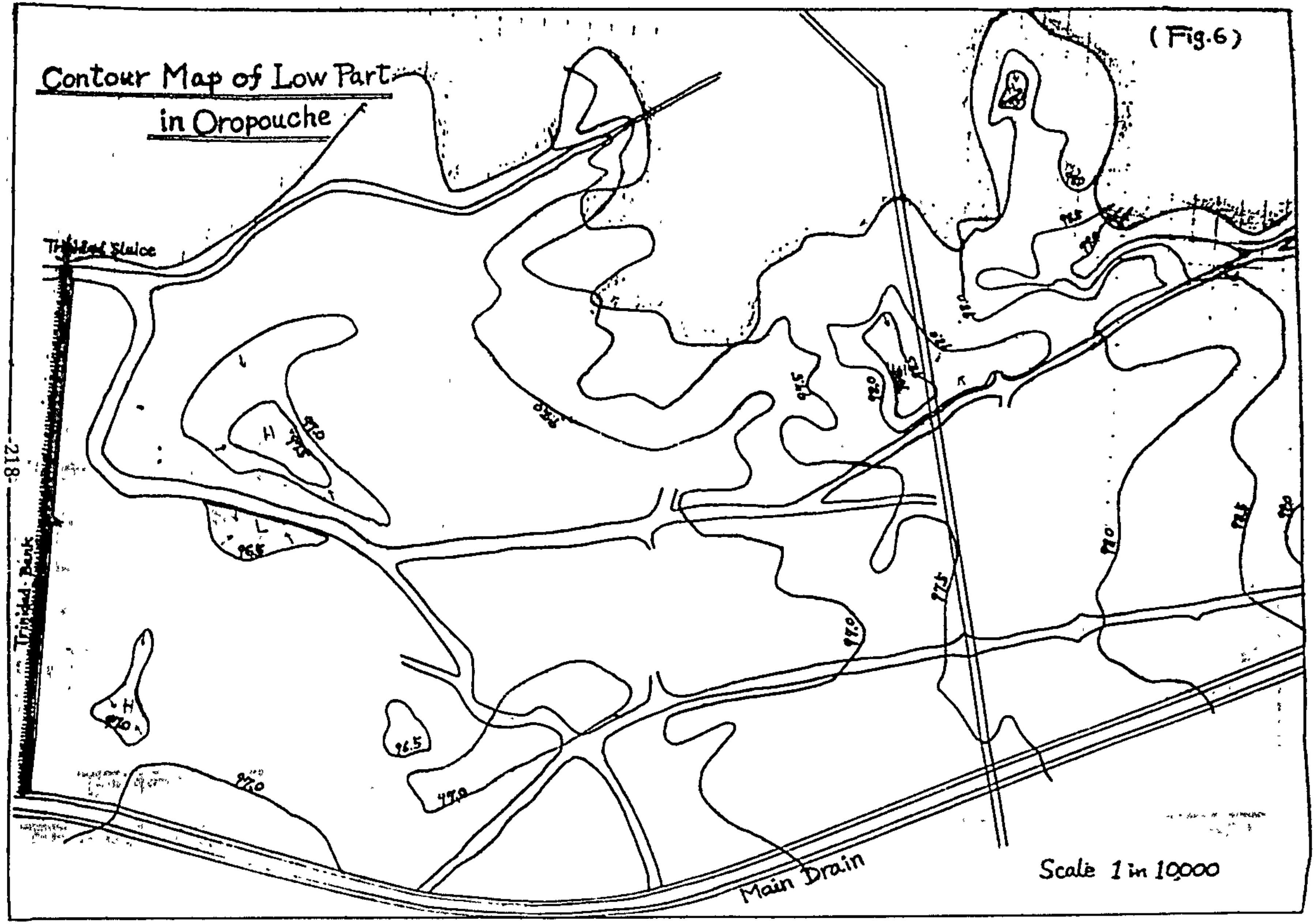
Flood Discharge Curve at Trinidad Gate





Relation Between External Water Level and Internal Water Level
(Fig. 4)


(Fig. 5)
Catchment Area of
Trinidad, Gucharon River
Scale 1 inch to 789 mile
San Fernando
2800 acres
6000 acres
Gucharon R.
Main Drain



Contour Map of Low Part in Oropouche
(Fig.6)
Trinidad Sluice
Trinidad Bank
Main Drain
Scale 1 in 10,000
97.0
97.5
98.0
98.5
96.5
97.0
H
L

OROPOUCHE (II) December 1967

MOVEMENT

26, Oct. Visited the area with Mr. Riley and saw Trinidad Sluice, Main Drain and St. John's Sluice.

INTRODUCTION

This time, the drainage plan of the catchment area of the St. John's Sluice will be set up. As mentioned on the Report Oropouche (I) it may be supposed that the drainage scheme as the first stage had been finished because the Main Drain had already been built. Consequently the scheme for the second stage has to be set up. According to the report by F.A.O. it is suggested that both the Negro River and the Cunapo River must be separated from the lower land along the Coora River by embankment and be directly discharged to St. John's Sluice, and that only the lower land along the Coora River must be drained by using pumps.

This scheme, however, will not bring a radical solution because of the shortage of discharging capacity of the St. John's Sluice in designed flood.

The catchment areas of the Negro River and the Cunapo River will flood instead of the lower land along the Coora River. It is thus recommended that the water from the whole catchment area of the St. John's Sluice be drained by pumps. (As recommended for the catchment area of the Trinidad Sluice).

DESCRIPTION OF THE AREA STUDIED: (cf. Fig. 1)

1) Present condition of the Area to be drained:

The area to be drained is situated on the left bank of the main drain and may be divided into five parts, that is, the catchment areas of the Negro River, the Cunapo River the St. Louis Ravine the Jogee Channel and the Timull's Channel or the Morocupano River respectively.

The total catchment area amounts to 65.53 km² (16,193 acres) that is nearly twice as wide as the catchment area of the Trinidad Sluice. The lowest land level is about 96.0 ft. However, there is a small area of land below 96.5 ft.

(Chart 1) Description of Confluences with the Coora River

Name of River	Location	Difference (H)		Length (1)		Catchment Area (A)	
Rio Negro	270Elft .. 91ft	179 ft	54.6m	8.28miles	13.3km	6,654.5acres	26.93km²
Cunapo River	220 ... 92	128	39.0	9.00	14.5	5,572.2	22.55
St. Louis Ravine	150 ... 93	57	17.4	1.70	2.7	946.4	3.83
Jogee Channel	150 ... 95	55	16.8	1.85	3.0	931.5	3.77
Morocupano R. Timull's Channel	300 ... 95	205	62.5	5.13	8.3	2,088.2	8.45

2) Flooding area and volume (cf. Fig. 2 & 3)

The flooding area in this catchment area is narrower than the one in the catchment area of the Trinidad Sluice.

(Viz)The possible water quantity to be stored up to the elevation of 98.0 ft. in this area is about 600 acre-feet, and in the catchment area of the Trinidad Sluice is 1,200 acre-feet.

(Chart 2)

Elevation	Depth	Area		Mean Area		Volume	Total Volume		
96.0ft	0ft	-acres	-ha	-acres	-ha	- acre-ft	-acre-ft	-ft	m^3
96.5 >	0.5	22.5	9.1	11.4	4.6	5.7	5.7	248,000	7,000
97.0 >	1.0	247.4	100.1	134.9	54.6	67.5	73.2	3,189,000	90,000
97.5 >	1.5	538.2	217.8	392.9	159.0	196.5	269.7	11,748,000	333,000
98.0 >	2.0	800.1	323.8	669.2	270.8	334.6	604.3	26,323,000	745,000
98.5 >	2.5	991.1	401.1	895.8	362.5	447.9	1,052.2	45,834,000	1,298,000
99.0 >	3.0	1,251.8	506.6	1,121.5	453.9	560.8	1,613.0	70,262,000	1,990,000
100.0 >	4.0	1,621.3	656.1	1,436.6	581.4	1,436.6	3,049.6	132,841,000	3,762,000

3) Flood discharge

The catchment area comprises low hilly lands and fields. The river-bed slope is rather steep except where each river meets the Coora River. On the contrary however, the river-bed slope of the Coora is very mild. Consequently the resultant flood discharge curve from the five tributaries at the St. John's Sluice will become a trapezoidal one. (cf. Figure 4)

4) Possible discharge from the St. John's Sluice

The possible discharge from the sluice is unexpectedly little as the sluice can get only 2 feet as a maximum head between outer water level and inner one during the designed flood. The maximum possible discharge will be less than 9.5 m^3/sec (= 330 cusecs) which is about 75% of the Trinidad Sluice (cf. Fig. 5)

5) Outer water level (cf. Figure 6)

Shown below are the tables of outer water level curves which were selected as the ones containing the highest water level each year - from 1956 till 1961 exclusive of 1958. These are all indicative of regular tidal rivers with the exception of that for 1956.

The curve in 1956 can be said to be a very abnormal one. Generally the outer water levels are found unexpectedly high. Even the minimum water level on the curves is still higher than 98 ft. Whereas the lowest level inland was found to be 96 ft.

(Chart 3-1)

1956 Date/Hour	Outer Water Level (ft)											
	1	2	3	4	5	6	7	8	9	10	11	12
7 Oct.	-	-	-	-	-	102.4	102.2	102.9	101.8	101.7	101.6	101.6
	101.6	101.6	101.9	102.0	102.2	102.5	-	-	102.6	-	-	102.0
8	-	-	102.2	-	-	102.7	102.8	102.8	102.9	103.0	103.1	103.1
	103.1	103.0	103.0	103.0	103.1	103.2	-	-	(103.4)	-	-	103.2
9	-	-	103.2	-	-	103.2	103.2	103.2	103.2	103.2	103.1	103.1
	103.1	103.0	103.0	102.9	102.9	103.0	-	-	103.1	-	-	103.0
10	-	-	102.7	-	-	102.7	102.6	102.7	102.7	102.7	102.7	102.7
	102.5	102.4	102.3	102.2	102.2	102.3	-	-	102.6	-	-	102.2
11	-	-	102.0	-	-	102.1	102.0	101.9	102.0	102.1	102.1	102.1
	102.1	102.1	101.7	101.7	101.7	101.5	-	-	101.8	-	-	101.9

(Chart 3-2)

1956 Date/Hour	Outer Water Level (ft)											
	1	2	3	4	5	6	7	8	9	10	11	12
20 Oct.	-	-	-	-	-	99.7	99.3	99.2	99.6	100.0	101.0	101.6
	101.8	102.2	102.6	102.4	101.7	-	-	-	-	-	-	-
21	-	-	-	-	-	100.6	100.0	99.0	99.5	100.0	100.5	101.2
	101.9	102.3	102.7	102.9	102.9	-	-	-	-	-	-	-
22	-	-	-	-	-	101.8	99.7	99.4	99.3	99.2	99.6	100.5
	101.4	102.2	102.4	(103.0)	102.7	-	-	-	-	-	-	-
23	-	-	-	-	-	102.2	101.5	100.6	99.4	99.0	99.4	100.1
	101.0	101.7	102.3	102.7	(103.0)	-	-	-	-	-	-	-
24	-	-	-	-	-	102.5	102.2	101.5	101.0	100.0	99.0	99.6
	100.1	101.2	101.9	102.4	102.7	-	-	-	-	-	-	-

(Chart 3-3)

1956 Date/Hour	Outer Water Level (ft)											
	1	2	3	4	5	6	7	8	9	10	11	12
29 Nov.	100.0	100.7	101.0	101.2	100.8	100.2	99.3	99.2	99.0	98.7	99.0	100.0
	100.8	101.3	101.6	101.9	101.7	101.5	100.6	99.3	99.1	98.8	98.9	99.3
30	99.8	100.4	100.9	101.3	101.5	101.0	100.1	99.7	99.4	99.2	99.2	99.6
	100.2	100.8	101.4	101.9	(102.3)	(102.3)	101.3	100.0	99.8	99.4	99.2	99.4
1 Dec	99.6	99.8	100.6	101.4	101.6	101.6	101.1	100.1	99.6	99.4	99.4	99.5
	99.9	100.2	100.9	101.4	101.9	102.1	101.4	100.8	100.3	100.0	99.8	99.5
2	99.5	99.8	100.1	100.5	100.9	101.1	101.3	100.9	100.5	100.2	100.0	99.9
	99.9	100.1	100.5	101.1	101.6	101.8	102.0	101.6	100.8	100.6	100.3	100.0
3	99.8	99.9	100.0	100.2	100.6	100.9	101.3	101.2	100.8	100.6	100.4	100.1
	100.1	100.1	100.3	100.6	101.0	101.4	101.9	101.5	101.5	100.9	100.7	100.5

(Chart 3-4)

1960 Date/ Hour	Outer Water Level (ft)											
	1	2	3	4	5	6	7	8	9	10	11	12
16 Dec	100.2	101.0	101.4	100.9	100.1	99.9	99.7	99.6	99.4	99.5	99.9	100.5
	101.0	101.5	101.9	101.8	101.5	100.5	99.7	99.3	99.0	99.1	99.4	99.7
17	100.0	100.5	101.1	101.6	101.1	100.5	99.8	99.6	99.5	99.5	99.7	100.1
	100.8	101.4	101.8	102.1	102.1	101.4	100.1	99.6	99.2	98.8	98.8	99.2
18	99.6	100.2	100.8	101.4	101.7	101.3	100.5	100.0	99.6	99.4	99.5	99.6
	100.1	100.7	101.2	101.8	102.2	102.1	101.4	100.6	99.4	99.0	98.8	99.0
19	99.4	99.8	100.3	100.9	101.3	101.5	101.1	100.4	99.8	99.4	99.3	99.3
	99.7	100.3	101.0	101.6	102.1	(102.3)	101.9	100.9	100.0	99.1	98.8	98.6
20	98.8	99.0	99.4	100.0	100.7	101.3	101.5	101.1	100.3	99.5	99.2	99.0
	99.1	99.6	100.5	101.1	101.6	101.9	102.0	101.7	100.8	99.7	99.2	98.7

(Chart 3-5)

1961 Date/ Hour	Outer Water Level (ft)											
	1	2	3	4	5	6	7	8	9	10	11	12
22 Oct	-	-	-	-	-	100.5	100.1	100.0	99.9	99.8	100.0	100.4
	100.7	101.2	101.7	101.9	101.7	-	-	-	-	-	-	-
23	-	-	-	-	-	101.5	100.9	100.5	100.1	100.0	100.0	100.2
	100.4	100.9	101.6	101.9	102.0	-	-	-	-	-	-	-
24	-	-	-	-	-	101.6	101.0	100.6	100.2	100.0	99.9	99.9
	100.2	100.5	101.1	101.6	(102.1)	-	-	-	-	-	-	-
25	-	-	-	-	-	101.7	101.5	100.9	100.4	100.1	99.9	99.8
	100.0	100.3	100.8	101.3	101.7	-	-	-	-	-	-	-

The highest outer water level each year is shown on the following table.

(Chart 4)

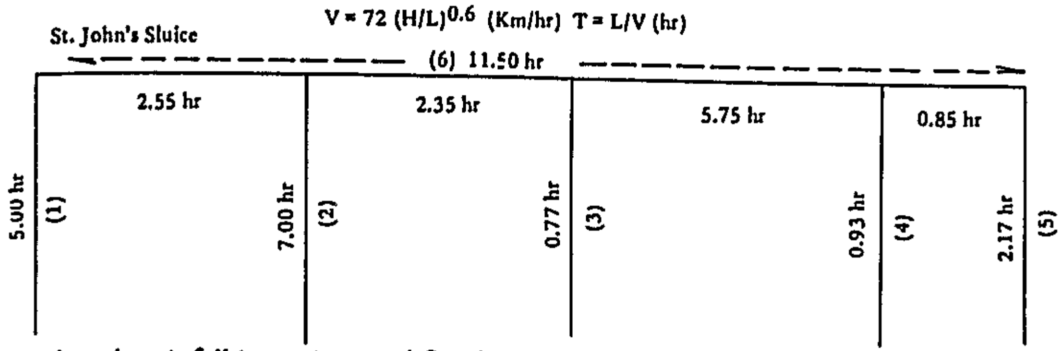
1938	1956	1957	1959	1960	1961
101.4ft	103.4ft	103.0ft	102.3ft	102.3ft	102.1ft

Calculation of the designed flood discharge

1) Flood arrival time at the end of each catchment area

(Chart 5)

	H	L	H/L	$(H/L)^{0.6}$	V	T
(1) Rio Negro	54.6m	13,300m	0.00411	0.0370	2.66km/hr	5.00hr
(2) Cunapo R.	39.0	14,500	0.00269	0.0287	2.07	7.00
(3) St Louis Ravine	17.4	2,700	0.00644	0.0485	3.49	0.77
(4) Jogee Channel	16.8	3,000	0.00560	0.0446	3.21	0.93
(5) Timull's Channel	62.5	8,300	0.00753	0.0532	3.83	2.17
(6) Coora R.	1.22	5,400	0.000226	0.00649	0.47	11.50



2) Mean hourly rainfall intensity until flood arrival which is r.
Mean daily rainfall once in five years is as follows:-

R = 4.15 inches = 105.3 mm (cf FAO report)

$$r = \frac{R}{24} \left(\frac{24}{T}\right)^{2/3}$$

(Chart 6)

	R/24	24/T	$(24/T)^{2/3}$	r
(1) Rio Negro	4.39	4.80	2.85	12.5mm
(2) Cunapo R.	4.39	3.43	2.28	10.0
(3) St Louis Ravine	4.39	24.00	8.32	36.5
(4) Jogee Channel	4.39	24.00	8.32	36.5
(5) Timull's Channel	4.39	11.06	4.96	21.8

T less than 1.0 is regarded as 1.0

3) Maximum flood discharge which is Q

$$Q = \frac{1}{3.6} frA$$

(Chart 7)

	1/3.6	f	r mm	A km^2	Q m^3/s
(1) Rio Negro	0.2778	0.5	12.5	26.93	46.8
(2) Cunapo R.	0.2778	0.5	10.0	22.55	31.3
(3) St Louis Ravine	0.2778	0.4	36.5	3.83	15.5
(4) Jogee Channel	0.2778	0.4	36.5	3.77	15.3
(5) Timull's Channel	0.2778	0.5	21.8	8.45	25.6

4) Flood continuing time after its peak which is λT

$$\lambda = \frac{RfA}{1.8\,TQ} - 1$$

(Chart 8)

	R(mm)	f'	A(km^2)	Rf'A	T(hr)	Q(m^3/s)	1.8TQ	Rf'A/1.8Tq	λ	λT(hr)
(1)	105.3	0.7	26.93	1,985.0	5.00	46.8	421.2	4.71	3.71	18.55
(2)	105.3	0.7	22.55	1,662.2	7.00	31.3	394.4	4.21	3.21	22.47
(3)	105.3	0.6	3.83	242.0	0.77	15.5	21.5	11.26	10.26	7.90
(4)	105.3	0.6	3.77	238.2	0.93	15.3	25.6	9.30	8.30	7.72
(5)	105.3	0.7	8.45	622.8	2.17	25.6	100.0	6.23	5.23	11.35

5) Whole flood discharge which is ΣQ

$$\Sigma Q = 1{,}000\, Rf'A$$

(Chart 9)

	ΣQ(m^3)
(1)	1,985,000
(2)	1,662,000
(3)	242,000
(4)	238,000
(5)	623,000
Total	4,750,000

6) Resolving the designed flood discharge from each tributary (cf. Fig. 4)

Designed outer water level (cf. Fig. 67)

The data on 18th to 19th of December in 1960 are adopted as the designed outer water level.

It is found that this outer water level may occur by flood once during spring tide every year as the result of studying data available for a six years period - 1938, 1956, 1957, 1959, 1960 and 1961.

In this connection the outer water level occurring by flood once during neap tide every year and once in more than one year will aggravate the discharging capacity of the inland water by the sluice.

Calculation of discharge from the St. John's Sluice

Sill level	90.80 ft
7 units each 6 ft wide	7 x 6 = 42 ft
1 unit 14 ft wide navigation sluice	14 ft
Total width b = 56 ft	

$q = kb\,(H - h)\sqrt{2gh}$

q: discharge from the sluice in m^3/sec

k: discharge coefficient which is 0.7

b: width of sluice which is 56 ft

H: upstream water head in ft

h: difference between upstream water level and down stream one

g: acceleration due to gravity which is 9.8 m/sec^2

(Chart 10)

Upstream W.L.	H	Downstream W.L.	k.b	h		(H - h)		$\sqrt{h}$	$\sqrt{2gh}$	q
	ft	ft	m	ft	m	ft	m			m^3/s
98.8ft	8.0	98.8	11.95	0.0	0.0	8.0	2.42	0.000	0.000	0.00
		98.6	11.95	0.2	0.6	7.8	2.36	0.245	1.085	3.06
99.0	8.2	99.0	11.95	0.0	0.00	8.2	2.48	0.000	0.000	0.00
		98.8	11.95	0.2	0.06	8.0	2.42	0.245	1.085	3.14
		98.6	11.95	0.4	0.12	7.8	2.36	0.346	1.532	4.32
99.5	8.7	99.5	11.95	0.0	0.00	8.7	2.64	0.000	0.000	0.00
		99.3	11.95	0.2	0.06	8.5	2.58	0.245	1.085	3.35
		99.1	11.95	0.4	0.12	8.3	2.51	0.346	1.532	4.60
		99.0	11.95	0.5	0.15	8.2	2.48	0.387	1.713	5.08
		98.8	11.95	0.7	0.21	8.0	2.42	0.461	2.040	5.90
		98.6	11.95	0.9	0.27	7.8	2.36	0.520	2.214	6.23
100 0	9.2	100.0	11.95	0.0	0.00	9.2	2.80	0.000	0.000	0.00
		99.8	11.95	0.2	0.06	9.0	2.74	0.245	1.085	3.55
		99.6	11.95	0.4	0.12	8.8	2.68	0 346	1.532	4.91
		99.4	11.95	0.6	0.18	8.6	2.62	0.424	1.879	5.88
		99.2	11.95	0.8	0.24	8.4	2.56	0.490	2.169	6.64
		99.0	11.95	1.0	0.30	8.2	2.48	0.548	2.425	7.19
		98.8	11.95	1.2	0.36	8.0	2.42	0.600	2.655	7.68
		98.6	11.95	1.4	0.42	7.8	2.36	0.648	2.869	8.09
100.5	9.7	100.5	11.95	0.0	0.00	9.7	2.96	0.000	0.000	0.00
		100.3	11.95	0.2	0.06	9.5	2.90	0.245	1.085	3.76
		100.1	11.95	0.4	0.12	9.3	2.83	0.346	1.532	5.18
		100.0	11.95	0.5	0.15	9.2	2.80	0.387	1.713	5.73
		99.8	11.95	0.7	0.21	9.0	2.74	0.461	2.040	6.68
		99.6	11.95	0.9	0.27	8.8	2.68	0.520	2.294	7.36
		99.4	11.95	1.1	0.33	8.6	2.62	0.574	2.542	7.96
100.5	9.7	99.2	11.95	1.3	0.39	8.4	2.56	0.624	2.763	8.45
		99.0	11.95	1.5	0.45	8.2	2.48	0.671	2.972	8.80
		98.8	11.95	1.7	0.51	8.0	2.42	0.714	3.162	9.14
		98.6	11.95	1.9	0.57	7.8	2.36	0.755	3.343	9.43

(cf. Fig. 5)

Computation of flood level under present condition (cf. Chart 11 & Fig. 7)

The sluice can discharge the capacity of only about 200,000m^3 (=160 acre ft) during one low tide.

It will take therefore more than ten days to drain the whole designed flood by using the luice alone. The maximum flooding water level reaches 100.47 ft which means 4.47 ft from the lowest land surface.

Computation of flood level by using pumps

1) Determination of number, capacity and type of pump

The whole designed flood discharge is 4,750,000m^3. Assuming that this volulme of water is drained by pumps within two days, the necessary capacity of pumps will be,

$$4{,}750{,}000/2 \times 24 \times 60 = 1{,}640\text{m}^3/\text{min}$$

Therefore,

6 pumps with capacity of 250m^3/min must be used with the St. John's Sluice.

As to the type, axial flow pumps are recommended because the actual lift is within 10 ft.

As a result of the computation, it is found that the whole designed flood discharge can be drained by pumps within 55 hours and the maximum flooding water level can be controlled below 99.40 ft.

(Chart 11) Flood Level under Present Condition

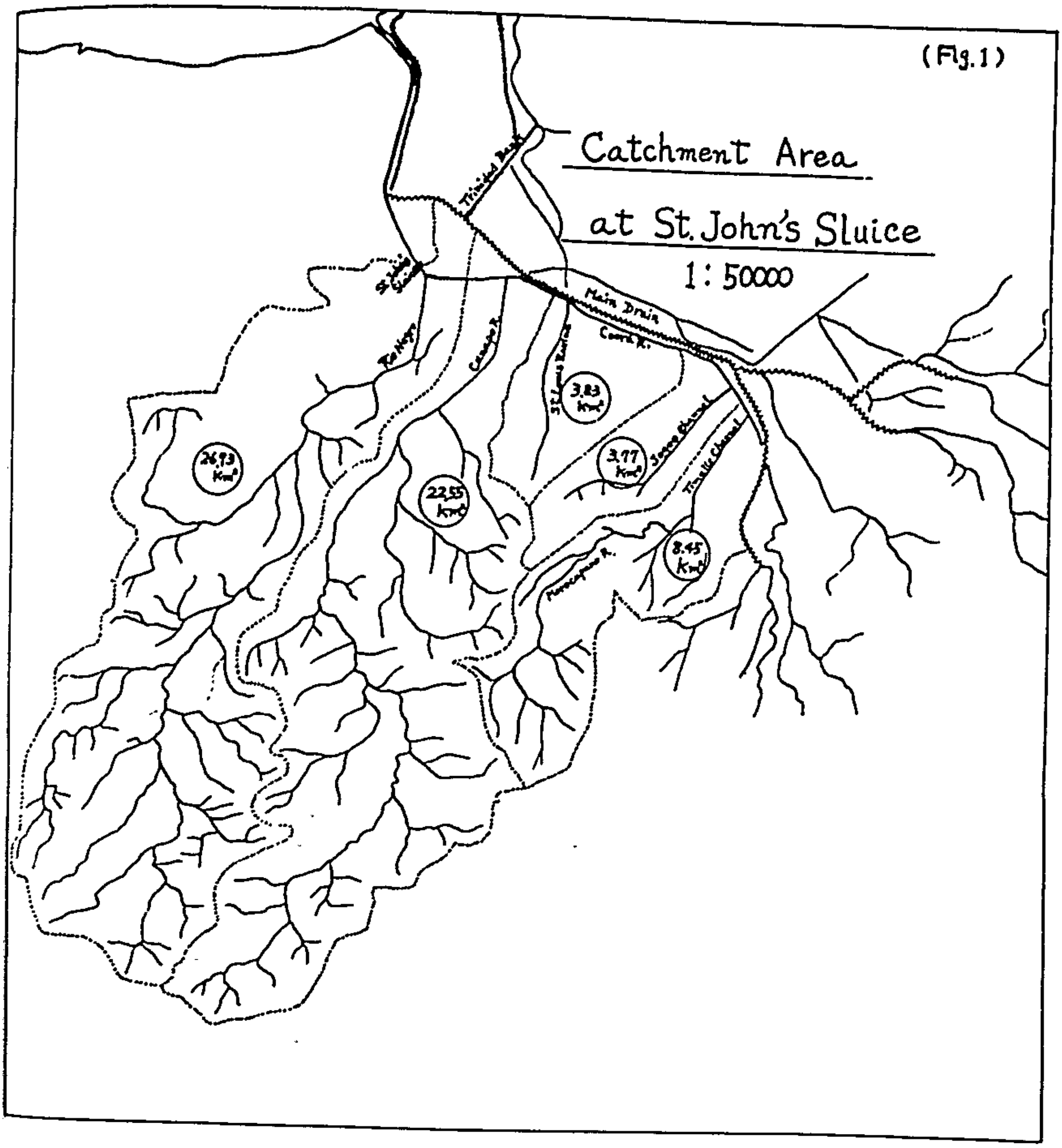
Hour	Outside W.L.	Flood Discharge	Mean Flood Discharge		Total	Discharge through Sluice gates		Flooding Volume	Flooding Depth	Inside W.L.	Remarks
C	-ft	$0m^3/s$	$0m^3/s$	$0m^3$	$0m^3$	$-m^3/s$	$-m^3$	$0m^3$	0 ft	96.00ft	
1	99.6	9.7	4.9	17,600	17,600	-	-	17,600	0.57	96.57	
2	100.2	19.1	14.4	51,800	69,400	-	-	69,400	0.90	96.90	
3	100.8	30.5	24.8	89,300	158,700	-	-	158,700	1.23	97.23	
4	101.4	44.3	37.4	134,600	293,300	-	-	293,300	1.45	97.45	
5	101.7	60.0	52.2	187,900	481,200	-	-	481,200	1.71	97.71	
6	101.3	74.9	67.5	243,000	724,200	-	-	724,200	1.98	97.98	
7	100.5	74.7	74.8	269,300	993,500	-	-	993,500	2.23	98.23	
8	100.0	74.5	74.6	268,600	1,262,100	-	-	1,262,100	2.47	98.47	
9	99.6	74.3	74.2	267,100	1,529,200	-	-	1,529,200	2.69	98.69	
10	99.4	71.8	73.3	263,900	1,793,100	-	-	1,793,100	2.88	98.88	
11	99.5	71.8	70.6	254,200	2,047,300	-	-	2,047,300	3.04	99.04	
12	99.6	80.6	76.2	274,300	2,321,600	-	-	2,321,600	3.20	99.20	
13	100.1	84.4	82.5	297,000	2,618,600	-	-	2,618,600	3.38	99.38	
14	100.7	84.1	85.1	306,400	2,925,000	-	-	2,925,000	3.54	99.54	
15	101.2	75.6	79.9	287,600	3,212,600	-	-	3,212,600	3.69	99.69	
16	101.8	67.5	71.6	257,800	3,470,400	-	-	3,470,400	3.82	99.82	
17	102.2	59.4	63.5	228,600	3,699,000	-	-	3,699,600	3.96	99.96	
18	102.1	51.3	55.4	199,400	3,898,400	-	-	3,898,400	4.06	100.06	
19	101.4	43.0	47.2	169,900	4,068,300	-	-	4,068,300	4.19	100.19	
20	100.6	36.6	39.8	143,300	4,211,600	-	-	4,211,600	4.30	100.30	
21	99.4	30.7	33.7	121,300	4,332,900	7.2	25,900	4,307,000	4.34	100.34	
22	99.0	24.6	27.7	99,700	4,406,700	8.3	29,900	4,376,800	4.38	100.38	
23	98.8	15.8	20.2	72,700	4,449,500	8.8	31,700	4,417,800	4.41	100.41	
24	99.0	13.6	14.7	52,900	4,470,700	8.5	30,600	4,440,100	4.43	100.43	
25	99.4	9.8	11.7	42,100	4,482,200	7.7	27,700	4,454,500	4.44	100.44	
26	99.8	8.2	9.0	32,400	4,486,900	6.4	23,000	4,463,900	4.44	100.44	
27	100.3	6.9	7.6	27,400	4,491,300	3.0	10,800	4,480,500	4.45	100.45	
28	100.9	5.5	6.2	22,300	4,502,800	-	-	4,502,800	3.36	100.46	
29	101.3	4.1	4.8	17,300	4,520,100	-	-	4,520,100	4.46	100.46	
30	101.5	2.7	3.4	12,200	4,532,300	-	-	4,532,300	4.47	100.47	
31	101.1	1.3	2.0	7,200	4,539,500	-	-	4,539,500	4.47	(100.47)	max El.100.47ft
32	100.4	0.1	0.7	2,500	4,542,000	2.0	7,200	4,534,800	4.47	100.47	
33	99.8	0	0	0	4,534,800	6.6	23,800	4,511,000	4.46	100.46	
34	99.4	0	0	0	4,511,000	7.4	26,600	4,484,400	4.45	100.45	

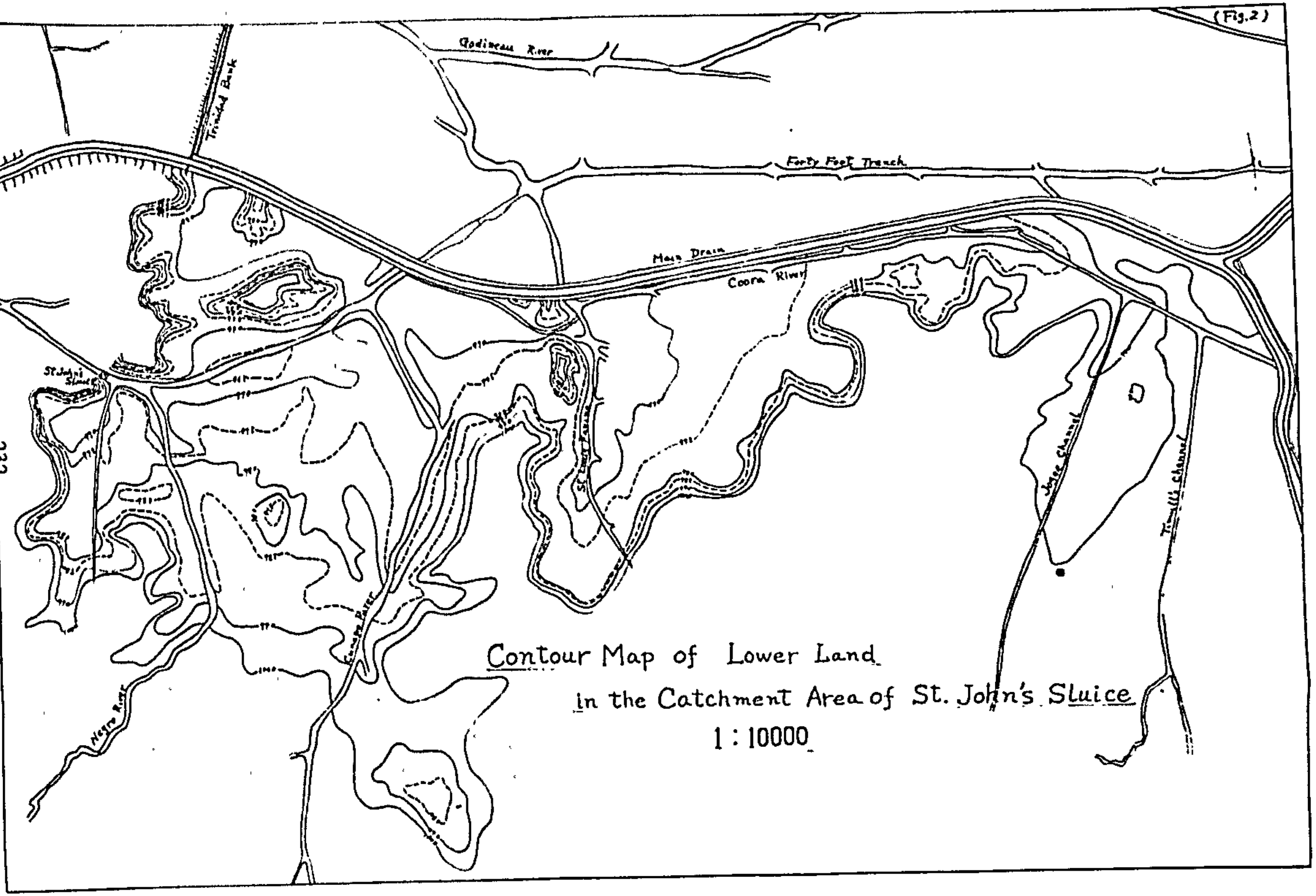
Hour	Outside W.L.	Flood Discharge	Mean Flood Discharge		Total	Discharge through Sluice gates		Flooding Volume	Flooding Depth	Inside W.L.	Remarks
35	99.3	0	0	0	4,484,400	8.1	29,200	4,455,200	4.44	100.44	
36	99.3	0	0	0	4,455,200	8.0	28,800	4,426,400	4.42	100.42	
37	99.7	0	0	0	4,426,400	6.7	24,100	4,402,300	4.40	100.40	
38	100.3	0	0	0	4,402,300	2.2	7,900	4,394,400	4.39	100.39	
39	101.0	0	0	0	4,394,400	-	-	4,394,400	4.39	100.39	
40	101.6	0	0	0	4,394,400	-	-	4,394,400	4.39	100.39	
41	102.1	0	0	0	4,394,400	-	-	4,394,400	4.39	100.39	
42	102.3	0	0	0	4,394,400	-	-	4,394,400	4.39	100.39	
43	101.9	0	0	0	4,394,400	-	-	4,394,400	4.39	100.39	
44	100.9	0	0	0	4,394,400	-	-	4,394,400	4.39	100.39	
45	100.0	0	0	0	4,394,400	4.9	17,600	4,376,800	4.38	100.38	
46	99.1	0	0	0	4,376,800	8.3	29,900	4,346,900	4.36	100.36	
47	98.8	-	-	0	4,346,900	8.8	31,700	4,315,200	4.35	100.35	
48	98.6	0	0	0	4,315,200	9.1	32,800	4,282,400	4.33	100.33	
49	98.8	0	0	0	4,282,400	8.6	30,960	4,251,440	4.31	100.31	
50	99.0	0	0	0	4,251,440	8.2	29,520	4,221,920	4.29	100.29	
51	99.4	0	0	0	4,221,920	7.2	25,900	4,196,020	4.27	100.27	
52	100.0	0	0	0	4,196,020	3.7	13,320	4,182,700	4.26	100.26	
53	100.7	0	0	0	4,182,700	-	-	4,182,700	4.26	100.26	
54	101.3	0	0	0	4,182,700	-	-	4,182,700	4.26	100.26	
55	101.5	0	0	0	4,182,700	-	-	4,182,700	4.26	100.26	

(Chart 12) Flood Level by Using Pump

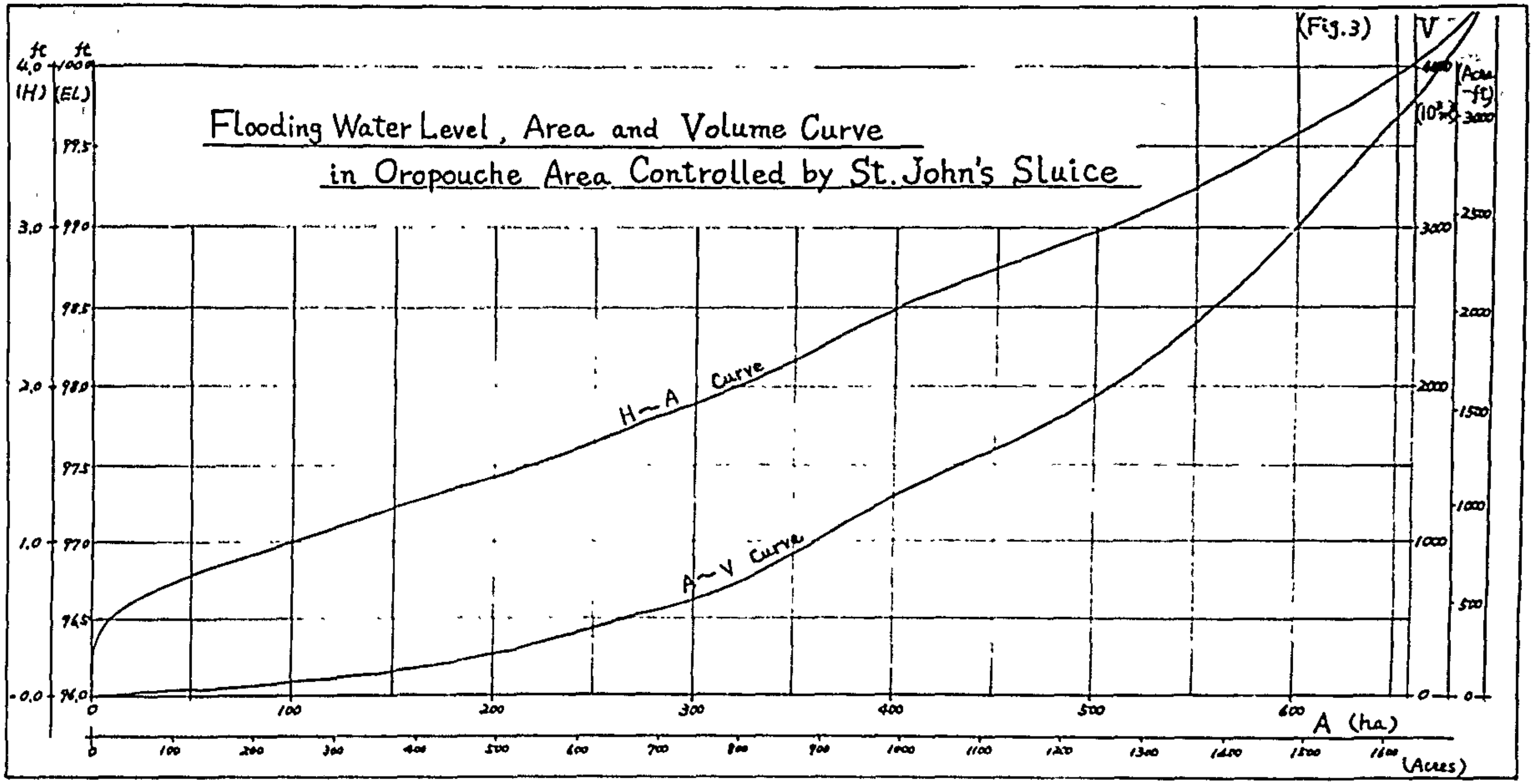
Hour	Outside W.L.	Flood Discharge	Mean Flood Discharge		Total	Discharge through Sluice gates		Discharge with pumps	Flooding Volume	Flooding Depth	Inside W.L.	Remarks
0	-ft	$0m^3/s$	$0m^3/s$	$0m^3$	$0m^3$	$-m^3/s$	$-m^3$	$-m^3$	$0m^3$	0 ft	96.00ft	
1	99.6	9.7	4.9	17,600	17,600	-	-	17,600	0	0	96.00	
2	100.2	19.1	14.4	51,800	51,800	-	-	51,800	0	0	96.00	Pumps Ø 1,350mm
3	100.8	30.5	24.8	89,300	89,300	-	-	89,300	0	0	96.00	x 6 sets
4	101.4	44.3	37.4	134,600	134,600	-	-	90,000	44,600	0.78	96.78	
5	101.7	60.0	52.2	187,900	232,500	-	-	90,000	142,500	1.17	97.17	Capacity
6	101.3	74.9	67.5	243,000	385,500	-	-	90,000	295,500	1.45	97.45	$Q=250m^3$/in each
7	100.5	74.7	74.8	269,300	564,800	-	-	90,000	474,800	1.70	97.70	
8	100.0	74.5	74.6	268,600	833,400	-	-	90,000	743,600	2.00	98.00	
9	99.6	74.3	74.2	267,100	1,010,500	-	-	90,000	920,500	2.16	98.16	
10	99.4	71.8	73.3	263,900	1,184,400	-	-	90,000	1,094,400	2.32	98.32	
11	99.5	71.8	70.6	254,200	1,348,600	-	-	90,000	1,258,600	2.47	98.47	
12	99.6	80.6	76.2	274,300	1,532,900	-	-	90,000	1,442,900	2.63	98.63	
13	100.1	84.4	82.5	297,000	1,739,900	-	-	90,000	1,649,900	2.79	98.79	
14	100.7	84.1	85.1	306,400	1,956,300	-	-	90,000	1,866,300	2.94	98.94	
15	101.2	75.6	79.9	287,600	2,153,900	-	-	90,000	2,063,900	3.05	99.05	
16	101.8	67.5	71.6	257,800	2,321,700	-	-	90,000	2,231,700	3.15	99.15	
17	102.2	59.4	63.5	228,600	2,460,300	-	-	90,000	2,370,300	3.24	99.24	
18	102.1	51.3	55.4	199,400	2,569,700	-	-	90,000	2,479,700	3.30	99.30	
19	101.4	43.0	47.2	169,900	2,649,600	-	-	90,000	2,559,600	3.34	99.34	
20	100.6	36.6	39.8	143,300	2,702,900	-	-	90,000	2,612,900	3.37	99.37	
21	99.4	30.7	33.7	121,300	2,734,200	-	-	90,000	2,644,200	3.40	99.40	max. El 99.40 ft
22	99.0	24.6	27.7	99,700	2,743,900	4.5	16,200	90,000	2,637,700	3.39	99.39	
23	98.8	15.8	20.2	72,700	2,710,400	4.8	17,300	90,000	2,603,100	3.37	99.37	
24	99.0	13.6	14.7	52,900	2,656,000	4.3	15,500	90,000	2,550,500	3.34	99.34	
25	99.4	9.8	11.7	48,100	2,592,600	-	-	90,000	2,502,600	3.31	99.31	
26	99.8	8.2	9.0	32,400	2,535,000	-	-	90,000	2,445,000	3.27	99.27	
27	100.3	6.9	7.6	27,400	2,472,400	-	-	90,000	2,382,400	3.25	99.25	
28	100.9	5.5	6.2	22,300	2,404,700	-	-	90,000	2,314,700	3.20	99.20	
29	101.3	4.1	4.8	17,300	2,332,000	-	-	90,000	2,242,000	3.16	99.16	
30	101.5	2.7	3.4	12,200	2,254,200	-	-	90,000	2,164,200	3.11	99.11	
31	101.1	1.3	2.0	7,200	2,171,400	-	-	90,000	2,081,400	3.06	99.06	
32	100.4	0.1	0.7	2,500	2,083,900	-	-	90,000	1,993,900	3.01	99.01	
33	99.8	0	0	0	1,993,900	-	-	90,000	1,903,900	2.96	98.96	
34	99.4	0	0	0	1,903,900	-	-	90,000	1,813,900	2.90	98.90	

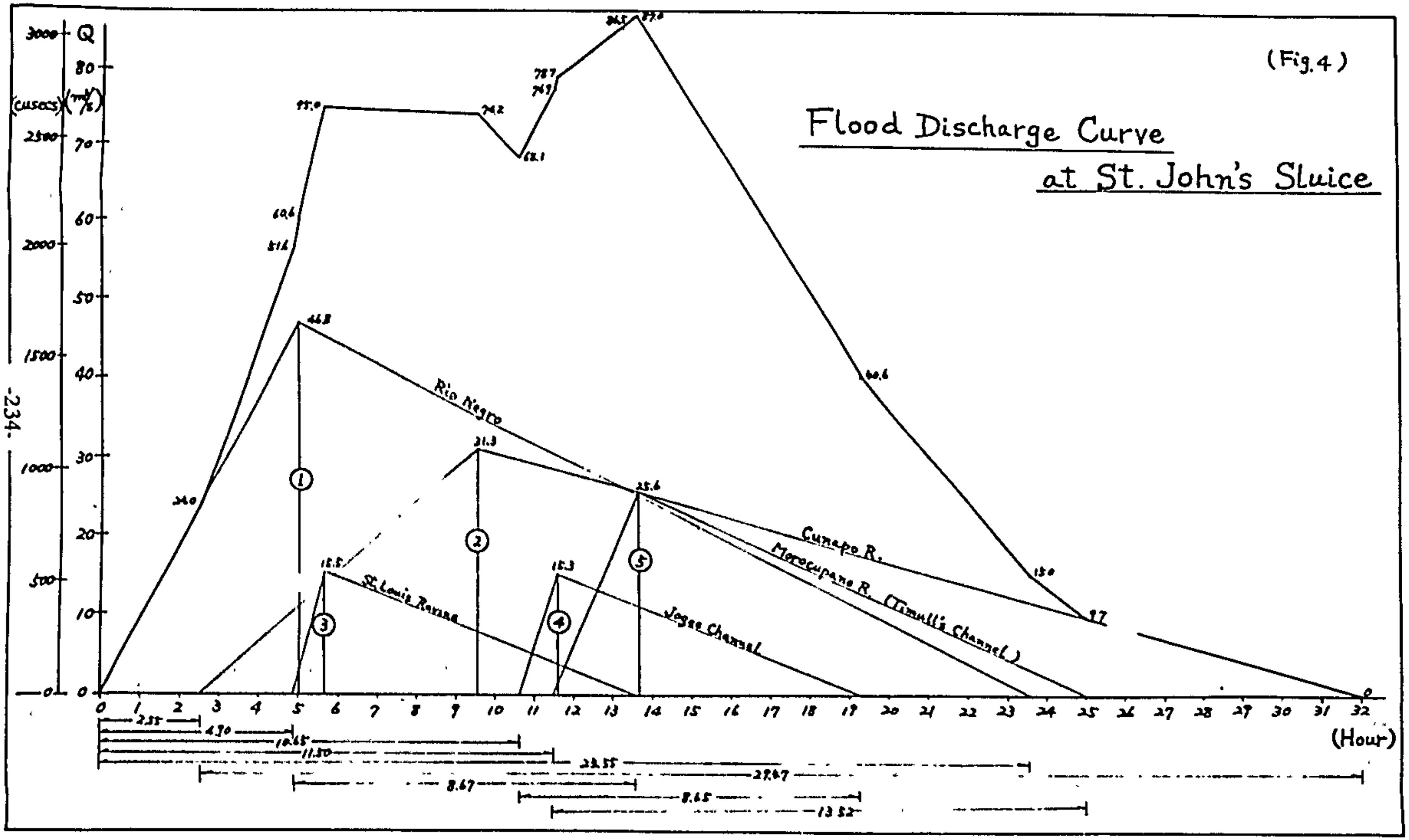
Hour	Outside W.L.	Flood Discharge	Mean Flood Discharge		Total	Discharge through Sluice gates		Discharge with pumps	Flooding Volume	Flooding Depth	Inside W.L.	Remarks
35	99.3	0	0	0	1,813,900	-	-	90,000	1,723,900	2.85	98.85	
36	99.3	0	0	0	1,723,900	-	-	90,000	1,633,900	2.78	98.78	
37	99.7	0	0	0	1,633,900	-	-	90,000	1,543,900	2.69	98.69	
38	100.3	0	0	0	1,543,900	-	-	90,000	1,453,900	2.63	98.63	
39	101.0	0	0	0	1,453,900	-	-	90,000	1,363,900	2.55	98.55	
40	101.6	0	0	0	1,363,900	-	-	90,000	1,273,900	2.48	98.48	
41	102.1	0	0	0	1,273,900	-	-	90,000	1,183,900	2.400	98.40	
42	102.3	0	0	0	1,183,900	-	-	90,000	1,093,900	2.31	98.31	
43	101.9	0	0	0	1,093,900	-	-	90,000	1,003,900	2.23	98.23	
44	100.9	0	0	0	1,003,900	-	-	90,000	913,900	2.15	98.15	
45	100.0	0	0	0	913,900	-	-	90,000	823,900	2.03	98.03	
46	99.1	0	0	0	823,900	-	-	90,000	733,900	1.99	97.99	
47	98.8	0	0	0	733,900	-	-	90,000	643,900	1.91	97.91	
48	98.6	0	0	0	643,900	-	-	90,000	553,900	1.82	97.82	
49	98.8	0	0	0	553,900	-	-	90,000	463,900	1.68	97.68	
50	99.0	0	0	0	463,900	-	-	90,000	373,900	1.55	97.55	
51	99.4	0	0	0	373,900	-	-	90,000	283,900	1.42	97.42	
52	100.0	0	0	0	283,900	-	-	90,000	193,900	1.30	97.30	
53	100.7	0	0	0	193,900	-	-	90,000	103,900	1.04	97.04	
54	101.3	0	0	0	103,900	-	-	90,000	13,900	0.50	96.50	
55	101.5	0	0	0	13,900	-	-	13,900	0	0	96.00	


(Fig.1)
Catchment Area
at St. John's Sluice
1 : 50000
Main Drain
Coore R.
26.93 Km²
22.55 Km²
3.83 Km²
3.77 Km²
8.45 Km²


(Fig.2)
Godineau River
Trinidad Bank
Forty Foot Trench
Main Drain
Coora River
St. John's Sluice
Negro River
Joyce channel
Contour Map of Lower Land
In the Catchment Area of St. John's Sluice
1 : 10000

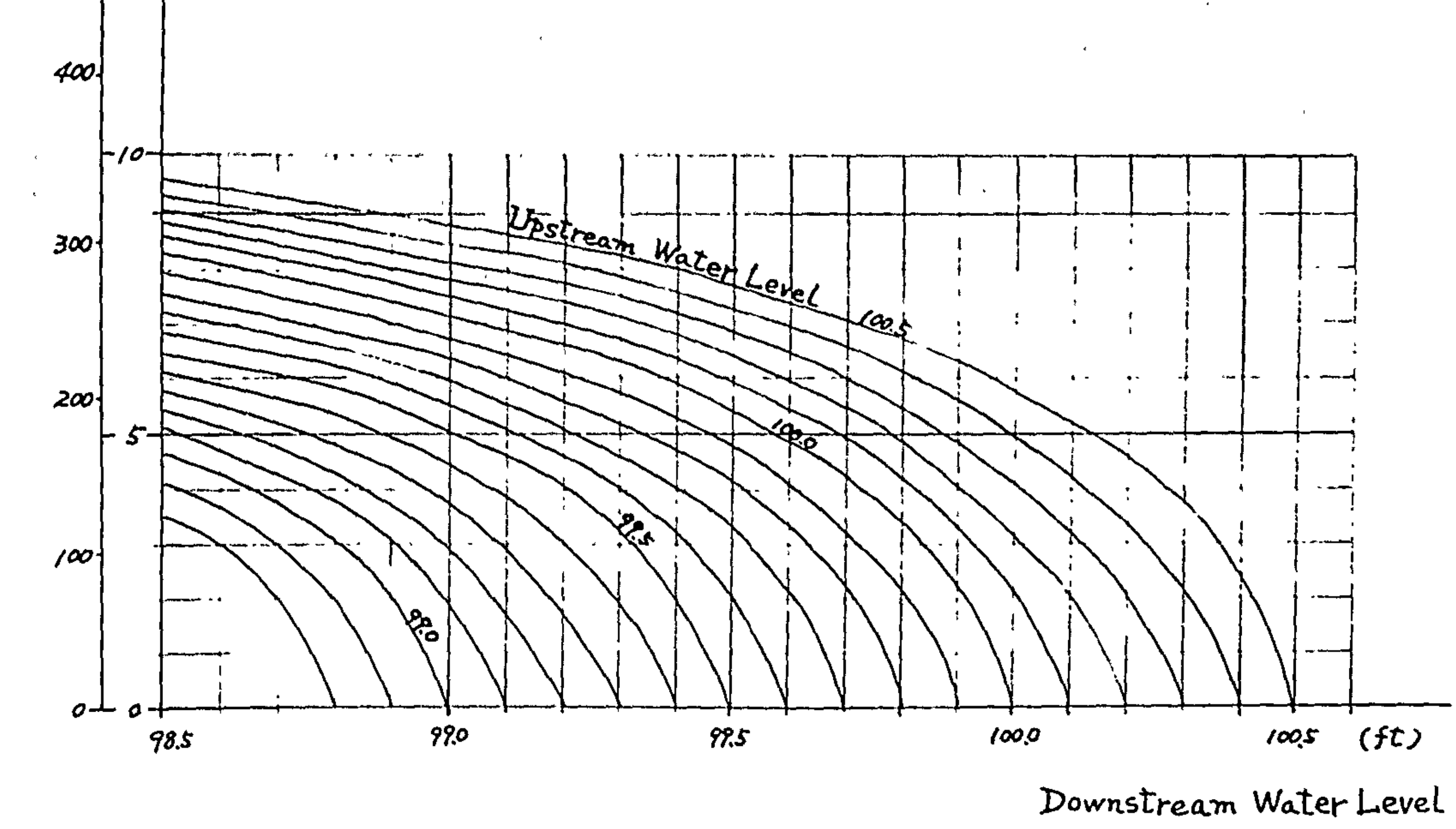
(Fig.3)


Flooding Water Level, Area and Volume Curve
in Oropouche Area Controlled by St. John's Sluice
H~A Curve
A~V Curve
ft
(H)
4.0
3.0
2.0
1.0
-0.0
ft
(EL)
100.0
99.5
99.0
98.5
98.0
97.5
97.0
96.5
96.0
A (ha)
0
100
200
300
400
500
600
(Acres)
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
1500
1600
V
($10^3 m^3$)
4000
3000
2000
1000
0
(Acre-ft)
3000
2500
2000
1500
1000
500
0


(Fig. 4)
Flood Discharge Curve
at St. John's Sluice
Q
(cusecs)
3000
2500
2000
1500
1000
500
0
80
70
60
50
40
30
20
10
0
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 31 32
(Hour)
Rio Negro
Cunepo R.
Morocupano R. (Timulli's Channel)
St. Louis Ravine
Jogee Channel
1
2
3
4
5
46.8
31.3
25.6
15.5
15.3
24.0
60.6
74.2
62.1
78.7
74.9
40.6
15.0
9.7
0
2.55
4.90
10.65
11.50
23.55
29.47
8.67
8.65
13.52

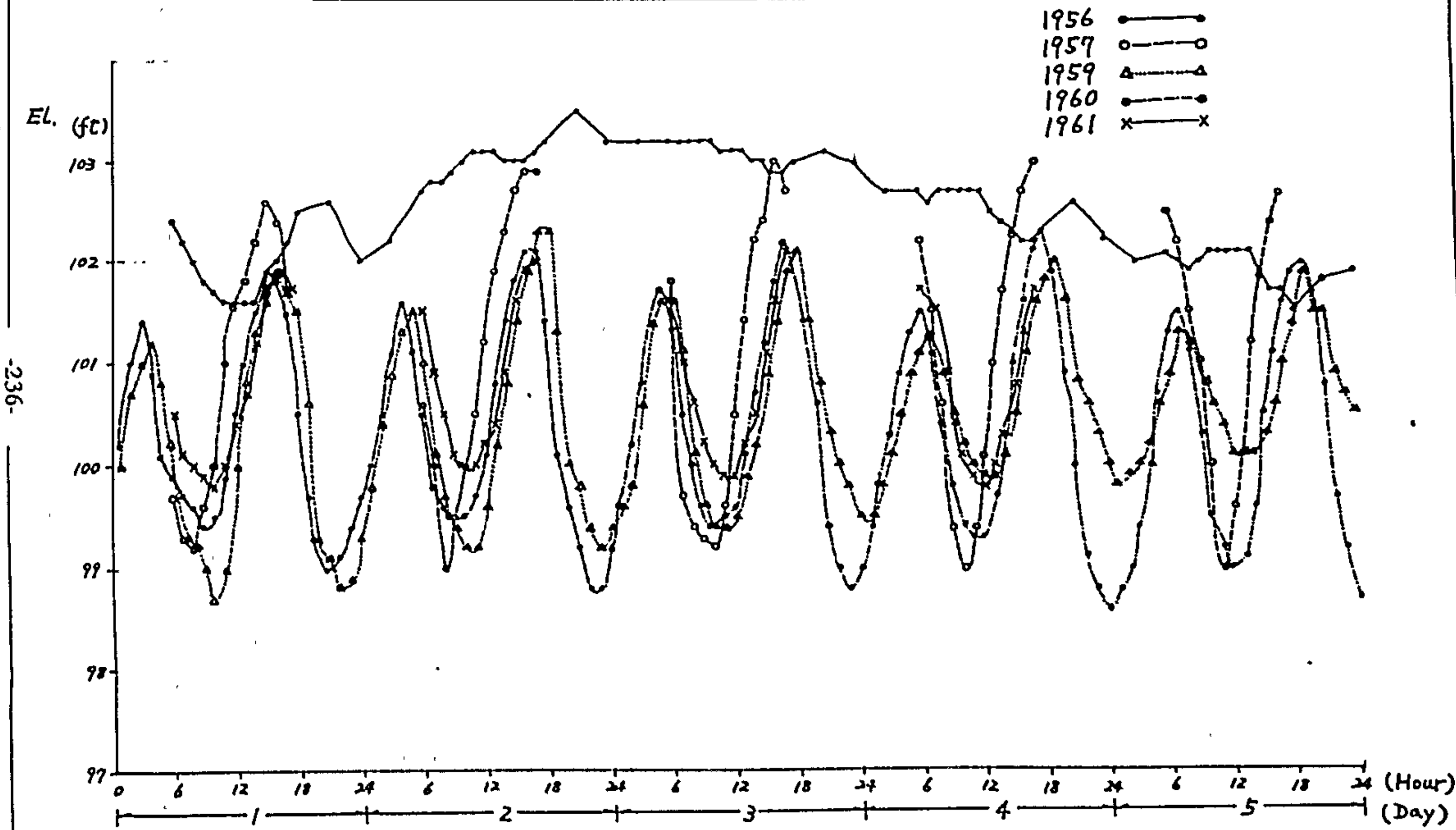
(Fig.5)

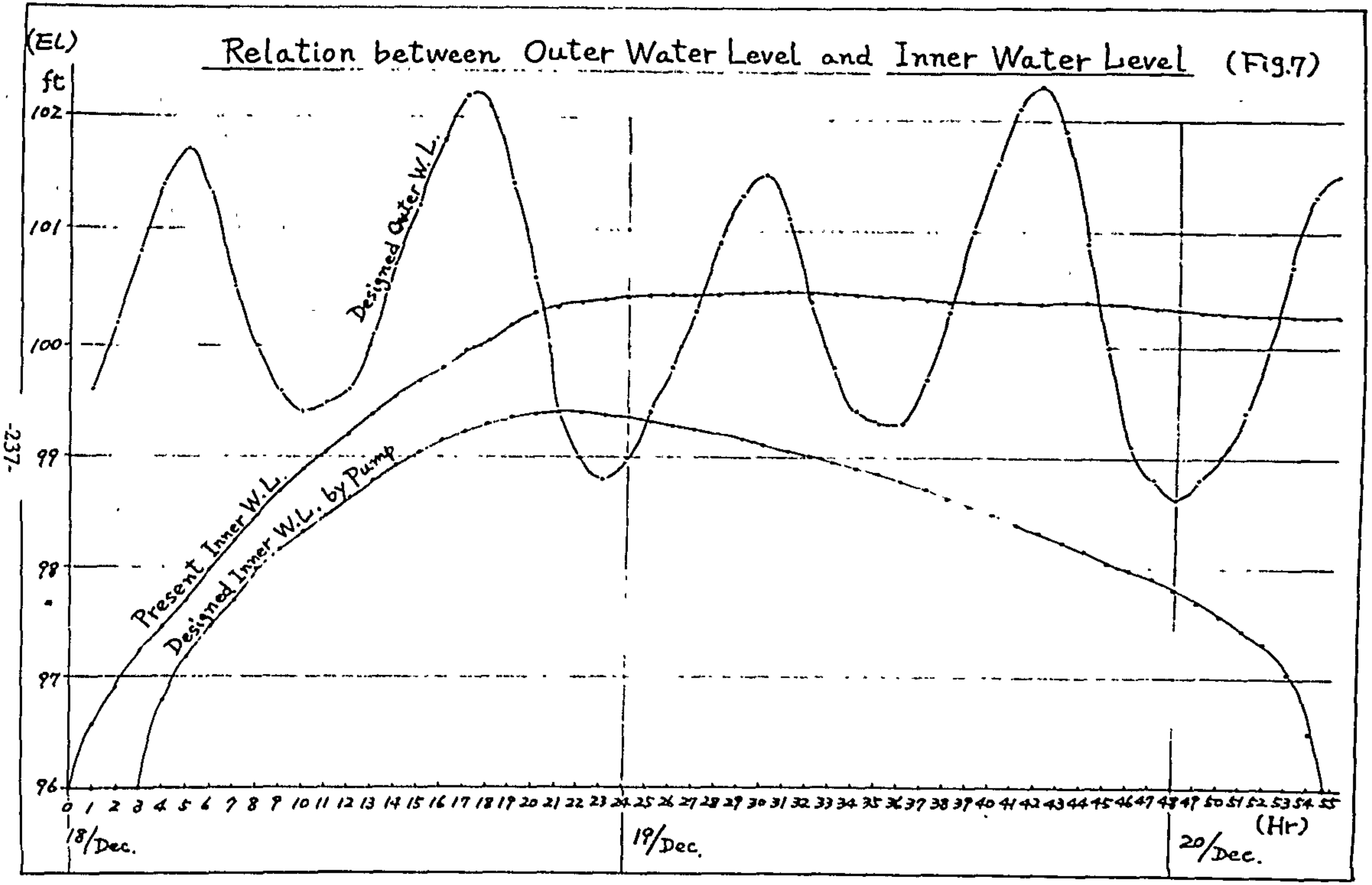
Discharge Curve through St. John's Sluice



(Fig. 6)

Outer Water Level at St. John's Sluice





Relation between Outer Water Level and Inner Water Level (Fig.7)
(EL)
ft
102
101
100
99
98
97
96
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 31 32 33 34 35 36 37 38 39 40 41 42 43 44 45 46 47 48 49 50 51 52 53 54 55
(Hr)
18/Dec.
19/Dec.
20/Dec.
Designed Outer W.L.
Present Inner W.L.
Designed Inner W.L. by Pump

OROPOUCHE (III) March, 1968

MOVEMENT

31, January (1) Travelled by boat along St. John's river to sea. Then, up to Trinidad Sluice on the Godeneau River and back by boat with Mr. M. Antonio (Superintendent South).

(2) Visited Puzzle Island on the Main Drain by boat.

8, February Visited a catchment area of the Blackwater Channel and investigated the mouth of the channel to the Main Drain and culverts with flap gates with Mr. E. Wyke and Mr. Antonio.

INTRODUCTION

At the end of January 1968, rain fell in the Oropouche Area for a few days notwithstanding the dry season. Farmers in this lower area suffered losses, to their crops; peas, beens, melons, tomatoes and etc, from flood due to rain and high tide. A newspaper reported that total amount of the damage was $40,000 T.T. Therefore, the Government will have to pay compensation money to the farmers for the damage. They are proposing that pumps for drainage be set as soon as possible.

In my reports, Oropouche (I) and (II), drainage schemes employing the use of pumps were already proposed for the catchment area of Trinidad Sluice and St. John's Sluice, because such a damage could easily be anticipated due to the low discharge capacity of gates in high tide.

The drainage scheme for the catchment area of the Blackwater channel will be studied on this report.

DESCRIPTION OF THE AREA STUDIED

1) Present condition of the area to be drained

The lower parts of the Blackwater channel was cut straight.

The Main Drain was built in 1963 and two culverts at the much of the Blackwater Channel to the Main Drain were built in 1967. They are corrugate pipes of 42 inches in diameter, 80 feet long and 98.5 feet in elevation each.

The basis on which the diameter and the elevation were determined is not clear. It cannot be denied however, that the discharging capacity is insufficient. The outer water level is very high. Then flooding occurred frequently in this lower area.

This catchment area is situated between the Carampo River and the Godeneau River the lowest of which is 100 feet in elevation.

Description of the Blackwater Channel at the confluence with the Main Drain (cf. Fig. 1)

Name of River	Location	Difference		Length		Catchment Area	
Blackwater Channel	200' - 90'	110'	33.53m	4.37 miles	7.03km	$8.33km^2$	2,058.4acres

2) Flooding area and volume (cf. Fig. 1 and 2)

Elevation	Depth	Area	Mean Area			Volume	Total Volume		
ft	ft		ha	acre	ha	acre-ft	acre-ft	ft^3	m^3
99 >	0	0	0	0	0	0	0	0	0
100 >	1	49.4	20.0	24.7	10.0	24.7	24.7	1,076,000	30,000
101 >	2	181.6	73.5	115.5	46.8	115.5	140.2	6,107,000	173,000
102 >	3	415.1	168.0	298.4	207.0	298.4	438.6	19,105,000	541,000
103 >	4	675.1	273.2	545.1	421.8	545.1	983.7	42,850,000	1,214,000
104 >	5	746.0	301.9	710.6	627.9	710.6	1,694.3	73,804,000	2,090,000
105 >	6	779.9	315.6	763.0	736.8	763.0	2,457.3	107,040,000	3,031,000

3) Flood discharge

In order to calculate the flood discharge for this scheme, the maximum daily rainfall once in five years in the Oropouche Area which was calculated as 4.15 inches by F.A.O. Report, is adopted.

4) Culvert Details

diameter	42" = 1.07m
length:	80' = 24.38m
sill level of pipe:	98.5 ft
top level of pipe:	102.0 ft

Calculation in case of open channel (having free water surface) could only be carried out because the internal water level is always below the top of the pipes as mentioned later. Cases of orifice and inverted siphon never occur on these culverts.

Determination of sill level of pipe

As a result of a recent survey, top levels of embankment of the Main Drain are 94.92 ft and 94.42 ft and sill levels of culverts are 86.84 ft and 86.87 ft based on T.B.M. of 100 ft.

On the other hand, according to a cross section of the Main Drain at the mouth of the Blackwater Channel, settled top level of embankment is shown 106.35 ft based on T.G.R. Datum.

Therefore, sill level of the culverts due to T.G.R. will be

$$106.35 - \frac{(94.92 + 94.42)}{2} + \frac{(86.84 + 86.87)}{2} = 98.54 \fallingdotseq 98.5 \text{ ft}$$

5) Outer Water Level

According to the cross section of the Main Drain at the mouth of the Black-water Channel by Dutch Consultant in 1960, the designed flood stage is 102.85'. The designed flood stage at the same position on F.A.O. Report is 103.83 ft due to the maximum tidal level at the mouth of the Main Drain which was recorded as 100.35'.

Design for drainage

Design depends on a detailed study of a relation between internal water level and outer water level to determine whether a flood could be drained by installing more culverts with flap gates, or not. There is, however, no records for the outer water levels which is the water level of the Main Drain, because it is a new cut four years old. Although the outer water level is surely determined by flood discharge from the upper catchment area and backwater due to variation of tidal level at sea, it will take much time to calculate.

It is therefore recommended that a pump be employed in addition to drainage due to gates as the only reliable means.

The catchment area is comparatively small and there is an adequate location for a pumping station at the mouth of the Blackwater Channel.

A design for drainage will be undertaken at this promises.

Calculation of designed flood discharge (cf. Fig. 3)

1) Flood arrival time at the mouth of the Blackwater Channel

$$H = 33.53\text{m}$$
$$L = 7{,}030\text{m}$$
$$V = 72\frac{(H)}{(L)}^{0.6}\ \text{km/hr)}$$
$$= 72\frac{(33.53)}{(7{,}030)}^{0.6} = 72(0.00477)^{0.6} = 72 \times 0.0405 = 2.92\ (\text{km/hr})$$
$$T = L/V = \frac{7.03}{2.92} = 2.41\ \text{hr}$$

2) Mean hourly rainfall intensity until flood arrival which is

$$R = 4.15\ \text{inches} = 105.3\ \text{mm}$$
$$r = \frac{R}{24}\frac{(24)^{2/3}}{T} = \frac{105.3}{24} \times \frac{(24)^{2/3}}{241} = 4.39 \times 4.63 = 20.3\text{mm}$$

3) Maximum flood discharge which is Q

$$Q = \frac{1}{3.6}\,\text{frA}$$
$$= \frac{1}{3.6} \times 0.4 \times 20.3 \times 8.33 = 18.79 \doteqdot 18.8\ \text{m}^3/\text{s}$$

4) Flood continuing time after its peak which is λ T

$$\lambda = \frac{Rf'A}{1.8\ TQ} - 1 = \frac{105.3 \times 0.6 \times 8.33}{1.8 \times 2.41 \times 18.8} - 1 = 6.45 - 1 = 5.45$$

$$\lambda T = 5.45 \times 2.41 = 13.13 \text{ hr.}$$

5) Whole flood discharge which is ΣQ

$$\Sigma Q = 1{,}000\ Rf'A = 1{,}000 \times 105.3 \times 0.6 \times 8.33 = 526{,}300\ m^3$$

Assuming that the whole flood discharge in the lower land, the flood level will be 102.0 ft on Fig. 2 On the other hand the maximum outer water level is nearly 103.0 ft. It means that the maximum outer water level is still 1.0 ft higher than the maximum flood level. Under these circumstances, drainage by gravity is of course impossible.

CALCULATION OF DISCHARGE FROM THE CULVERTS (CORRUGATE PIPES)

1) Loss of head at inlet

$$hi = fi\frac{V_2}{2g} + \frac{V_2^2 - V_1^2}{2g}$$

hi: Loss of head at inlet

fi: Coefficient of loss which is now 0.2

V_1: Velocity before inflow

V_2: Velocity after inflow

g: Acceleration due to gravity which is 9.8 m/sec^2

Assuming that friction loss in corrugate pipe: hf = 0.055m

$$l = 24.38m \quad \therefore\ I = \frac{hf}{l} = 0.00226$$

Assuming that loss of head at inlet be 0.04m

$$H = 1.07 - 0.04 = 1.03m$$

$$\therefore\ H/r = 1.03/0.535 = 1.925 \quad \therefore\ R/r = 0.554 \text{ (On a hydraulic table)}$$

$$R = 0.554 \times 0.535 = 0.296m$$

$$V_1 = 0m/s$$

$$V_2 = \frac{1}{n} R^{2/3} I^{1/2} = 40 \times (0.296)^{2/3} \times (0.00226)^{1/2} = 40 \times 0.444 \times 0.0475$$

$= 0.844$ m/s (n: Coefficient of roughness of corrugated pipe which is 0.025)

$$\therefore\ hi = 0.2 \times \frac{(0.844)^2}{2 \times 9.8} + \frac{(0.844)^2}{2 \times 9.8} = 1.2 \times 0.036 = 0.0432 \fallingdotseq 0.04m \ (\fallingdotseq 0.13')$$

This result coincides very nearly with the assumed value

102.00' - 0.13' = 101.87'

2) Loss of head due to friction in corrugate pipe

$$hf = \frac{n^2 Vm^2 l}{R_m{}^{4/3}}$$

hf: Loss of head due to friction

n: Coefficient of roughness which is 0.025

Vm: Mean velocity in pipe V_1: Velocity at inlet of pipe

l: length of pipe V_2: Velocity at outlet of pipe

Rm: Mean hydraulic radius of pipe

Assuming that hf = 0.55m as mentioned above

H = 1.030 - 0.055 = 0.975m

H/r = 0.975/0.535 = 1.822 ∴ R/r = 0.583 (on a hydraulic table)

R = 0.583 x 0.535 = 0.312m

V_1 = 0.844 m/s

$$V_2 = \frac{1}{n} R^{2/3} \quad I^{1/2} = 40 \times (0.312)^{2/3} \times (0.00226)^{1/2} = 0.874 \text{ m/s}$$

$$\therefore \; Vm = \frac{V_1 + V_2}{2} = \frac{0.844 + 0.874}{2} = 0.859 \text{ m/s} \qquad Rm = \frac{R_1 + R_2}{2} = \frac{0.296 + 0.312}{2} = 0.304\text{m}$$

$$\therefore \; hf = \frac{(0.025)^2 \times (0.859)^2 \times 24.38}{(0.304)^{4/3}} = \frac{0.000625 \times 0.738 \times 24.38}{0.2044} = 0.0550\text{m} \; (=0.18')$$

This result coincides with the assumed value. Then the water level will be

101.87' - 0.18' = 101.69'

3) Calculation of discharge at outlet of pipe

H/r = 0.975/0.535 = 1.822 ∴ A/r_2 = 3,001 (on a hydraulic table)

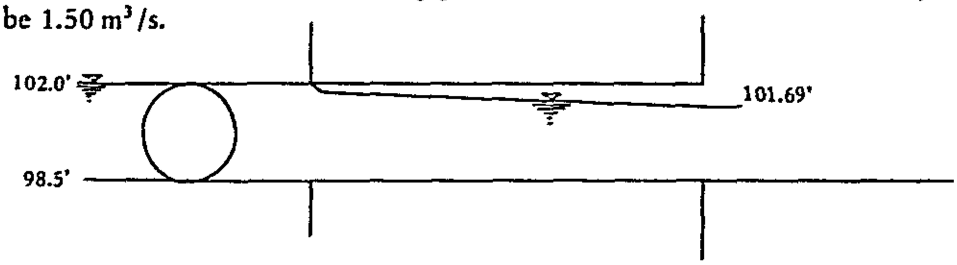
$A = 3.001 \times (0.535)^2 = 0.859 \text{ m}^2$

$Q = AV_2 = 0.859 \times 0.874 = 0.751\text{m}^3/\text{s}$

As there are two pipes, total discharge will be

$\Sigma Q = 2Q = 2 \times 0.751 = 1.50\text{m}^3/\text{s}$

When the water level at the inlets of the pipes is 102.0 ft and water can flow freely, the total discharge will be 1.50 m^3/s.



DETERMINATION OF CAPACITY OF PUMP

Even if the maximum possible discharge from the pipes can continue during a whole day, the total discharge will be only 131,300m^3.

(Total Q = 1.52m^3/s x 24 x 60 x 60 s = 131,300m^3/day)

On the other hand, the total flood discharge is calculated as 526,300m^3/day. Therefore, the minimum discharge with pump will be

526,300 - 131,300 = 395,000 ≑ 400,000m^3

So as to discharge this volume during a day, a total capacity of pump will be,

$$q = \frac{400{,}000}{24 \times 60} = 278 \text{ m}^3/\text{min}$$

actual head:	103' - 99' = 4'
total head requried:	≒ 6'
type of pump:	axial flow pump
combination of pumps	choose (1) or (2)

1) 2 sets x 140 m^3/min (= 4,950ft^3/min)

axial horse power:	40 kW
	350 r.p.m.
diameter of suction end:	1,200 mm (=48")

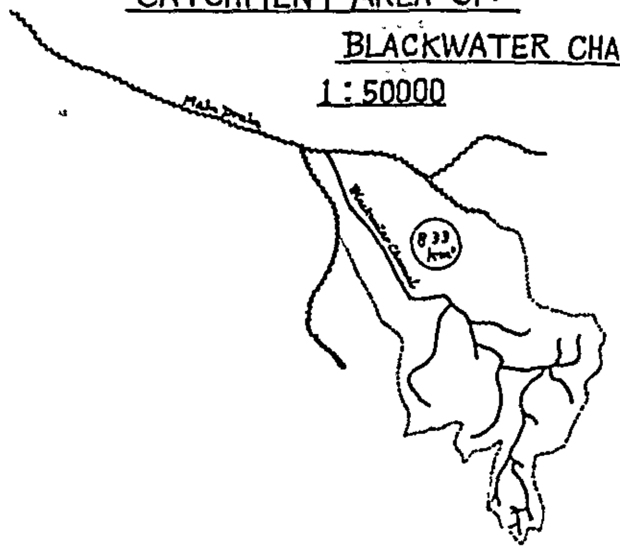
2) 3 sets x 95 m^3/min (=3,360ft^3/min)

axial horse power:	40 kW
	350 r.p.m.
diameter of suction end:	800 mm (=32")

Junichi Kitamura

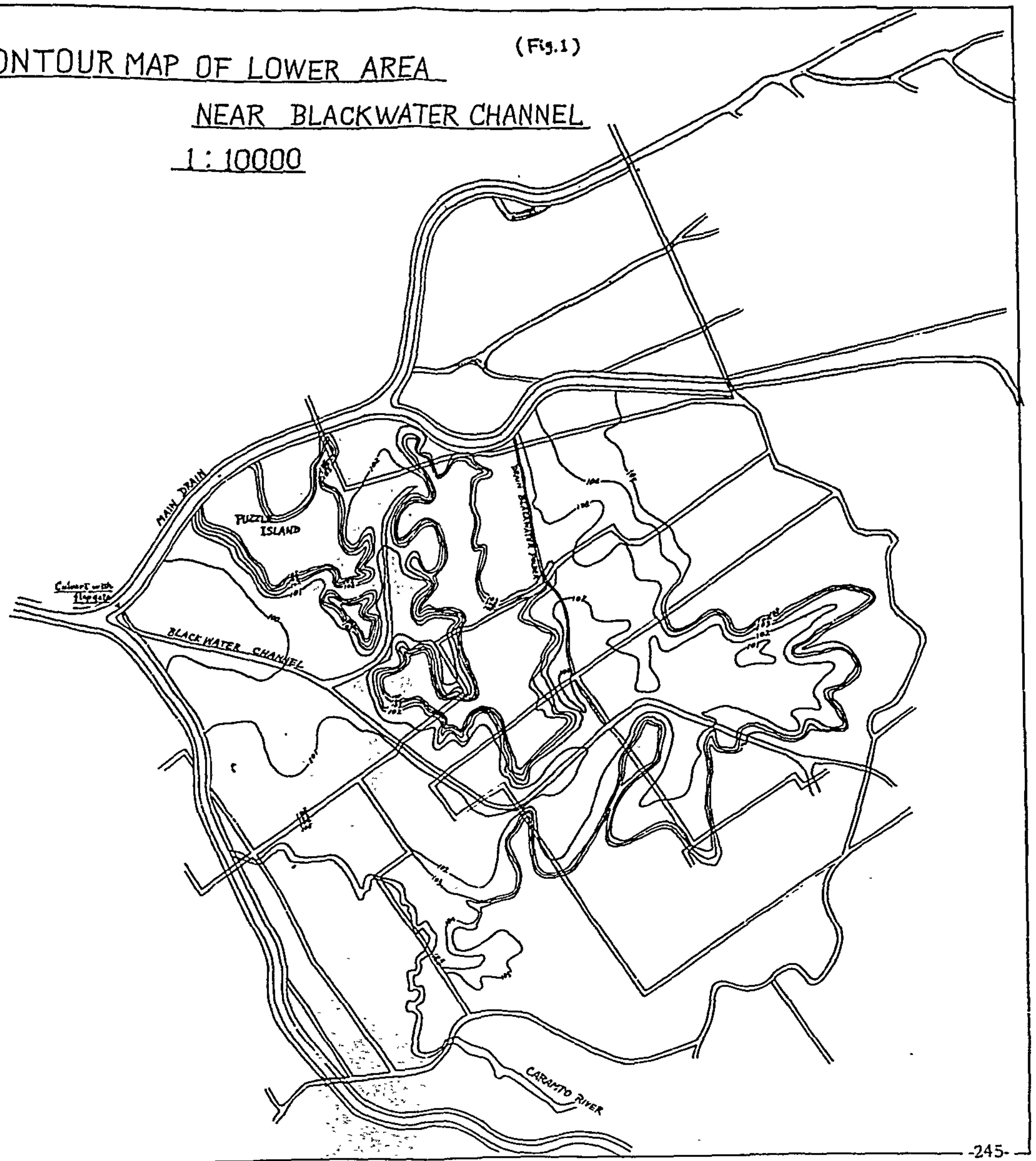
CATCHMENT AREA OF BLACKWATER CHANNEL (Fig.1')

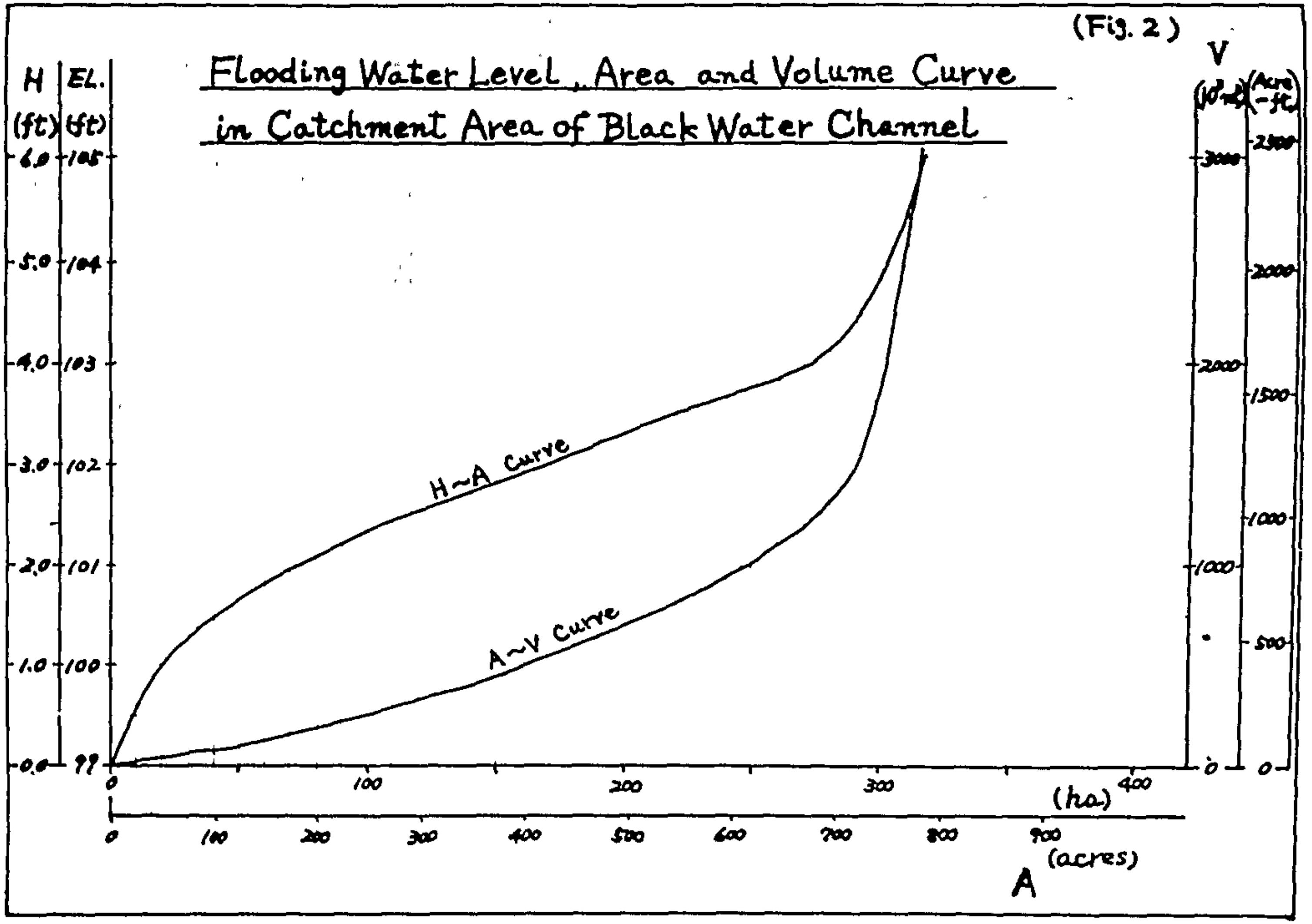
1 : 50000



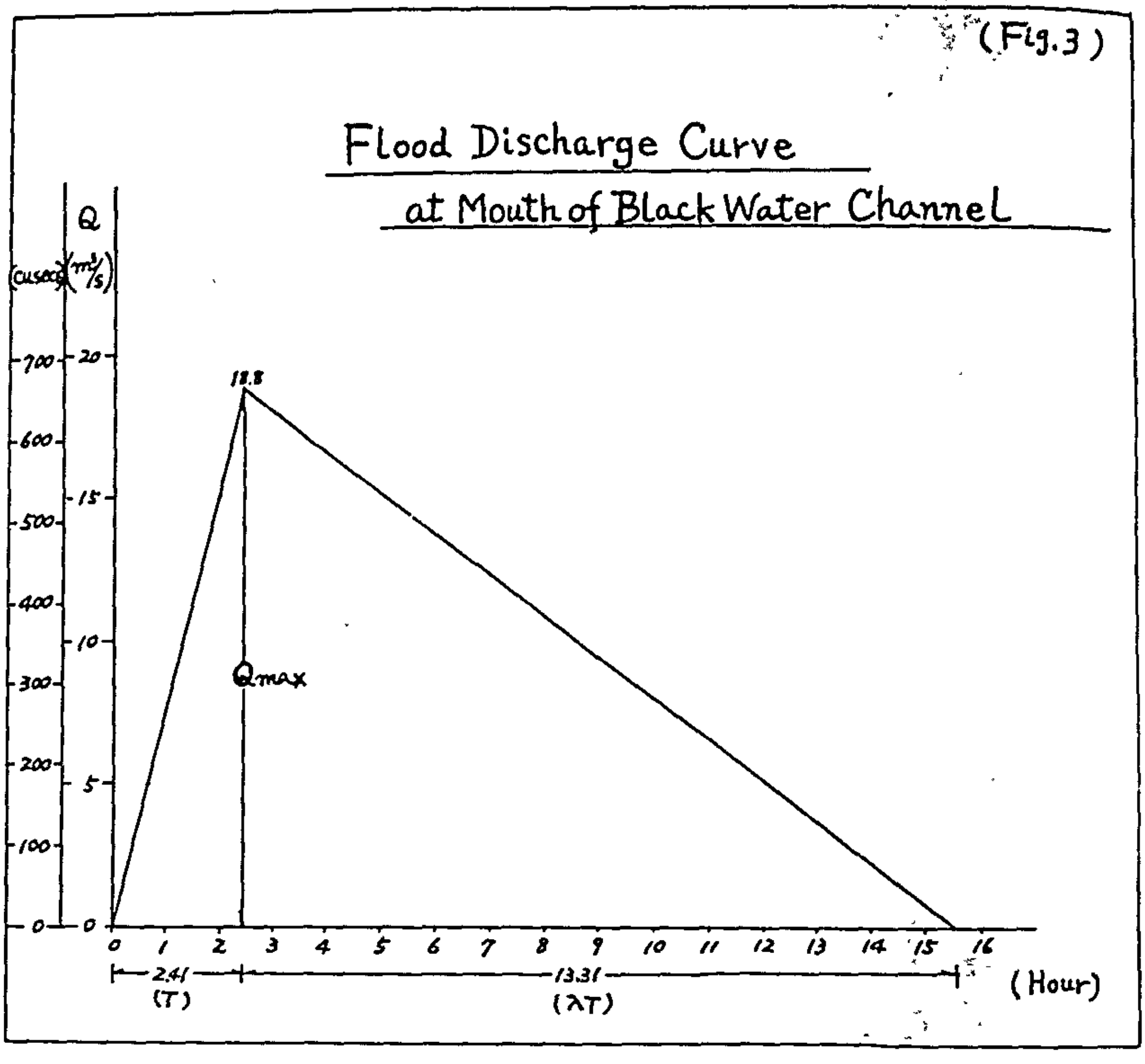
CONTOUR MAP OF LOWER AREA NEAR BLACKWATER CHANNEL (Fig.1)

1 : 10000



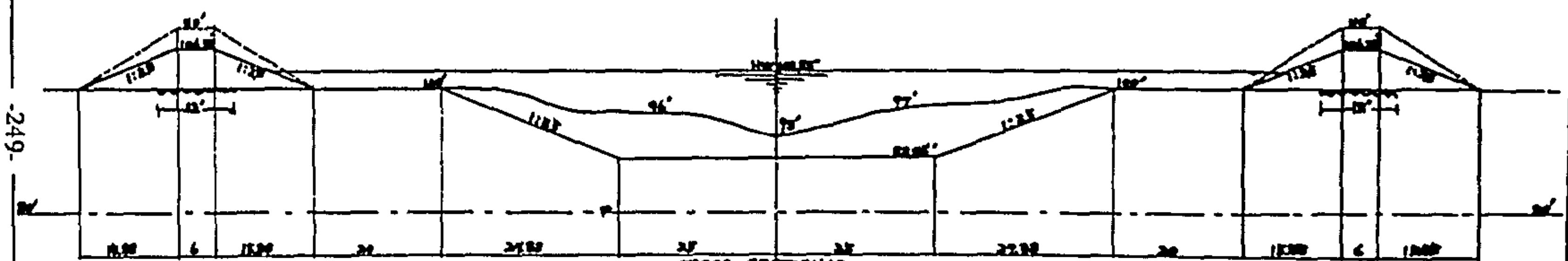

(Fig. 2)
Flooding Water Level, Area and Volume Curve
in Catchment Area of Black Water Channel
H (ft)
EL. (ft)
6.0
5.0
4.0
3.0
2.0
1.0
0.0
105
104
103
102
101
100
99
H~A Curve
A~V Curve
V
3000
2000
1000
0
2500
2000
1500
1000
500
0
0
100
200
300
400
(ha)
0
100
200
300
400
500
600
700
800
900
(acres)
A

(Fig.3)


Flood Discharge Curve
at Mouth of Black Water Channel
Q
(cusecs)
(m³/s)
700
600
500
400
300
200
100
0
20
15
10
5
0
18.8
Qmax
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
2.41
(T)
13.31
(λT)
(Hour)

MAIN DRAIN AT MOUTH OF BLACK WATER CHANNEL

(Fig.4)



CROSS-SECTION 12
SCALE 1:200

Distance from Trinidad Embankment : 13812'
Width of Bottom : 50'
Side Slopes : 1 : 2.5
$i = 0.000181$
$K = 67$

OROPOUCHE (IV) June, 1968

MOVEMENT

30, Apr.	Requested survey for contour mapping in the vicinity of the mouth of Black-water Channel
10, May	Visited there to select the site for a pumping station with Mr. C. Taylor, Mr. W. Riley and Mr. M. Antonio
11, May	Drew the contour map from the result of survey
15, May	Began a design of a pumping station, for the catchment area of Blackwater Channel
18, May	Requested for an estimated cost of pumps and accessories from Neal & Massy Ltd.
21, May	Finished hydraulic calculations for pumping station
22, May	Investigated the site again with Mr. E. Wyke and requested a soil test in at least two locations from the soil section of the Ministry of Works. (sounding with a cone penetrometer; physical and dynamic test)
4, June	Completed the layout plan and construction details plans of Pumping Station
6, June	Inspected one of Caroni Co. Limited Pumping Station at Caroni Estate, with Mr. E. Wyke and Mr. W. Riley
7, June	Discuss supply of electricity for the pumps with Mr. Antonio and arranged with him to request from the T & T.E.C. an estimated cost of electricity supply (installation and consumption)
15, June	The soil test field work was completed and laboratory has since been started.
20, June	Completed dynamic calculations for the station except for that of the house.
13, July	Completed computation for quantity of materials and estimate for the cost for foundation works

Object of the Design

As stated in the report (III), the lands in lower reaches of Blackwater Channel is one of the three lower areas which were intercepted by the embankment of Main Drain in Oropouche Lagoon. The urgent countermeasure for the area must be set up so as to eliminate damage due to flood. For this purpose, the design of a pumping station for the area will be carried out in this report and the next.

Description of the Lower Land studied (cf. General Plan)

This land is situated near Puzzle Island and is partly surrounded by the embankment of Main Drain. Blackwater Channel flows through the land and joins Main Drain by two culverts under the dyke. The land is relatively very flat with an approximate T.G.R datum of 97 ft - 102 ft. The top of the embankment is 103 - 105 ft in elevation. The vegetation consist mainly of rice and vegetables with wild grass in the lowest areas. The soil seems to be consist of silty or clayey alluviam. The Blackwater Channel is about 25 - 40 ft wide and its lowest bed level is 94 ft. The two culverts mentioned before, which were made in 1967, are each 3.5 ft in diameter, with automatic control gates.

Design of Pumping Station

1. Determination of the location (cf. General Plan)

(i) Topography
The lowest land in the vicinity of the confluence of Blackwater Channel and Main Drain is desirable. However, the land along the embankment of Blackwater Channel is not suitable for the location, because the vibrations by pump-operation for a long time may result in the breaking of the embankment.

(ii) Foundation
Ideally the foundation should be located in an area of uniform geological formation and good bearing capacity. In this area 250 yds by 250 yds, it is expected that conditions would be uniform, however, sufficient foundation works must be undertaken as the bearing capacity of the soil appears to be somewhat low.

(iii) Facility of motive power
Electricity is available because, there are suitable transmission lines along the road about 0.7 mile away from the mouth of Blackwater Channel.

From the above points of views, the location is determined as shown on the general plan.

2. Type, scale and number of pumps

(i) Type; axial flow pump with horizontal axis (cf. Report III)
(ii) Scale and number: $Q = 3 \text{ sets} \times 93 \text{ m}^3/\text{min.} = 278 \text{ m}^3/\text{min.}$
$= 3 \text{ sets} \times 1.54 \text{ m}^3/\text{sec.}$
(iii) Diameter of pumps

$$D = 1{,}000 \sqrt{\frac{4}{\pi} \times \frac{Q}{V}}\, 1{,}000 \sqrt{\frac{4 \times 1.54}{3.14 \times 3.0}} \doteqdot 800 \text{ m}$$

D: diameter of pump

Q: discharge which is 1.54 m^3/sec

V: mean velocity elocity in pipe which is adopted as 3.0 m/sec (2.0 - 3.2 m/sec for axial flow pump)

During flood the pumping operation must be smoothly continued for twentyfour hours each day in accordance with the variation of rate of inflow. On the other hand the pumping operation for drainage in dry season and for small flood must also be considered from an economic stand point. Because of this three sets of pumps with the same capacity are recommended.

3. Head of pump

(i) Determination of floor level to install pumps
As mentioned in report III, the highest inner water level is 102 ft, in elevation, therefore the floor level is set at 103 ft and the center of the pump at 105 ft.

(ii) Determination of designed lowest inner water level.
The lowest ground level in the cultivated land can be regarded as 98 ft. Then, the lowest inner water level is set at 97.5 ft in front of the intake of the pumping station.

(iii) Desinged highest outer water level.
The designed flood stage of the Main Drain at the confluence of the Blackwater Channel, is shown as 102.85 ft on the cross section plotted by Dutch Consultants.

This value is therefore adopted as the highest outer water level at the mouth of the culverts leading from the pumping station.

(iv) Calculation for loss of head in each pump

(1) Loss of head at inlet of suction pipe (hi)

$$hi = Cib\frac{V^2}{2g}$$

$$V = Q/\frac{\pi}{4}d^2$$

hi: Loss of head at inlet of suction pipe

Cib: Coefficient in case of bell-mouth which is 0.05 - 0.10

V: Mean velocity in suction pipe

g: Acceleration due to gravity which is 9.8 m/sec^2

d: Diameter of suction pipe

$$V = \frac{1.54}{\frac{3.14}{4} \times (0.80)^2} = \frac{1.54}{0.785 \times 0.64} = \frac{1.54}{0.502} = 3.07 \text{ m/s}$$

$$hi = 0.10 \times \frac{(3.01)^2}{2 \times 9.8} = \frac{0.906}{19.6} = 0.046\text{m}$$

(2) Loss of head in straight pipes (hp)

$$hp = \frac{L}{D} \cdot \frac{V^2}{2g}$$

λ: Coefficient of friction which is $1.5\left(0.02 + \frac{1}{2{,}000D}\right)$

L: Total length of straight parts in suction and delivery pipes which is 6.7m.

D: Diameter of pipe which is 0.8 m

$$hp = 1.5(0.02 + \frac{1}{2{,}000 \times 0.8}) \times \frac{6.7}{0.8} \times \frac{(3.01)^2}{2 \times 9.8}$$

$$= \frac{1.5 \times 0.021 \times 6.7 \times 9.06}{0.8 \times 19.6} = \frac{1.912}{15.68} = 0.122m$$

(3) Loss of head in bending pipes (hb)

In the case of one (1) 45° bend

$$hb = 3/4\ C_b \frac{V^2}{2g} \quad C_b\text{: Coefficient for 90° bends in pipes which is 0.2 - 0.3}$$

$$= 3/4 \times 0.2 \times \frac{(3.01)^2}{2 \times 9.8} = \frac{0.6 \times 9.06}{4 \times 19.6} = \frac{5.44}{78.40} = 0.069m$$

In the case of two (2) 45° bends

$\therefore \quad hb = 2 \times 0.069 = 0.138m$

(4) Loss of head due to gradual enlargement at the end of pipe (hc)

$$hc = Cc \times \frac{(V_1 - V_2)^2}{2g}$$

Cc: Coefficient for gradual enlargement in pipes which is 0.1 - 0.2

V_1: Velocity before enlargement

V_2: Velocity after enlargement

$D_1 = 0.8m \qquad \therefore\ V_1 = 3.0\ \dfrac{1.54}{\frac{3.14}{4} \times (1.15)^2} = \dfrac{1.54}{0.785 \times 1.323} = \dfrac{1.54}{1.039} = 1.48m/s$

$$hc = 0.15 \times \frac{(3.01 - 1.48)^2}{2 \times 9.8} = \frac{0.15 \times (1.53)^2}{19.6} = \frac{0.351}{19.6} = 0.018m$$

(5) Loss of head due to a check value (hn)

$$hn = Cn \times \frac{V_2{}^2}{2g} \quad \text{Cn: Coefficient for check valves which is 0.5 - 1.0}$$

$$= 0.75 \times \frac{(1.48)^2}{2 \times 9.8} = \frac{1.64}{19.6} = 0.084m$$

(6) Loss of head due to remaining velocity (hr)

$$hr = \frac{V_2{}^2}{2g} = \frac{(1.48)^2}{2 \times 9.8} = \frac{2.19}{19.6} = 0.112m$$

Thus total loss of head h = hi + hp + hb + hc + hn + hr = 0.520m = 1.71ft

(v) Actual head

	at intake	at sumps	• The calculation for loss of head in connecting canals will be shown below. Only the results are shown on this table.
Inner water level	(a-1) 102.00ft (a-2) 97.50	= 102.00 ft 97.38	
Outer water level	at outlet of pipes (b) 103.30 ft	at mouth of culverts 102.85 ft	

Thus actual head at suction side = 105.00 ft - 97.38 = 7.62 ft

at delivery side = $\frac{103.30 - 105.00 = -1.70}{\text{Balance} \quad 5.92}$

As the pumps must be positioned high, the pipes will function as siphons.

(vi) Total head required:

Total head = Actual head + total loss of head = 5.92 ft + 1.71 ft = 7.63 ft

(vii) Calculation for loss of head in connecting canals (cf. general plan)

a) From intake to sump

a-1) In the case of the highest inner water level which is 102.00 ft
(Three pumps must be operated)
Although the calculation was carried out, the loss of head was, however, negligible because of low velocity.

(i.e.) Velocity at intake V = Q/A A = H(B+nH)

$A = 1.81\,(11.0 + 2.5 \times 1.81) = 28.11 m^2$

$Q = 4.63 m^3/s$

$V = \frac{4.63}{28.11} = 0.165\ m^3/s$

A: Sectional area
H: Water depth
n: Slope which is 1:25
B: Width of river bed

Velocity at sump

$A = HB = 1.82 \times 10.0 = 18.20 m^2$

$Q = 4.63$

$V = \frac{4.63}{18.20} = 0.254 m^3/s$

Thus the details of the calculation can be omitted.

a-2) In case of the lowest inner water level which is 97.50 ft
(One pump is operated)

(1) At inlet from Blackwater Channel Change of velocity head (hv)

$hv = \frac{V_2{}^2 - V_1{}^1}{2g}$

$= \frac{(0.28)^2 - 0}{2 \times 9.8} = 0.004m$

V_1: Velocity before inlet is 0 m/s

V_2: Velocity after inlet

$V_2 = \frac{Q}{A_2} = \frac{1.54}{0.45\,(11.0 + 1.12)} = 0.28 m/s$

(Assuming that H_2 = 1.48' = 0.45m)

Loss of head due to inflow (hi)

$hi = fi \frac{V_2{}^2}{2g} = 0.2 \times \frac{(0.28)^2}{2 \times 9.8} = 0.0008m$

fi = 0.2

$\therefore$ Total head loss = hv + hi = 0.004 + 0.0008 = 0.0048m = 0.016' ÷ 0.02'

97.50' - 0.02' = 97.48'

H_2 = 97.48' - 96.00 = 1.48'

Thus, this result coincides with the assumed vlue.

(2) At intake gate

Loss of head due to friction in canal between the inlet and the intake gate (hf)

V_1 = 0.28 m/s R: Hydraulic radius Assuming that H_2 = 1.40' = 0.43m

A_2 = 0.43 (11.00 + 2.5 x 0.43) = 5.19m^2 P : Wetted perimeter

$\therefore$ $V_2 = Q/A_2$ = 1.54/5.19 ÷ 0.30m/s n : Coefficient of roughness

$$R_1 = \frac{A_1}{P_1} = \frac{5.45}{2 \times 1.21 + 11.0} = \frac{5.45}{13.42} = 0.41\text{m}$$ L: Distance between inlet and intake gate.

$$R_2 = \frac{A_2}{P_2} = \frac{5.19}{2 \times 1.16 + 11.0} = \frac{5.19}{13.32} = 0.39\text{m}$$

$$Vm = ½ (V_1 + V_2) = 0.29 \text{ m/s}$$

$$Rm = ½ (R_1 + R_2) = 0.40\text{m}$$

$$hf = \frac{n^2 V^2 mL}{Rm\ 4/3} = \frac{(0.05)^2 \times (0.29)^2 \times 36.6}{(0.40)\ 4/3} = 0.026\text{m} = 0.085'$$

$\therefore$ 97.48' - 0.085' = 97.40' H_2 = 97.40' - 96.00' = 1.40'

Thus, the result coincides with the assumed value

Change of velocity head (hv)

Assuming that H_2 = 1.39' = 0.41m $V_2 = Q/A_2 = Q/BH_2 = \frac{1.54}{0.42 \times 10.0} = \frac{1.54}{4.20}$ = 0.37 m/s

$$hv = \frac{V_2^2 - V_1^2}{2g} = \frac{(0.37)^2 - (0.30)^2}{2 \times 9.8} = 0.0024\text{m}$$

= 0.008' = 0.01' 97.40' - 0.01' = 97.39' $\therefore$ H_2 = 97.39' - 96.00' = 1.39'

Loss of head due to reduction of section (hc)

$$hc = fc \frac{V_2^2 - V_1^2}{2g}$$ fc = 0.5 (Sharp-corner)

= 0.5 x 0.0024 = 0.0012m = 0.004'

$\therefore$ Total head loss = hv + hc = 0.008' + 0.004' = 0.012' = 0.01'

$\therefore$ 97.40' - 0.01' = 97.39' H_2 = 97.39' - 96.00' = 1.39'

Loss of head due to pier (hp)

$$hp = \frac{Q^2}{2g} \left[\frac{1}{C^2 b_2^2 (H_1 - hp)^2} - \frac{1}{b_1^2 H_1^2} \right]$$

C: Coefficient of pier-section = 0.92

b_1: Width of canal before intake gate

b_2: Width of canal exclusive of widths of piers

H_1: Water depth before gate

Assuming that hp = 0.003m (= 0.01')

$$hp = \frac{1.54^2}{2 \times 9.8} \left[\frac{1}{(0.92)^2 \times (9.0)^2 (0.42 - 0.003)^2} - \frac{1}{(10.0)^2 \times (0.42)^2} \right]$$

$$= \frac{2.372}{19.6} \left(\frac{1}{11.92} - \frac{1}{17.64}\right)$$

$$= 0.121\ (0.084 - 0.057) = 0.0032m = 0.003m = 0.010'$$

$\therefore$ 97.39' - 0.01' = 97.38'

H_2 = 97.38 - 96.00 = 1.38'

(3) At the bottom of the end of sedimentation basin.

Loss of head due to friction in canal between intake gate and end of sedimentation basin. (hf)

V_1 = 0.37m/s R_1 = 0.39m L = 30' = 9.14m

Assuming that H_2 = 2.39' = 0.73m

$A_2 = BH_2 = 10.0 \times 0.73 = 7.30m^2$

$\therefore$ $V_2 = Q/A_2 = 1.54/7.30 = 0.21m/s$

$$R_2 = A_2/P_2 = \frac{7.30}{2 \times 0.73 + 10.0} = 0.63m$$

$\therefore$ $V_m = ½(V_1 + V_2) = ½(0.37 + 0.21) = 0.29m/s$

$Rm = ½(R_1 + R_2) = ½\ (0.39 + 0.63) = 0.51m$

$$hf = \frac{n^2 Vm^2 L}{Rm^{4/3}} = \frac{(0.015)^2 \times (0.29)^2 \times 9.14}{(0.51)^{4/3}} = 0.0004m$$

Change of velocity head (hv)

$$hv = \frac{V_2{}^2 - V_1{}^2}{2g} = (0.21)^2 = 0.0047m$$

Total head loss = hf + hv = 0.0004 - 0.0047 = -0.0043m = - 0.014'

$\therefore$ 97.38' + 0.014' = 97.39' H_2 = 97.39 - 96.00 = 1.39'

(4) At the crest of the end of sedimentation basin

$$hv = \frac{V_2{}^2 - V_1{}^2}{2g} = \frac{(0.37)^2 - (0.21)^2}{19.6} = 0.0047m = 0.014' = 0.01'$$

$\therefore$ 97.39' - 0.01' = 97.38'

(5) At screen

Loss of head due to screen (hg)

$$V = Q/A = \frac{1.54}{3.0 \times 2.8} = 0.19m/s$$

$$hg = B(1 + \frac{2h_1}{H_1}) \sin A \frac{(t)}{(b)}^{4/3} \frac{V^2}{2g}$$

$$= 2.42\ (1 + \frac{2 \times 0.2}{2.8}) \sin 70^\circ \frac{(0.006)}{(0.06)}^{4/3} \frac{(0.19)^2}{19.6}$$

$= 2.42 \times 1.14 \times 0.93 \times 0.0461 \times 0.0018$

$= 0.00002m \doteqdot 0$

It is negligible

B: Coefficient due to cross section of bars which is 2.42

A: Angle of inclination of screen which is now 70°

t: Thickness of each bar

b: Breadth of opening between bars

h_1: Water depth affected by floating matters

H_1: Water depth before screen

(6) At sump
Loss of head due to friction in canal between the end of sedimentation basin and the centre line of suction pipe.

H_1 = 1.38' = 0.42m
A_1 = 10.00 x 0.42 = 4.20m^2
V_1 = Q/A_1 = 1.54/4.20 = 0.37m/s
$R_1 = A_1/P_1 = \frac{4.20}{2 \times 0.42 + 10.0} = 0.39m$

Assuming that
H_2 = 7.38' = 2.25m
A_2 = 3.00 x 2.25 = 6.75m^2
V_2 = 1.54/6.75 = 0.23m/s
$R_2 = \frac{6.75}{2 \times 2.25 + 3.0} = 0.90m$

$\therefore$ Vm = ½(0.37 + 0.23) = 0.30m/s
Rm = ½ (0.39 + 0.90) = 0.64m
L = 41.0' = 12.3m

$$hf = \frac{n^2 Vm^2 L}{Rm^{4/3}} = \frac{(0.015)^2 \times (0.030)^2 \times 12.3}{(0.64)^{4/3}} = 0.0004m$$

Change of velocity head (hv)

$$hv = \frac{V_2^2 - V_1^2}{2g} = \frac{(0.23)^2 - (0.37)^2}{19.6} = -0.0042m$$

Loss of head due to enlargement of section in sump (he)

$$he = fe\frac{V_1^2 - V_2^2}{2g} = 0.5 \times 0.0042 = 0.0021m \qquad fe = 0.5$$

$\therefore$ hf + hv + he = 0.0004 - 0.0042 + 0.0021 = - 0.0017m = - 0.006' ≑ 0
$\therefore$ 97.38' + 0.00' = 97.38'
$\therefore$ H_2 = 97.38' - 90.00' = 7.38'

b) From outlets of box culverts (Main Drain) to outlets of pipes
Design flood level of Main Drain at confluence with Blackwater Channel is 102.85'

The calculation must be carried out upstream from the outlets of box culverts where is 102.85' in elevation. At that time, three pumps must be operated.

(1) At outlet of box culvert
Loss of head due to friction between inlet of box culvert and outlet of it. (hf)

H_2 = 102.85' - 100.00' = 2.85' = 0.87m
A_2 = BH_2 = 2.0 x 0.87 = 1.74m^2
$V_2 = \frac{Q/2}{A_2} = \frac{2.32}{1.74} = 1.33m/s$
$R_2 = A_2/P_2 = \frac{1.74}{2 \times 0.87 + 2.0} = \frac{1.74}{3.74} = 0.47m$

Assuming that
H_1 = 2.90' = 0.88m
A_1 = BH_1 = 2.0 x 0.88 = 1.76m
$V_1 = \frac{Q/2}{A_1} = \frac{2.32}{1.76} = 1.32m/s$
$R_1 = A_1/P_1 = \frac{1.76}{2 \times 0.88 + 2.0} = \frac{1.76}{3.76} = 0.47m$

$\therefore$ Vm = ½(V_1 + V_2) = ½ (1.33 + 1.32) = 1.33m/s
Rm = ½ (R_1 + R_2) = ½(0.47 + 0.47) = 0.47m
n = 0.015 L = 40' = 12.19m

$$hf = \frac{n^2 Vm^2 L}{Rm^{4/3}} = (0.015)^2 \times \frac{(1.33)^2 \times 12.19}{(0.47)^{4/3}} = 0.013m = 0.043'$$

Change of velocity head (hv)

$$hv = \frac{V_2^2}{2g} = \frac{(1.33)^2}{2 \times 9.8} = 0.09m = 0.30'$$

Loss of head due to sudden enlargement on outflow (he)

$$he = -fe\frac{V_2^2}{2g} \quad fe = 1.0$$

$$= -1.0 \times \frac{(1.33)^2}{2 \times 9.8} = -0.09m = -0.30'$$

$\therefore$ Total head loss = hf + hv + he = 0.043' + 0.30' - 0.30' = 0.043'

$\therefore$ 102.85' + 0.043' = 102.90'

$\therefore$ H_1 = 102.90' - 100.00' = 2.90'

(2) At inlet of box culvert

Loss of head due to friction between beginning of open channel and inlet of box culvert i.e. end of open channel (hf)

V_2 = 1.32m/s

R_2 = 0.47m

n = 0.015

L = 20' = 6.10m

Assuming that:

H_1 = 3.00' = 0.91m

$A_1 = BH_1$ = 4.00 x 0.91 = 3.64m^2

$V_1 = Q/A_1$ = 4.63/3.64 = 1.27m/s

$$R_1 = A_1/P_1 = \frac{3.64}{2 \times 0.9 + 4.0} = \frac{3.64}{5.82} = 0.63m$$

$\therefore$ Vm = ½ ($V_1 + V_2$) = ½ (1.27 + 1.32) = 1.30m/s

Rm = ½ ($R_1 + R_2$) = ½ (0.63 + 0.47) = 0.55m

$$\therefore hf = \frac{(0.015)^2 \times (1.30)^2 \times 6.10}{(0.55)^{4/3}} = \frac{0.00232}{0.451} = 0.0051m = 0.016'$$

Change of velocity head (hv)

$$hv = \frac{V_2^2 - V_2^2}{2g} = \frac{(1.32)^2 - (1.27)^2}{2 \times 9.8} = 0.0066m = 0.022'$$

Loss of head due to centre pier in culvert (hp)

$$hp = \frac{Q^2}{2g}\left[\frac{1}{C^2 b_2^2 (H_1 - hp)^2} \quad \frac{1}{b_1^2 - H_1^2}\right]$$

$b_1 = b_2$ = 4.0m

H_1 = 0.91m

C = 0.92 (cf. p. 225)

Q = 4.63m/s

Assuming that hp = 0.02m

$$hp = \frac{(4.63)^2}{2 \times 9.8}\left[\frac{1}{(0.92)^2 \times (4.0)^2 (0.91 - 0.02)^2} - \frac{1}{(4.0)^2 \times (0.91)^2}\right]$$

$$= \frac{21.44}{19.6}\left(\frac{1}{10.73} - \frac{1}{13.25}\right) = 0.0194 \doteqdot 0.02m = 0.066'$$

$\therefore$ Total head loss = hf + hv + hp = 0.016' + 0.022' + 0.066' = 0.104' $\doteqdot$ 0.10'

$\therefore$ 102.90' + 0.10' = 103.00'

$\therefore$ H_1 = 103.00' - 100.00' = 3.00'

(3) At the end of transition i.e. beginning of open channel.
Loss of head due to friction between beginning of transition and end of it (hf)

$V_2 = 1.27m/s$ Assuming that $H_1 = 8.30' = 2.53m$ $A_1 = BH_1 = 11.0 \times 2.53 = 27.83\ m^2$

$R_2 = 0.63m$ $V_1 = Q/A_1 = 4.63/27.83 = 0.17\ m/s$

$n = 0.015$

$L = 25.0' = 7.50m$ $R_1 = A_1/P_1 = \dfrac{27.83}{2 \times 2.53 + 11.0} = 1.73m$

$\therefore\ Vm = \frac{1}{2}(V_1 + V_2) = \frac{1}{2}(0.17 + 1.27) = 0.72m/s$

$Rm = \frac{1}{2}(R_1 + R_2) = \frac{1}{2}(1.73 + 0.63) = 1.18m$

$$hf = \frac{n^2 Vm^2 L}{Rm^{4/3}} = \frac{(0.015)^2 \times (0.72)^2 \times 7.50}{(1.18)^{4/3}} = 0.0007m = 0.002'$$

Change of velocity head (hv)

$$hv = \frac{V_2^2 - V_1^2}{2g} = \frac{(1.27)^2 - (0.17)^2}{2 \times 9.8} = 0.081m = 0.266'$$

Loss of head due to gradual reduction(hc)

$$hc = fc\frac{V_2^2 - V_1^2}{2g} = 0.2 \times \frac{(1.27)^2 - (0.17)^2}{2 \times 9.8} \qquad fc = 0.2$$

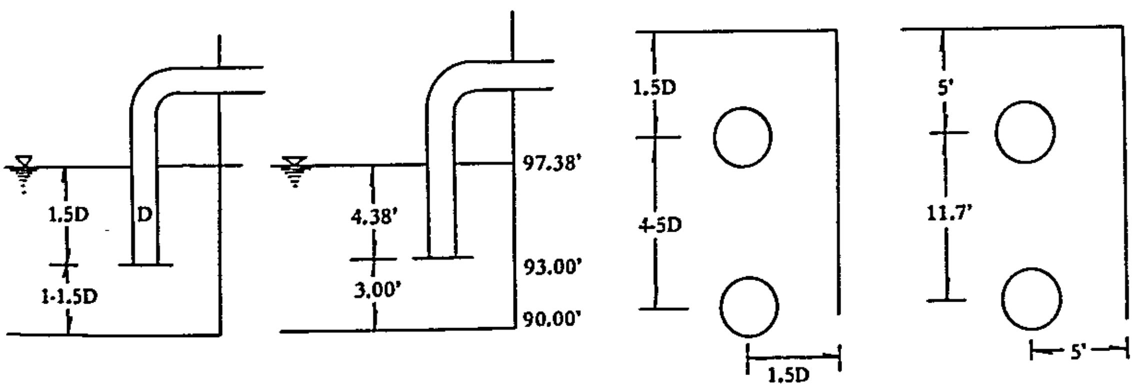
$\therefore$ Total head loss = hf + hv + hc = 0.002' + 0.266' + 0.052' = 0.320'

$\therefore$ 103.00' + 0.32' = 103.32' ≑ 103.3'

$\therefore\ H_1 = 103.3' - 95.00' = 8.30'$

(4) Facilities attached to pumps

(i) Location between sump and suction pipe



D = 800mm = 2.62'
(cf detailed profile)

(cf. detailed plan)

The velocity in sump must be below 1m/sec. $Q = 4.63 = 1.54m^3/s$

In this case, $B = 3.00m^3 = 10.00'$

$$V = \frac{Q}{A} = \frac{Q}{BH} = \frac{1.54}{3.0 \times 2.25} = 0.23m/s \quad H = 2.25m = 7.38'$$

and $< 1.00m/s$

(ii) Delivery pipe

The outlet of delivery pipe must be submerged into the water tank so as to raise efficiency of the pumps. As a result the suction and delivery pipes will be connected as a siphon and inflow of air from the mouth will be prevented. The bell-mouth pipe with a check valve must be used at the end of the delivery pipes in order to reduce velocity at the mouth and to prevent the delivery pipes in order to reduce velocity at the mouth and to prevent water from flowing back, because the loss of head at the mouth is equal to velocity head itself. A velocity at the mouth of about 1.5m/s is desirable.

$$D = 1{,}000\frac{4}{\pi} \times \frac{Q}{V} = 1{,}000\frac{4 \times 1.54}{3.14 \times 1.5}$$

D: Diameter of bell-mouth pipe
Q: Discharge (mm)
V: Velocity which is 1.5m/s

5) Motive power

(i) Axial horsepower is calculated by the following formula.

$$(S.H.P) = \frac{QHWo}{4{,}500Yp} = \frac{0.222QH}{Yp}$$
$$= \frac{0.222 \times 1.54 \times 60 \times 2.22}{0.75} = 63.4\ H.P.$$

Wo: Density of water which is 1,000kg/m^3
Q: Discharge which is 1.54 x 60m^3/min.
H: Total head required which is 7.62ft (=2.32m)
Yp: Efficiency of pump which is 0.65 - 0.80

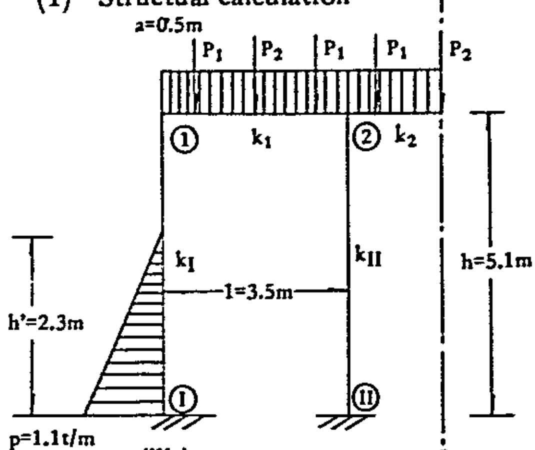
(ii) Necessary horsepower for motor. In case of using motors with reduction gears. (R.H.P.)

$$R.H.P = S.H.P \times \frac{1}{Yg} \times em$$
$$= 63.4 \times \frac{1}{0.92} \times 1.15 = 79.2\ H.P = 59.4kW\ (= 60kW)$$

Yg: Efficiency of reduction gear which is 0.92 - 0.95
em: Coefficient for clearance of electricity which is 1.10 - 1.20

6) Intake

(1) Structual calculation



(We)
Density of soil = 1.6t/m^3 Density of reinforced concrete (Wc) = 2.4t/m^3

(cf. detailed cross section)

P_1: A half of the force required for winding up the gate which is assumed as 5.0t $P_1 = 5.0 \times ½ = 2.5t$

P_2: Weight of winch and operator
$P_2 = 0.30t + 2 \times 0.075t \doteqdot 0.5t$

$P_3^3 = (4.65 \times 0.5 + 0.15 \times 0.45) \times 24 = 5.76t$

$P_4 = (4.65 \times 0.5 + 2 \times 0.15 \times 0.45) \times 2.4 = 5.93t$

$W_1 = 0.45 \times 1.0 \times 2.4 = 1.08t/m$

$P = C_eW_eh' = 0.3 \times 1.6 \times 2.3 = 1.10t/m$
(Ce = coff. of earth pressure)

$P = ½ph' = ½ \times 1.10 \times 2.30 = 1.27t$

$$\Sigma M_1 = M_{1.I} + M_{1.2} = K_I(2f_1 + m) + C_{1.I} + k_1(2f_1 + f_2) - C_{1.2}$$
$$= 2(kI + k1)\, f_1 + k_1 f_2 + k_I m - C_{1.2} + C_{1.I} = 0$$
$$\therefore r_1 f_1 + k_1 f_2 + k_I m = C_{1.2} - C_{1.I} \qquad \ldots (1)$$

$$\Sigma M_2 = M_{2.1} + M_{2.3} + M_{2.II}$$
$$= k_1(2f_2 + f_1) + C_{2.1} + k_2(2f_2 - f_2) - C_{2.3} + k_{II}(2f_2 + m)$$
$$= 2(k_1 + k_2 + k_{II}) - k_2 f_2 + k_1 f_1 + k_{II} m + C_{2.1} - C_{2.3} = 0)$$
$$\therefore (r_2 - k_2) f_2 + k_1 f_1 + k_{II} m = C_{2.3} - C_{2.1} \qquad \ldots (2)$$
$$1/3Ph' + Mo + Mu = ph'/3 + M_{1.I} + M_{2.II} + M_{I.1} + M_{II.2}$$
$$= \frac{ph'}{3} + k_I(2f_1 + m) + C_{1.I} + k_{II}(2f_2 + m) + k_I(f_1 + m) - C_{I.1} + k_{II}(f_2 + m) = 0$$
$$\therefore 3k_I f_1 + 3k_I f_2 + 2(k_I + k_{II})\, m = \frac{ph'}{3} + C_{I.1} - C_{1.I}$$
$$\therefore k_I f_1 + k_{II} f_2 + 2/3\,(k_I + k_{II})\, m = 1/3\,(-\frac{ph'}{3} + C_{I.1} - C_{1.I})$$
$$\therefore k_I f_1 + k_{II} f_2 + X.m = R \qquad \ldots (3)$$

No. of formula	Left side (of equation)			Right side (of equation)
	f_1	f_2	m	
(1)	r_1	k_1	k_I	$C_{1.2} - C_{1.I}$
(2)	k_1	r_2	k_{II}	$C_{2.3} - C_{2.1}$
(3)	k_I	k_{II}	X	R

$$k_1 = k_2 = \frac{bt^3}{12} \times \frac{1}{h} = \frac{100 \times 45^3}{12} \times \frac{1}{350} = 2{,}170\ (cm^3)$$

$$k_I = k_{II} = \frac{bt^3}{12} \times \frac{1}{h} = \frac{100 \times 50^3}{12} \times \frac{1}{510} = 2{,}042$$

$$X = 2/3(k_I + k_{II}) = 2/3 \times 2 \times 2{,}042 = 2{,}723$$
$$r_1 = 2(k_I + k_1) = 2\,(2{,}042 + 2{,}170) = 8{,}424$$
$$r_2 = 2\,(k_1 + k_2 + k_{II}) = 2\,(2 \times 2{,}170 + 2{,}042) = 12{,}764$$
$$r'_2 = r_2 - k_2 = 12{,}764 - 2{,}170 = 10{,}594$$
$$R = 1/3\,(-1.27 \times \frac{230}{3} + 0.70 - 0.13) = -0.13 t\text{-}m = -13{,}000\ kg\text{-}cm$$

$$C_{I.1} = \frac{\frac{ph'}{3}(h - \frac{h'}{3})^2}{H^2} = \frac{1.27 \times 0.77 \times (4.33)^2}{(5.1)^2} = 0.70\ t\text{-}m = 70{,}000\ kg\text{-}cm$$

$$C_{1.I} = \frac{\frac{P(h-h')}{3}\frac{(h')^2}{3}}{h^2} = \frac{1.27 \times 4.33 \times (0.77)^2}{(5.1)^2} = 0.13\ t\text{-}m = 13{,}000\ kg$$

$$C_{1.2} = C_{2.1} = C_{2.3} = \frac{W_1 l^2}{12} + \frac{P_2 l}{8} + \frac{P_1 a\,(1 - a)}{1}$$
$$= \frac{1.08 \times (3.5)^2}{12} + \frac{0.5 \times 3.5}{8} + \frac{2.5 \times 0.5\,(3.5 - 0.5)}{3.5}$$
$$= 1.10 + 0.22 + 1.07 = 2.39\ t\text{-}m = 239{,}000\ kg\text{-}cm$$
$$C_{1.2} - C_{1.I} = 226{,}000\ kg\text{-}cm$$
$$C_{2.3} - C_{2.1} = 0$$

No. of formula	Left side (of equation)			Right side (of equation)
	f_1	f_2	m	
(1)	8,424	2,170	2,042	226,000
(2)	2,170	10,594	2,042	0
(3)	2,042	2,042	2,723	- 13,000

$$f_1 = \frac{1}{8,424}(226,000 - 2,170\, f_2 - 2,042m)$$

$$f_2 = \frac{1}{10,594}(- 2,170f_1 - 2,042m)$$

$$m = \frac{1}{2,723} \quad - 13,000 - 2,042\,(f_1 + f_2)^3$$

This calculation was carried out by numerical reiteration method and the results are shown on the following table.

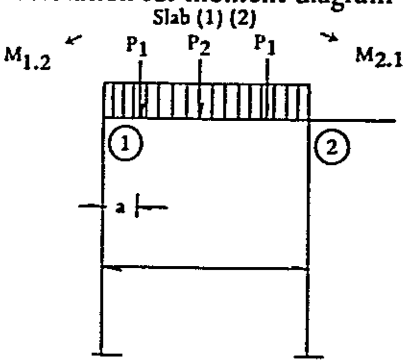
No. of trial	f_1	f_2	m
(1)	27	0	-5
(2)	28	-5	-22
(3)	33	-3	-27
(4)	35	-2	-30
(5)	35	-1	-30
(6)	34	-1	-30

$\therefore \quad f_1 = 34$
$f_2 = -1$
$m = -30$

$M_{1.I} = k_I\,(2f_1 + m) + C_{1.I} = 2,042\,(2 \times 34 - 30) + 13,000 = 77.596 + 13,000 = 90,596$ (kg-cm)
$M_{1.2} = k_1(2f_1 + f_2) - C_{1.2} = 2,170\,(2 \times 34 - 1) - 239,000 = 145,390 - 239,000 = - 93,610$
$M_{2.1} = k_1\,(2f_2 + f_1) + C_{2.1} = 2,170\,[\,2 \times (-1) + 34] + 239,000 = 69,440 + 239,000 = 308,440$
$M_{2.3} = k_2\,(2f_2 - f_2) - C_{2.3} = 2,170\,[2 \times (-1) - (-1)] - 239,000 = -241,170$
$M_{2.II} = k_{II}\,(2f_2 + m) = 2,042\,[2 \times (-1) - 30\,] = -65,344$
$M_{I.1} = k_I(f_1 + m) - C_{I.1} = 2,042\,(34 - 30) - 70,000 = 8,168 - 70,000 = -61,832$
$M_{II.2} = ke_{ii}\,(f_2 + m) = 2,042\,(-1 - 30) = - 63,302$

These results prove to be correct by the following calculation
$M_{1.I} + M_{1.2} = 90,596 - 93,610 = 3,014$ (kg-cm) $= -0.03$ (t-m) $\doteqdot 0$
$M_{2.1} + M_{2.3} + M_{2.II} = 308,440 - 241,170 - 65,344 = 1,926$ (kg-cm) $= 0.02$ (t-m) $\doteqdot 0$
$\frac{Ph'}{3} + M_{1.I} + M_{2.II} + M_{I.1} + M_{II.2} = 97,000 + 90,596 - 65,344 - 61,832 - 63,302 = -2,882$ (kg-cm) $= -0.03$ (t-m) $\doteqdot 0$

Calculation for moment diagram



$$R_1 = S_1 = P_1 + \frac{P_2}{2} + \frac{W_1 1}{2} - \frac{M_{1.2} + M_{2.1}}{1}$$

(R = reaction force S = shear force)

$$= 2.5 + \frac{0.5}{2} + \frac{1.08 \times 3.5}{2} - \frac{(-0.94 + 3.08)}{3.5} = 4.03t$$

$- R_2 = S_2 = R_1 = 2P_1 - P_2 - W_1$
$= 4.03 - 5.0 - 0.5 - 1.08 \times 3.5 = - 5.25$ t

$0 \leq x \leq 0.5$ $\quad M = R_1x - \frac{w_1x^2}{2} + M_{1.2} = 4.03x - 0.54x^2 - 0.94$

$0.5 \leq x \leq 1.75$ $\quad M = R_1x - P_1(x-0.5) - \frac{W_1x^2}{2} + M_{1.2} = 4.03x - 2.5x + 1.25 - 0.54x^2 - 0.94$

$= 1.53x - 0.54x^2 + 0.31$

$1.75 \leq x \leq 3.0$ $\quad M = R_1x - P_1(x - 0.5) - P_2(x - 1.75) - \frac{W_1x^2}{2} + M_{1.2}$

$= 4.03x - 2.5x + 1.25 - 0.5x + 0.88 - 0.54x^2 - 0.94 = 1.03x - 0.54x^2 + 1.19$

$3.0 \leq x \leq 3.5$ $\quad M = R_1x - P_1(x - 0.5) - P_2(x - 1.75) - P_1(x - 3.0) - \frac{W_1}{2}x^2 + M_{1.2}$

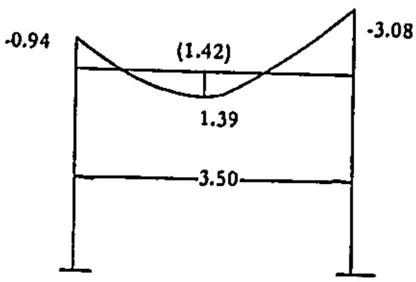
$= 4.03x - 2.5x + 1.25 - 0.5x + 0.88 - 2.5x + 7.50 - 0.54x^2 - 0.94$

$= -1.47x - 0.54x^2 + 8.69$

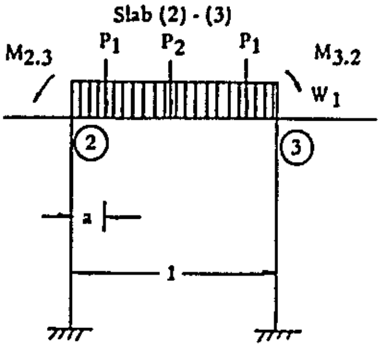
$0.5 \leq x \leq 1.75$

$\frac{dM}{dx} = 1.53 - 1.08x = 0 \quad x = \frac{1.53}{1.08} = 1.42m$

$\therefore$ M max $= 1.53 \times 1.42 - 0.54 \times (1.42)^2 + 0.31 = 1.39$ t-m



Slab (2) - (3)



$R_2 = S_2 = \frac{P_2}{2} + \frac{W_1}{2} - \frac{M_{2.3} + M_{3.2}}{1}$

$= 2.5 + \frac{0.5}{2} + \frac{1.08 \times 3.5}{2} - 0 = 4.64t$

$-R_3 = S_3 = R_2 - 2P_1 - P_2 - W_1$

$= 4.64 - 5.0 - 0.5 - 1.08 \times 3.5 = -4.64$ t

$0 \leq x \leq 0.5$ $\quad M = R_2x - \frac{W_1x^2}{2} + M_{2.3} = 4.64x - 0.54x^2 - 2.41$

$0.5 \leq x \leq 1.75$ $\quad M = R_2x - P_1(x - 0.5) - \frac{W_1x^2}{2} + M_{2.3} = 4.64x - 2.5x + 1.25 - 0.54x^2 - 2.41$

$= 2.14x - 0.54x^2 - 1.16$

$1.75 \leq x \leq 3.0$ $\quad M = R_2x - P_1(x - 0.5) - P_2(x - 1.75) - \frac{W_1x^2}{2} - M_{2.3}$

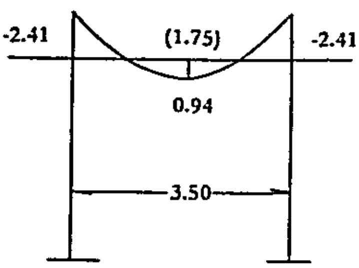
$= 4.64x - 2.5x + 1.25 - 0.5x + 0.88 - 0.54x^2 - 2.41 = 1.64x - 0.54x^2 - 0.28$

$3.0 \leq x \leq 3.5$ $\quad M = R_2x - P_1(x - 0.5) - P_1(x - 1.75) = P_1(x - 3.0) - \frac{W_1x^2}{2} - M_{2.3}$

$= 4.64x - 2.5x + 1.25 - 0.5x + 0.88 - 2.5x + 7.50 - 0.54x^2 - 2.41$

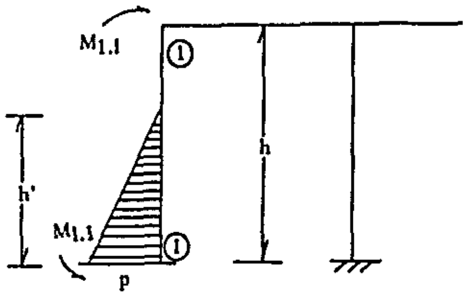
$= -0.86x - 0.54x^2 + 7.22$

M max $\quad = 2.14 \times 1.75 - 0.54x\,(1.75)^2 - 1.16 = 0.94$ t-m



End Pier (1) - (I) $\quad R_I = S_I = \frac{P(h-\frac{h'}{3})}{h} - \frac{M_{1.I}+M_{I.1}}{h} = 1.27\frac{(5.1-0.77)}{5.1} - \frac{(0.91-0.62)}{5.1} = 1.02t$

$-R_1 = S_1 = R_I - P = 1.02 - 1.27 = -0.25t$



$P = ½ph' = 1.27t$

$0 \leq x \leq 2.3 \qquad M = R_I x - \frac{Px^2}{2} + \frac{Px^3}{6h'} + M_{I.1} = 1.02x - \frac{1.10}{2}x^2 + \frac{1.10x^3}{6 \times 2.3} - 0.62$

$= 1.02x - 0.55x^2 + 0.08x^3 - 0.62$

$2.3 \leq x \leq 5.1 \qquad M = R_I x - P(x - \frac{h'}{3}) + M_{I.1} = 1.02x - 1.27(x - 0.77) - 0.62 = -0.25x + 0.36$

$0 \leq x \leq 2.3$

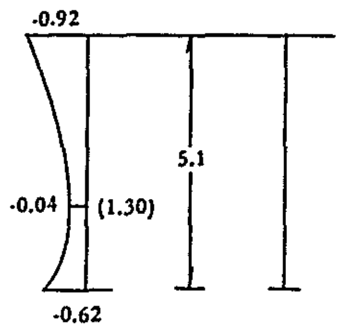
$\frac{dM}{dx} = 1.02 - 1.10x + 0.24x^2 = 0$

$x^2 - 4.58x + 4.25 = 0$

$x = 2.29 \pm \sqrt{(2.29)^2 - 4.25} = 2.29 \pm 0.99 \quad \therefore x = 1.30$

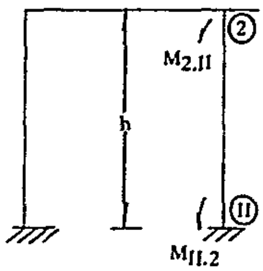
$M1.30 = 1.02 \times 1.03 - 0.55(1.30)^2 + 0.08(1.30)^3 - 0.62$

$= 1.33 - 0.93 + 0.18 - 0.62 = -0.04$ t-m



Intermediate Pier

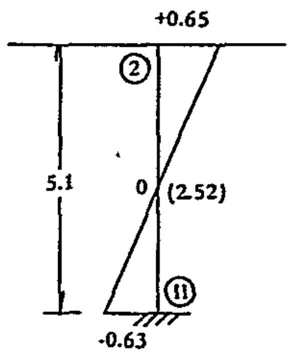
(2) - (II)



$R_{II} = S_{II} = \frac{M_{2.II}+M_{II.2}}{h}$

$= -\frac{(-0.65-0.63)}{5.1} = 0.25t$

$-R_2 = S_2 = R_{II} = 0.25t$



$M = R_{II}x + M_{II.2} = 0.25x - 0.63$

$M = 0.25x - 0.63 = 0$

$\therefore x = \frac{0.63}{0.25} = 2.52m$

Summary of moments (M)

Slab		Piers	
joint (1)	-9.94t-m	joint (1)	- 0.92 t-m
midway	+ 1.39	midway	- 0.04
joint (2)	- 3.08	joint (I)	- 0.62
joint (2)	- 2.41	joint (2)	+ 0.65
midway	+ 0.94	joint (II)	- 0.63
joint (3)	- 2.41		

Calculation for reinforcement

The calculations done by moments and also axial forces are shown below.

Psa = allowable tensile stress of reinforcement bars which is chosen to be 1,200kg/cm^2
(= 17,000 lbs/in^2)

Pca = allowable compressive stress of concrete which is chosen to be 50kg/cm^2
(= 710 lbs/in^2)

$d\ min \geq C_1 \frac{M}{b}$ d min = minimum effective depth required in section.

b = unit width = 100cm C_1 = Coefficient which is 0.345

No.	Position	C_1	M (kg-cm)	M/b	$\sqrt{M/b}$	d min	designed effective depth with haunching d
(1)	Slab joint (2)	0.345	-308,000	3,080	55.5	19.1 cm <	45 + 15 - 6 = 54(cm)
(2)	Slab midway (1) - (2)	0.345	+139,000	1,390	37.3	12.9 <	45 - 6 = 39
(3)	Pier joint (1)	0.345	- 92,000	920	30.3	10.5 <	50 - 6 = 44
(4)	Pier joint (2)	0.345	+65,000	650	25.5	8.8 <	50 - 6 = 44
(5)	Pier (I)	0.345	- 62,000	620	24.0	8.6 <	50 - 6 = 44
(6)	Pier (II)	0.345	- 63,000	630	25.1	8.7 <	50 - 6 = 44

$$As = \frac{C_1 C_2 M_s}{d} - \frac{N}{Psa}$$

$$Ms = M + NC$$

$$C = \frac{t}{2} - d'$$

$$C_1 \times C_2 = 0.00096$$

As = area of reinforcement bars required.
C_2 = Coefficient which is 0.00277
N = Axial force
d = Designed effective depth with haunching
t = Overall depth in section.
d' = Concrete cover = 6cm

No.	Position	M kg-cm	Nkg	tcm	d'cm	Ccm	NCkg-cm	Ms kg-cm	d cm	0.00096 d 2	N/1200	As
(1)	Slab joint(2)	-308,000	250	45	6	16.5	4,125	312,125	54	5.55cm	0.21	5.34
(2)	Slab midway (1) - (2)	+139,100	0	45	6	16.5	0	139,000	39	3.42	0	3.42
(3)	Pier joint (1)	- 92,000	4,030	50	6	19 0	76,760	168,760	44	3.68	3.35	0.33
(4)	Pier joint (2)	+ 65,000	9,890	50	6	19.0	187,910	252,910	44	5.52	8.24	-
(5)	Pier joint (I)	- 62,000	9,790	50	6	19.0	186,010	248,010	44	5.41	8.16	-
(6)	Pier joint (II)	- 63,000	15,820	50	6	19.0	300,580	363,580	44	7.93	13.18	-

Diameter and spacing of reinforcement bar

(1) d = 16mm (= 5/8") ctc 25cm (=10")
(2) d = 13 (= ½") ctc 25 (=10")
(3) d = 13 (=½") ctc 25 (=10")
(4) d = 13 (=½") ctc 25 (=10')
(5) d = 13 (=½") ctc 25 (=10")
(6) d = 13 (=½") ctc 25 (=10")

Calculation for compressive stress (Pc)

At joint II $N = 15{,}820kg$ $Ms = 363{,}580$ kg-cm $b = 100cm$

$e = Ms/N = 363{,}580/15{,}820 = 23.0cm$ $L = 50cm$

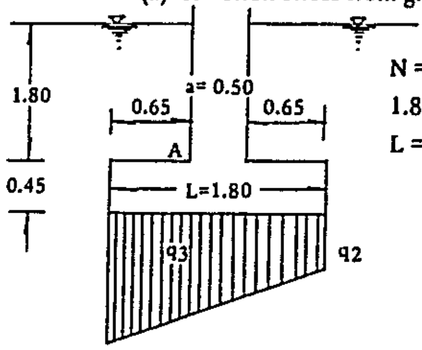
$$Pc = \frac{N}{bL}(1 + \frac{6e}{L}) = \frac{15{,}820}{100x50}(1 \pm \frac{6 \times 23.0}{50}) = 3.164\ (1 \pm 2.76)$$

$\therefore$ $Pc = 11.9\ kg/cm^2$ or $-5.6\ kg/cm^2 < Pca = 50kg/cm^2$

Reinforcement bars are to be used for tensile force.

Calculation for footing (analysis in cross section)

(a) Reaction stress from ground (q)



$N = 15{,}820kg$ Pw = weight of water on footing which is $2 \times 0.65 \times 1.80 \times 1.0 = 2.34t = 2{,}340kg$

$L = 180cm$ $a = 50cm$

Pf = weight of footing itself which is $0.45 \times 1.80 \times 1.00 \times 2.4 = 1.944t = 1{,}944kg$

Assuming that there is no uplift in this case,

$$e = \frac{Ms}{N + Pw + Pf} = \frac{363{,}580}{20{,}104} = 18.1cm$$

$$q = \frac{N + Pw + Pf}{bL}(1 \pm \frac{6e}{L}) = \frac{20{,}104}{100 \times 180}(1 \pm \frac{6 \times 18.1}{180}) = 1.117\ (1 \pm 0.61)$$

$q1 = 1.80kg/cm^2$ and $q2 = 0.44\ kg/cm^2$

$(= 18.0t/m^2)$ $(= 4.4t/m^2)$

(b) Punching shear at A: (S_p)

$$S_p = \frac{b}{4}(q1 + q3)(L - a)$$

$$q3 = \frac{q1(L + a) + q2(L - a)}{2L}$$

Overall depth required for punching shear: $t = \frac{S_p}{b\ T_{pa}} + d'$

$$q3 = \frac{1.80(180 + 50) + 0.44(180 - 50)}{2 \times 180} = 1.31\ kg/cm^2$$

$(= 13.1t/m^2)$

$$S_p = \frac{100}{4}(1.80 + 1.31)(180 - 50) = 10{,}108\ kg$$

$$\therefore\ t = \frac{10{,}108}{100 \times 8.0} + 7.5 = 12.6 + 7.5 = 20.1\ cm < 45cm$$

T_{pa} = allowable punching shear - stress which is $8\text{-}10kg/cm^2$

d' = concrete cover = 7.5cm

(c) Moment at A (M)

Moment due to reaction force from ground (M_1)

$$M_1 = \frac{q_3 l^2}{2} + \frac{(q_1 - q_3) l^2}{6} = \frac{13.1 \times 0.65^2}{2} + \frac{(18.0 - 13.1) \times 0.65^2}{6} = 3.12 \text{ t-m}$$

Resistant moment due to, water on footing and footing itself.

$$M_2 = \frac{(P_w + P_f') \times 1}{2} = \frac{2.4 + (0.45 \times 0.65 \times 2.4) \times 0.65}{2} = 0.99 \text{t-m}$$

$\therefore$ $M = M_1 - M_2 = 3.12 - 0.99 = 2.13$ t-m = 213,000 kg-cm

(d) Reinforcement bars

$$A_s = \frac{M}{P_{sa} j d} = \frac{213,000}{1,200 \times 7/8 \times 37.5} = 5.41 \text{cm}^2$$

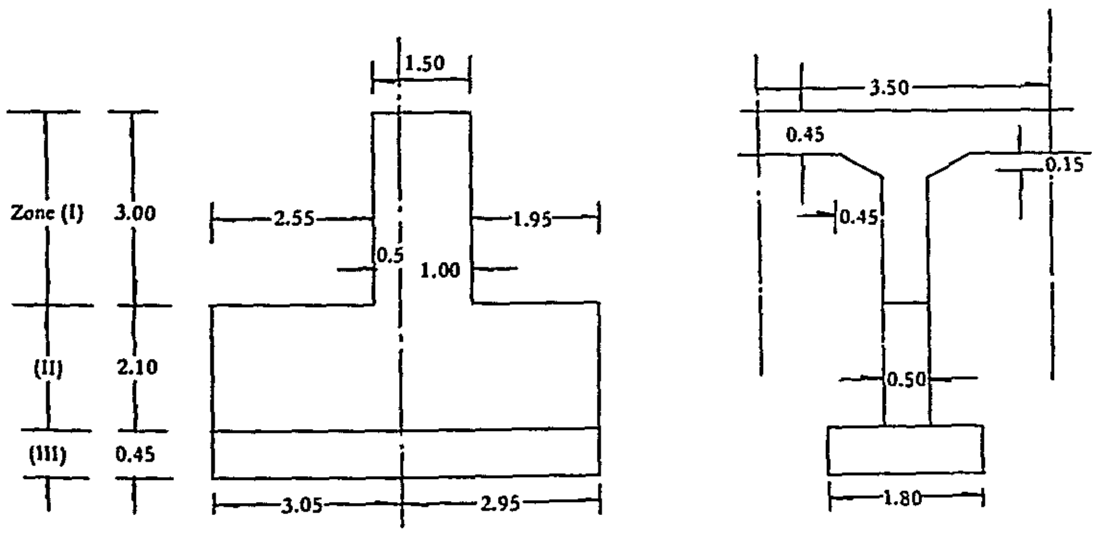
$\therefore$ d = 16mm (5/8") ctc 25cm (10")

(2) Stability analysis (in Profile)

Conditions Gates are shut

Water level at intake = 102.00'
Sill level of intake = 96.00'
Bulk density of concrete w_c = 2.4 t/m^3
Coefficient of uplift u = 0.4

(Unit: m) Unit weight of sedimental sand and silt in water: W_s = 1.0 t/m^3



No.	W	X	W.X	Y	W.Y
I - 1	3.5 x 0.45 x 1.5 x 2.4 = 5.67	0.25	1.42	5.33	30.22
I - 2	½ (1.4 + 0.5) x 0.15 x 1.5 x 2.4				
	= 0.51	0.25	0.13	5.04	2.57
I - 3	2.40 x 0.5 x 1.5 x 2.4 = 4.32	0.25	1.08	3.75	16.20
Total	10.50		2.63		48.99
Mean		0.25		4.67	
II	0.5 x 2.1 x 6.0 x 2.4 = 15.12	-0.05	-0.76	1.50	22.68
III	0.45 x 1.8 x 6.0 x 2.4 = 11.66	-0.05	-0.58	0.23	2.68
TOTAL I	= 10.50	0.25	2.63	4.67	48.99
TOTAL I + II	10.50 + 15.12 = 25.62	1.87/25.62=0.07	2.63-0.76=1.87	71.67/25.62=2.80	48.99+22.68=71.67
TOTAL I+II+III	25.62+11.66=37.28	1.29/37.28=0.03	1.87-0.58=1.29	74.35/37.28=1.99	71.67+2.68=74.35

(a) Sedimental earth pressure acting against the upperstream side of the pier.

Zone (II) $Pe = \frac{1}{2} WsKah^2 B = \frac{1}{2} \times 1.0 \times 0.33 \times 1.80^2 \times 0.5 = 0.27t$

$Ype = 0.60m$

(Assuming that the internal friction angle of earth is 30^o, the earth pressure coefficient will be.

$$Ka = \frac{1 - \sin f}{1 + \sin f} = \frac{1 - \frac{1}{2}}{1 + \frac{1}{2}} = 0.33)$$

(b) Hydrostatic pressure acting against the gate and pier.
Interval of each center of the pier B = 3.50m (=11.7')

Zone (II) $Ps = \frac{1}{2} Woh^2 B = \frac{1}{2} \times 1.0 \times 1.8^2 \times 3.50 = 5.67t$

$Y_{ps} = 0.60m$

(c) Uplift acting upward against the base

Zone (II) $U_2 = \frac{1}{2} uwohA = \frac{1}{2} \times 0.4 \times 1.0 \times 1.8 \times (0.5 \times 6.0) = 1.08t$

Zone (III) $U_3 = \frac{1}{2} \times 0.4 \times 1.0 \times 2.25 \times (1.8 \times 6.0) = 4.86t$

$Xu_2 = Xu_3 = \frac{6.0}{3} - 3.05 = -1.05m$

(d) Design loads Gd
gate 1.8m x 3.20m = 5.76m 5.76m x $0.04t/m^2$ = 0.23t

Spindle = 0.1t
Winch = 0.3t
Operators = 0.15t
Total Gg ≑ 0.9t

- Stability against overturning
  When all the forces calculated above are compounded on figure, it is founded that the resultant of these forces are within the middle third of the base of each of each section.

- Stability against sliding
  Total vertical forces $\Sigma V = G_3 + G_g - U_3 = 37.28 + 0.90 - 4.86 = 33.32t$
  Total horizontal forces $\Sigma H = Pe + Ps = 0.27 + 5.67 = 5.94t$

$$m \geqq \frac{2\Sigma H}{\Sigma V}$$

m = Coefficient of friction between concrete and earth which is 0.5

$$\frac{2 \times 5.94}{33.32} = 0.36 < m = 0.5$$

- Bearing capacity of earth
  $\Sigma V = W = 33.32t$

Eccentric distance $e = \frac{\Sigma M}{W}$

$\Sigma M = G_3X_3 + PeY_{pe} + PsY_{ps} + UX_u$

$= 37.28 \times 0.03 + 0.27 \times 0.60 + 5.67 \times 0.60 + (-4.86) \times (-1.05)$

$= 1.12 + 0.16 + 3.40 + 5.10 = 9.78$ t-m

$\therefore\ e = 9.78/33.32 = 0.29$m

Maximum and minimum compressive stresses are,

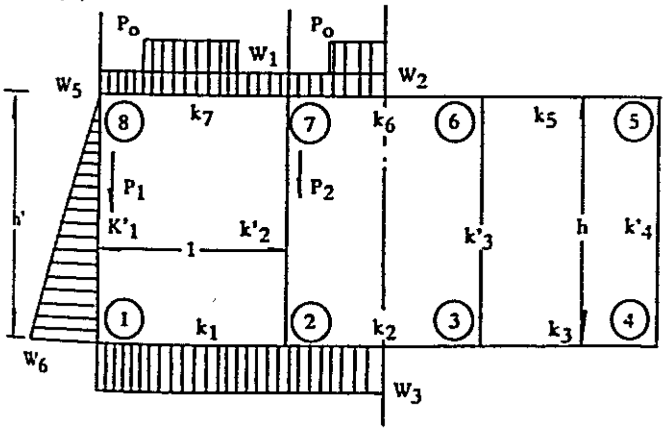
max. min. $Pc = \frac{W}{bL}(1 \pm \frac{6e}{L}) = \frac{33.32}{1.8 \times 6.0}(1 \pm \frac{6 \times 0.29}{6.0}) = 3.085\ (1 \pm 0.29)$

$= 3.98t/m^2$ and $2.19t/m^2$

Although the bearing capacity of the earth seems to be enough for this maximum compressive stress, in profile, it is surely insufficient for the maximum compressive stress in cross section. It will therefore be necessary for a design for Piling.

7) Sump

(1) Structural calculation



The corss section at the center of suction pipe is selected as the one for this calculation. Each column of the pump house must support a load of about 20 tons at the base. Although there is no column in this section concerned it is assumed that a concentrated load of 5 ton (Po) acts at each corresponding joint on the upper slab.

Weight of side wall $P_1 = [0.5 \times 3.50 + \frac{(0.3)^2}{2} \times 2\ ] \times 2.4 = 4.42t$

$P_2 = [0.5 \times 3.50 + \frac{(0.3)^2}{2} \times 4\ ] \times 2.4 = 4.63t$

Weight of pump, suction pipe and the base.

Axial flow pump (d = 800mm) 5.0t

Suction pipe $\frac{0.87t/m \times 3.5m}{\text{Total}} = \frac{3.05}{8.05t}$

This total weight must be supported by the base 1.5m (5') by 2.1m (7')

$$W_1 = \frac{8.05}{1.5 \times 2.1} + \frac{0.2 \times 1.0 \times 2.4}{\text{weight of base}} = 2.56 + 0.48 = 3.04\ t/m$$

Weight of the upper slab (W_2)

$W_2 = 0.45 \times 1.0 \times 2.4 = 1.08t/m$

reaction force from the ground against the base slab (W_3)

$$W_3 = \frac{(1.08 \times 3 \times 3.5) + 2x(4.63+4.42) + (5.0 \times 4) + (3 \times 3.04 \times 1.5)}{(3 \times 3.5)} = \frac{11.34 + 18.10 + 20.0 + 13.68}{10.5}$$

$$= \frac{63.12}{10.5} = 6.01t/m$$

Earth pressure at the base, $= W_4 = CeWeh' = 0.3. \times 1.6 \times 4.15 = 1.99t/m$

Earth pressure at the top $= W_5 = 0.3 \times 1.6 \times 0.15 = 0.07 \doteqdot 0t/m$

Therefore the earth pressure acting against side wall can be regarded as triangular load.

$\Sigma M_1 = M_{1.2} + M_{1.8} = k_1(2f_1 + f_2 + m) + C_{1.2} + k'_1(2f_1 + f_8) - C_{1.8}$

$= 2(k_1 + k'_1)f_1 + k_1f_2 + k'_1f_8 + k_1m + C_{1.2} - C_{1.8} = 0$

$\therefore\ r_1f_1 + k_1f_2 + k'_1f_8 + k_1m = C_{1.8} - C_{1.2}$ ... (1)

$\Sigma M^2 = M_{2.1} + M_{2.7} + M_{2.3} = k_1(2f_2 + f_1 + m) - C_{2.1} + k'_2(2f_2 + f_7) + k_2(2f_2 - f_2) + C_{2.3}$

$= [2(k_1 + k'_2 + k_2) - k_2]\ f_2 + k_1f_1 + K'_2f_7 + k_1m - C_{2.1} + C_{2.3} = 0$

$\therefore\ (r_2 - k_2)f_2 + k_1f_1 + k'_2f_7 + k_1m = C_{2.1} - C_{2.3}$ ... (2)

$\Sigma M_7 = M_{7.8} + m_{7.2} + M_{7.6} = k_7(2f_7 + f_8 + m) + C_{7.8} + k'_2(2f_7 + f_2) + k_6(2f_7 - f_7) - C_{7.6}$

$= [2(k_7 + k'_2 + k_6) - k_6]f_7 + k_7f_8 + k'_2f_2 + k_7m + C_{7.8} - C_{7.6}$

$\therefore\ (r_7 - k_6)\ f_7 + k_7f_8 + k'_2 + f_2 + k_7m = C_{7.6} - C_{7.8}$ ... (3)

$\Sigma M_8 = M_{8.1} + M_{8.7} = k'_1(2f_8 + f_1) + C_{8.1} + k_7\ (2f_8 + f_7 + m) - C_{8.7}$

$= 2(k'_1 + k_7)f_8 + k_7f_7 + k'_1f_1 + k_7m + C_{8.1} - C_{8.7} = 0$

$\therefore\ r_8f_8 + k_7f_7 + k_1f_1 + k_7m = C_{8.1} - C_{8.7}$ ... (4)

$\Sigma Mo + \Sigma Mu = M_{8.7} + M_{7.8} + M_{1.2} + M_{2.1} = (Po + P_1)\ 1 + \frac{W_1s1}{2} + \frac{W_21^2}{2} - \frac{W_31^2}{2}$

$k_7(2f_8 + f_7 + m) - C_{8.7} + k_7(2f_7 + f_8 + m) + C_{7.8} + k_1(2f_1 + f_2 + m) + C_2 + k_1(2f_2 + f_1 + m) - C_{2.1}$

$= (Po + P_1)^1 + \frac{W_1s1}{2} + \frac{W_21^2}{2} - \frac{W_31^2}{2}$

$$3k_7f_8 + 3k_7f_7 + 3k_1f_1 + 3k_1f_2 + 2(k_7 + k_1)m = (Po + P_1)1 + \frac{W_1s1}{2} + \frac{W_2 1^2}{2} - \frac{W_3 1^2}{2}$$

$$\therefore\ k_7f_8 + k_7f_7 + k_1f_1 + k_1f_2 + 2/3\,(k_7 + k_1)m = 1/3\,[(Po + P_1)\,1 + \frac{W_1s1}{2} + \frac{W_2 1^2}{2} - \frac{W_3 1^2}{2}\,]$$

$$\therefore\ k_7f_8 + k_7f_7 + k_1f_1 + k_1f_2 + X.m = R \qquad \text{... (5)}$$

No. of formula	Left side of equation				m	Right side of equation
	f_1	f_2	f_7	f_8		
(1)	r_1	k_1	-	k'_1	k_1	$C_{1.8} - C_{1.2}$
(2)	k_1	r'_2	k'_2	-	k_1	$C_{2.1} - C_{2.3}$
(3)	-	k'_2	r'_7	k_7	k_7	$C_{7.6} - C_{7.8}$
(4)	k'_1	-	k_7	r_8	k_7	$C_{8.7} - C_{8.1}$
(5)	k_1	k_1	k_7	k_7	X	R

$$k_1 = k_2 = \frac{bt^3}{12} \times \frac{1}{1} = \frac{100 \times 50^3}{12} \times \frac{1}{350} = 2{,}976$$

$$k_6 = k_7 = \frac{100 \times 45^3}{12} \times \frac{1}{350} = 2{,}170$$

$$k'_1 = k'_2 = \frac{bt^3}{12} \times \frac{1}{h} = \frac{100 \times 50^3}{12} \times \frac{1}{400} = 2{,}604$$

$X = 2/3\,(k_7 + k_1) = 2/3\,(2{,}170 + 2{,}976) = 3{,}431$

$r_1 = 2(k_1 + k'_1) = 2\,(2{,}976 + 2{,}604) = 11{,}160$

$r_2 = 2(k_1 + k'_2 + k_2) = 2(2{,}976 + 2{,}604 + 2{,}976) = 17{,}112$

$r_7 = 2(k_7 + k'_2 + k_6) = 2(2{,}170 + 2{,}604 + 2{,}170) = 13{,}888$

$r_8 = 2(k'_1 + k_7) = 2\,(2{,}604 + 2{,}170) = 9{,}548$

$r'_2 = r_2 - k_2 = 17{,}112 - 2{,}976 = 14{,}136$

$r'_7 = r_7 - k_6 = 13{,}888 - 2{,}170 = 11{,}718$

$$C_{1.8} = \frac{W_4h^2}{20} = \frac{1.99 \times (4.0)^2}{30} = 1.59\text{t-m} = 159{,}000 \text{ kg-cm}$$

$$C_{8.1} = \frac{W_4h^2}{30} = \frac{1.99 \times (4.0)^2}{30} = 1.06 \text{ t-m} = 106{,}000 \text{ kg-cm}$$

$$C_{1.2} = C_{2.1} = C_{2.3} = \frac{W_3 1^2}{12} = \frac{6.01 \times (3.5)^2}{12} = 6.14 \text{ t-m} = 614{,}000 \text{ kg-cm}$$

$$C_{8.7} = C_{7.8} = C_{7.6} = \frac{W_2 1^2}{12} + \frac{W_1S}{241}\,(3\,1^2 - S^2) = \frac{1.08 \times (3.5)^2}{12} + \frac{3.04 \times 1.5}{24 \times 3.5} \cdot [3 \times (3.5)^2 - (1.5)^2]$$

$= 2.97$ t-m $= 297{,}000$ kg-cm

$$R = 1/3\,[(Po + P_1)1 + \frac{W_1S1}{2} + \frac{W_2 1^2}{2} - \frac{W_3 1^2}{2}\,]$$

$$= 1/3\,[(5.00 + 4.42) \times 3.5 + \frac{3.04 \times 1.5 \times 3.5}{2} + \frac{1.08 \times (3.5)^2}{2} - \frac{6.01\,(3.5)^2}{2}\,]$$

$= 1/3\,(32.97 + 7.98 + 6.62 - 36.81) = 3.59$ t-m $= 359{,}000$ kg-cm

$C_{1.8} - C_{1.2} = 159{,}000 - 614{,}000 = -455{,}000$ kg-cm

$C_{2.1} - C_{2.3} = 0$ kg-cm

$C_{7.6} - C_{7.8} = 0$ kg-cm

$C_{8.7} - C_{8.1} = 297{,}000 - 106{,}000 = 191{,}000$ kg-cm

No. of formula	Left side of equation					Right side of equation
	f_1	f_2	f_7	f_8	m	
(1)	11,160	2,976	-	2,604	2,976	- 455,000
(2)	2,976	14,136	2,604	-	2,976	0
(3)	-	2,604	11,718	2,170	0	
(4)	2,604	-	2,170	9,548	2,170	191,000
(5)	2,976	2,976	2,170	2,170	3,431	359,000

$$f_1 = \frac{1}{11,160} [- 455,000 - 2,976 (f_2 + m) - 2,604 f_8]$$

$$f_2 = \frac{1}{14,136} [- 2,604 f_7 - 2,976 (f_1 + m)]$$

$$f_7 = \frac{1}{11,718} [- 2,604 f_2 - 2,170 (f_8 + m)]$$

$$f_8 = \frac{1}{11,718} [- 191,000 - 2,604 f_1 - 2,170 (f_7 + m)]$$

$$m = \frac{1}{3,431} [359,000 - 2,976 (f_1 + f_2) - 2,170 (f_7 + f_8)]$$

This calculation was carried out by numerical reiteration method and the result are shown on the following table.

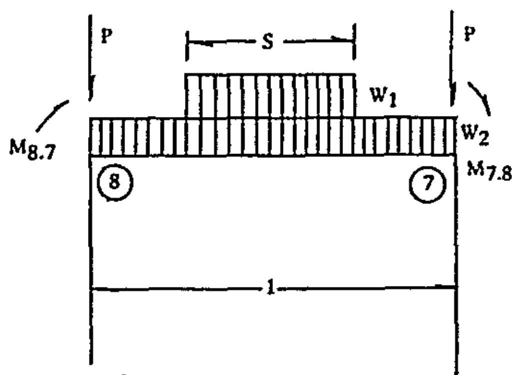
No. of trial	f_1	f_2	f_7	f_8	m
(1)	- 41	0	0	+ 20	+ 105
(2)	- 73	- 7	- 22	+ 23	+ 169
(3)	- 89	- 13	- 33	+ 13	+ 206
(4)	- 95	- 17	- 37	+ 8	+ 220
(5)	- 97	- 19	- 38	+ 5	+ 226
(6)	- 97	- 20	- 38	+ 4	+ 228
(7)	- 97	- 21	- 38	+ 3	+ 229
(8)	97	- 21	- 38	+ 3	+ 229

$M_{1.2} = k_1(2f_1 + f_2 + m) + C_{1.2}$
$= 2,976\ [2 \times (-97) + (-21) + 229] + 614,000 = 2,976\ (-194 - 21 + 229) + 614,000$
$= 2,976 \times 14 + 614,000 = 41,664 + 614,000 = (+)\ 655,664$ kg-cm $= (+)\ 6.56$ t-m
$M_{1.8} = k'_1\ (2f_1 + f_8) - C_{1.8}$
$= 2,604\ [2 \times (-97) + 3] - 159,000 = 2,604\ (-194 + 3) - 159,000$
$= 2,604 \times (-191) - 159,000 = -497,364 - 159,000 = (-)\ 656,364$ kg-cm $= (-)\ 6.56$ t-m
$M_{2.1} = k_1(2f_2 + f_1 + m) - C_{2.1}$
$= 2,976\ [2 \times (-21) - 97 + 229] - 614,000 = 2,976\ (-42 - 97 + 229) - 614,000$
$= 2,976 \times 90 - 614,000 = 267,840 - 614,000 = (-)\ 346,160$kg-cm $= (-)\ 3.46$ t-m
$M_{2.7} = k'_2\ (2f_2 + f_7)$
$= 2,604\ [2 \times (-21) - 38] = 2,604\ (- 42 - 38)$
$= 2,604 \times (-80) = (-)\ 208,320$ kg-cm $= (-)\ 2.08$ t-m
$M_{2.3} = k_2(2f_2 - f_2) + C_{2.3}$
$= 2,976 \times (-21) + 614,000 = -62,496 + 614,000 = (+)\ 551,504$ kg-cm $= (+)\ 5.52$ t-m

$M_{7.8} = k_7(2f_7 + f_8 + m) + C_{7.8}$
$= 2{,}170[2 \times (-38) + 3 + 229] + 297{,}000 = 2{,}170(-76 + 3 + 229) + 297{,}000$
$= 2{,}170 \times 156 + 297{,}000 = 338{,}520 + 297{,}000 = (+)\ 635{,}520$ kg-cm $= (+)\ 6.35$ t-m
$M_{7.2} = k'_2(2f_7 + f_2)$
$= 2{,}604\ [2 \times (-38) - 21] = 2{,}604\ (-76 - 21)$
$= 2{,}604 \times (-97) = (-)\ 252{,}588$ kg-cm $= (-)\ 2.53$ t-m
$M_{7.6} = k_6(2f_7 - f_7) - C_{7.6}$
$= 2{,}170 \times (-38) - 297{,}000 = -82{,}460 - 297{,}000 = (-)\ 379{,}460$kg-cm $(-)\ 3.80$t-m
$M_{8.1} = k'_1(2f_8 + f_1) + C_{8.1}$
$= 2{,}604\ (2 \times 3 - 97) + 106{,}000 = 2{,}604 \times (-91) + 106{,}000$
$= -236{,}964 + 106{,}000 = (-)\ 130{,}964$ kg-cm $= (-)\ 1.31$ t-m
$M_{8.7} = k_7(2f_8 + f_7 + m) - C_{8.7}$
$= 2{,}170\ (2 \times 3 - 38 + 229) - 297{,}000 = 2{,}170 \times 197 - 297{,}000$
$= 427{,}490 - 297{,}000 = (+)\ 130{,}490$ kg-cm $= (+)\ 1.31$ t-m

These results prove to be correct by the following calculation.

$\Sigma M_1 = M_{1.2} + M_{1.8} = +6.56 - 6.56 = 0$
$\Sigma M_2 = M_{2.1} + M_{2.7} + M_{2.3} = -3.46 - 2.08 + 5.52 = -0.02 \fallingdotseq 0$
$\Sigma M_7 = M_{7.8} + M_{7.2} + M_{7.6} = +6.35 - 2.53 - 3.80 = +0.02 \fallingdotseq 0$
$\Sigma M_8 = M_{8.1} + M_{8.7} = -1.31 + 1.31 = 0$
$M_{8.7} + M_{7.8} + M_{1.2} + M_{2.1} - 3R = +1.31 + 6.35 + 6.56 - 3.46 - 3 \times 3.59$
$= +1.31 + 6.35 + 6.56 - 3.46 - 10.77 = -0.01 \fallingdotseq 0$



(R = reaction force, S = shear of force)
Slab (8) - (7)

$R_8 = S_8 = P + \frac{W_1 S}{2} + \frac{W_2 l}{2} - \frac{M_{8.7} + M_{7.8}}{l}$

$= 5.0 + \frac{3.04 \times 1.5}{2} + \frac{1.08 \times 3.5}{2} - \frac{1.31 + 6.35}{3.50}$

$= 5.0 + 2.28 + 1.89 - 2.19 = 6.98$t
$-R_7 = S_7 = R_8 - 2P - W_1 S - W_2 l$
$= 6.98 - 10.0 - 4.56 - 3.78 = -11.36$t

$0 \leq x \leq 1.0$
$M = (R_8 - P)x - \frac{W_2}{2}x^2 + M_{8.7}$
$= 2.52x - 0.54x^2 + 1.31$
$1.0 \leq x \leq 2.5$
$M = (R_8 - P)x \frac{-W_1}{2}(x - 1.0)^2 - \frac{W_2}{2} + M_{8.7}$
$= 2.52x - 1.52(x - 1.0)^2 - 0.54x^2 + 1.31$
$= -2.06x^2 + 5.56x - 0.21$
$2.5 \leq x \leq 3.5$
$M = (R_8 - P)x - W_1 S\ (x - ½) - \frac{W_2}{2}x^2 + M_{8.7}$
$= 2.52x - 4.56\ (x - 1.75) - 0.54x^2 + 1.31$
$= -0.54x^2 - 2.04x + 9.29$

$1.0 \leq x \leq 2.5$

$\frac{dM}{dx} = -4.12x + 5.56 = 0$

$\therefore x = \frac{5.56}{4.12} = 1.35m$

$\therefore M_{1.35} = -2.06 \times (1.35)^2 + 5.56 \times 1.35 - 0.21 = 3.75 + 7.51 - 0.21 = 3.55$ t-m

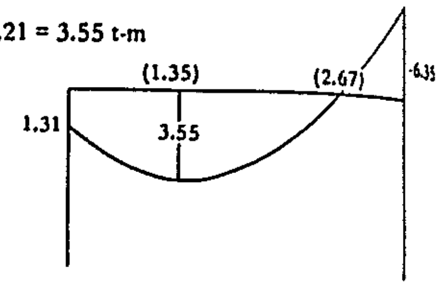
$2.5 \leq x \leq 3.5$

$M = -0.54x^2 - 2.04x + 9.29 = 0$

$x^2 + 3.78x - 17.20 = 0$

$x = -1.89 \pm \sqrt{(1.89)^2 + 17.20} = -1.89 \pm 4.56$

$\therefore x = 2.67m$

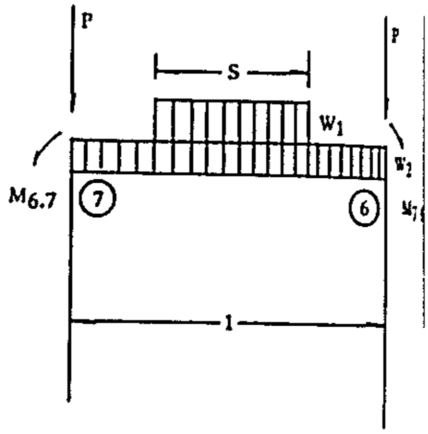


Slab (7) - (6)

$R_7 = S_7 = P + \frac{W_1S}{2} + \frac{W_2l}{2} - \frac{M_{7.6} + M_{6.7}}{l}$

$= 5.0 + 2.28 + 1.89 - \frac{(-3.80 + 3.80)}{3.5} = 9.17t$

$-R_6 = S_6 = R_7 - P - W_1S + W_2l - P = -9.17t$



$0 \leq x \leq 1.0$

$M = (R_7 - P)x - \frac{W_2x^2}{2} + M_{7.6} = 4.17x - 0.54x^2 - 3.80$

$1.0 \leq x \leq 2.5$

$M = (R_7 - P)x - \frac{W_1}{2}(x - 1.0)^2 - \frac{W_2x^2}{2} + M_{7.6}$

$= 4.17x - 1.52\,(x - 1.0)^2 - 0.54x^2 - 3.80$

$= -2.06x^2 + 7.21x - 5.32$

$2.5 \leq x \leq 3.5$

$M = (R_7 - P)x - WS\,(x - ½) - \frac{W_2}{2}x^2 + M_{7.6}$

$= 4.17x - 4.56(x - 1.75) - 0.54x^2 - 3.80$

$= -0.54x^2 - 0.39x + 4.18$

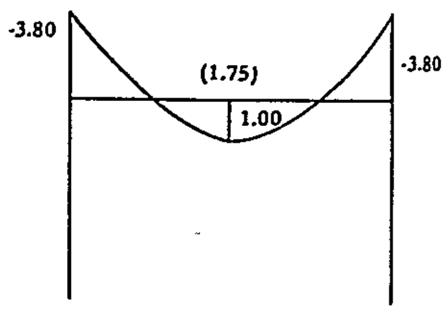
$1.0 \leq x \leq 2.5$

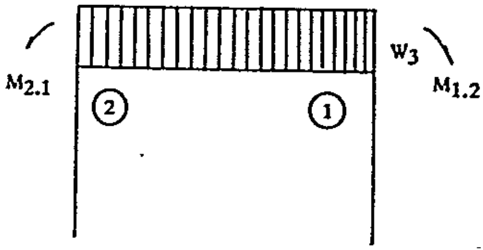
$\frac{dM}{dx} = -4.12x + 7.21 = 0$

$\therefore x = \frac{7.21}{4.12} = 1.75m$

$M_{1.75} = -2.06 \times (1.75)^2 + 7.21 \times 1.35 - 5.32$

$= -6.30 + 12.62 - 5.32 = 1.00t\text{-}m$





Slab (2) — (1)

$R_2 = S_2 = \frac{W_3l}{2} - \frac{M_{2.1} + M_{1.2}}{l}$

$= \frac{6.07 \times 3.5}{2} - \frac{(-3.46 + 6.56)}{3.5} = 10.62 - 0.89 = 9.73t$

$-R_1 = S_1 = R_2 - W_3l = 9.73 - 21.24 = -11.51t$

$M = R_2x - \frac{W_3x^2}{2} + M_{2.1} = 9.73 - 3.035x^2 - 3.46$

$\frac{dM}{dx} = \ 9.73 + 6.07x = 0$

$x = \frac{9.73}{6.07} = \ 1.60m$

$M_{1.60} = 9.73 \times 1.60 - 3.085 \times (1.60)^2 - 3.46$

$= 15.57 - 7.77 - 3.46 = 4.34$ t-m

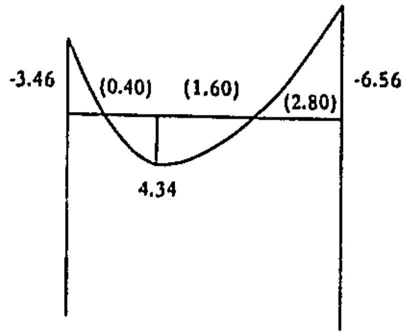
$M = 9.73x - 3.035x^2 - 3.46 = 0$

$\therefore\ x^2 - 3.21x + 1.14 = 0$

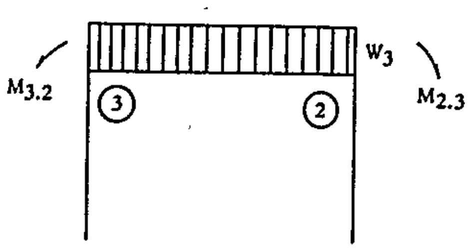
$x = ½\,(3.21 \pm \sqrt{(3.21)^2 - 4 \times 1.14})$

$= ½\,(3.21 \pm 2.40)$

$\therefore\ x = 0.40m$ or $2.80m$



Slab (3) – (2)



$R_3 = S_3 = \frac{W_3 l}{2} - \frac{M_{3.2} + M_{2.3}}{l}$

$= \frac{6.07 \times 3.5}{2} - \frac{(-5.52 + 5.52)}{3.5} = 10.62t$

$- R_2 = S_2 = R_3 - W_3 l = 10.62 - 21.24 = -10.62t$

$M = R_3x - \frac{W_3}{2}x^2 + M_{3.2}$

$= 10.62x - 3.035x^2 - 5.52$

$\frac{dM}{dx} = \ 10.62 - 6.97x = 0$

$\therefore\ x = 1.75m$

$M_{1.75} = 10.62 \times 1.75 - 3.035 \times (1.75)^2 - 5.52$

$= 18.59 - 9.30 - 5.52 = 3.77$ t-m

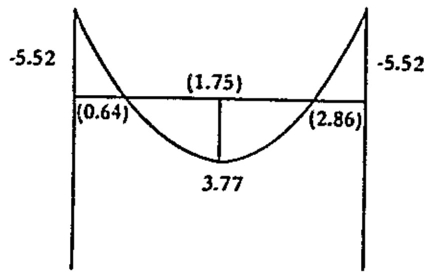
$M = 10.62x - 3.035x^2 - 5.52 = 0$

$x^2 - 3.50x + 1.82 = 0$

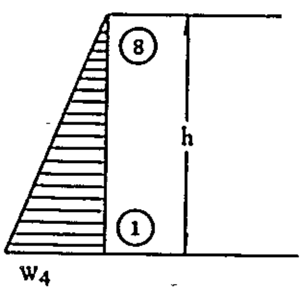
$x = 1.75 \pm \sqrt{(1.75)^2 - 1.82}$

$= 1.75 \pm 1.11$

$\therefore\ x = 0.64m$ or $2.86m$



Side wall (1) - (8)



$R_1 = S_1 = \frac{W_4h}{3} - \frac{M_{1.8} + M_{8.1}}{h} = \frac{1.99 \times 4.0}{3}$

$- \frac{(-6.56 - 1.31)}{4.0} = 2.65 + 1.97 = 4.62t$

$-R_8 = S_8 = R_1 - \frac{W_4h}{2} = 4.62 - \frac{1.99 \times 4.00}{2}$

$= 4.62 - 3.98 = 0.64t$

$M = R_1x - \frac{W_4x^2}{2} + \frac{W_4x^3}{6h} + M_{1.8}$

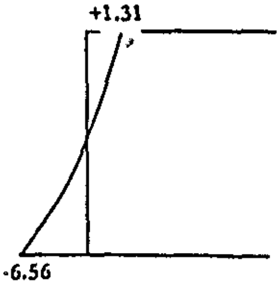
$= 4.62x - \frac{1.99}{2}x^2 + \frac{1.99x^3}{6 \times 4.0} - 6.56$

$= 4.62x - 0.995x^2 + 0.083x^3 - 6.56$

$$\frac{dM}{dx} = 4.62 - 1.99x + 0.25x^2 = 0$$

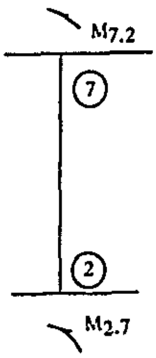
$$x^2 - 7.96x + 18.48 = 0$$

$$x = 3.98 \pm \sqrt{(3.98)^2 - 18.48}$$



There is no solution. Therefore there is no maximum value.

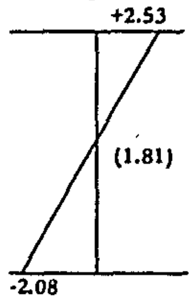
Side wall (2) - (7)



$$R_2 = S_2 = \frac{M_{2.7} + M_{7.2}}{h}$$

$$= -\frac{(-2.08 - 2.53)}{4.0} = 1.15t$$

$$-R_7 S_7 = -\frac{M_{2.7} + M_{7.2}}{h} = 1.15t$$



$$M = R_2 x + M_{2.7}$$

$$= 1.15x - 2.08$$

$$M = 1.15x - 2.08 = 0$$

$$x = \frac{2.08}{1.15} = 1.81m$$

Summary of moments (M)

Slab (joint (1) -6.56(t-m)
midway +4.34
joint (2) - 3.46

(joint (2) - 5.52
midway + 3.77
joint (3) - 5.52

(joint (8) + 1.31
midway + 3.55
joint (7) - 6.85

(joint (7) - 3.80
midway + 1.00
joint (6) - 3.80

Sidewall (joint (1) - 6.56
joint (8) + 1.31
(joint (2) - 2.08
joint (7) + 2.53

Calculation for reinforcement

The calculation done by moments and axial forces are shown below.

Psa = allowable tensile stress of reinforcement bars which is 1,200kg/cm^2 (= 17,000 lbs/in^2)
Pca = allowable compressive stress of concrete which is 50 kg/cm^2 (= 710 lbs/in^2)
d min = minimum effective depth required in cross section.
b = unit width which is 100 cm
C_1 = coefficient which is 0.345

$$d\ min \geqq C_1 \sqrt{\frac{M}{b}}$$

No.	Position	C_1	M	M/b	$\sqrt{M/b}$	d min	Designed effective depth with haunching
			(kg-cm)			(cm)	
1	Slab joint (1)	0.345	- 656,000	6,560	81.0	27.9 <	50 + 30 - 6 = 74 (cm)
2	Slab midway (1) - (2)	0.345	+ 434,000	4,340	65.9	22.7 <	50 - 6 = 44
3	Slab joint (2) - (3)	0.345	- 552,000	5,520	74.3	25.6 <	50 + 30 - 6 = 74
4	Slab midway (8) - (7)	0.345	+ 355,000	3,550	59.6	20.6 <	45 - 6 = 39
5	Slab joint (7)	0.345	- 635,000	6,350	79.7	27.5 <	45 + 30 - 6 = 69
6	Slab joint (6) - (7)	0.345	- 380,000	3,800	61.6	21.3 <	45 + 30 - 6 = 69
7	Sidewall joint (1)	0.345	- 656,000	6,560	81.0	27.9 <	50 + 30 - 6 = 74
8	Sidewall joint (7)	0.345	+ 253,000	2,530	50.3	17.4 <	50 + 30 - 6 = 74

$$As = \frac{C_1 C_2 Ms}{d} - \frac{N}{Psa}$$

$$Ms = M + NC$$

$$C = \frac{t}{2} - d'$$

$$C_1 \times C_2 = 0.00096$$

As = Area of reinforcement bars required

C_2 = Coefficient which is 0.00277

N = Axial force

d = designed effective depth with hunching

t = overall depth in cross section

d' = concrete cover

No	Position	M kg-cm	N kg	t cm	d' cm	C cm	NC kg-cm	Ms kg-cm
1	Slab joint (1)	- 656,000	4,620	50	6	19.0	87,780	743,780
2	Slab midway (1) - (2)	+ 434,000	2,880	50	6	19.0	54,720	488,720
3	Slab joint (2) - (3)	- 552,000	1,150	50	6	19.0	21,850	573,850
4	Slab midway (8) - (7)	+ 355,000	900	45	6	16.5	14,850	369,850
5	Slab joint (7)	- 635,000	1,150	45	6	16.5	18,975	653,975
6	Slab joint (6) - (7)	- 380,000	1,150	45	6	16.5	18,975	398,975
7	Sidewall joint (1)	- 656,000	11,510	50	6	19.0	218,690	874,690
8	Sidewall joint (7)	+ 253,000	20,570	50	6	19.0	390,830	643,830

d cm	0.00096Ms/d	N/1200	As
74	9.65cm²	3.85cm²	5.80cm²
44	10.66	2.40	8.26
74	7.44	0.96	6.48
39	9.10	0.75	8.35
69	9.10	0.96	8.14
69	5.55	0.96	4.59
74	11.35	9.59	1.76
74	8.35	17.14	-

Diameter and spacing of reinforcing bars

For (1) (2) (3) (4) (5) and (6)

(d = 16mm (5/8") ctc 50cm (=20")

d = 19mm (3/4") ctc 50cm (=20")] ∴ Bars to be placed alternately at 10" ctc.

For (7) and (8)
d = 13mm (½") ctc 25cm (=10")

Calculation for compressive stress (Pc)

at joint (7) N = 20,570 kg Ms = 643,830 kg-cm
e = Ms/N = 643,830/20,570 = 31.3cm

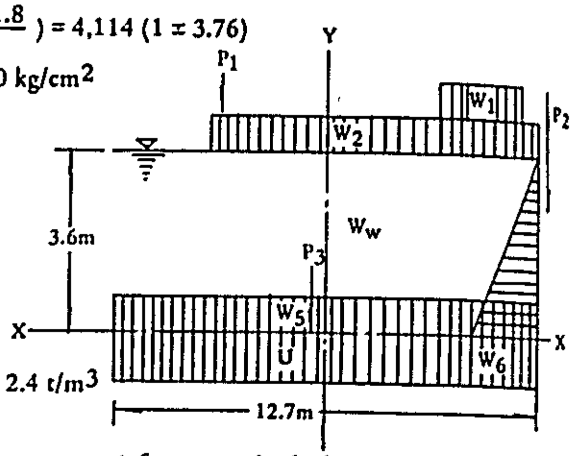
$$\therefore \ Pc = \frac{N}{bL}\left(1 \pm \frac{6e}{L}\right) = \frac{20{,}570}{100 \times 50}\left(1 \pm \frac{6 \times 31.8}{50}\right) = 4{,}114\ (1 \pm 3.76)$$

$Pc = 19.6\ kg/cm^2$ or $- 11.4 kg/cm^2 < Psa = 50\ kg/cm^2$

Reinforcement bars are to be used for tensile stress.

(2) Stability analysis

Conditions
Water level at sump = 102.00'
Sill level at sump = 90.00'
Bulk density of concrete Wc = 2.4 t/m^3
Coefficient of uplift U = 0.4



W_w = weight of water in sump which is

$$\frac{3.6 \times 12.50 \times 1.0 \times (3 \times 3.0)}{3 \times 3.5} = 38.57t$$

W_1 = weight of pumps which is
$W_1S = 3.04 \times 2.1 = 6.38t$

W_2 = weight of top slab which is
$W_2 l_2 = 1.08 \times 8.1 = 8.75t$

W_5 = weight of bottom slab which is
$W_5 l_5 = (0.5 \times 1.0 \times 1.0 \times 2.4) \times 12.7$
$= 15.24t$

W_6 = hydrostatic pressure acting against backwall which is
$½W_6h = ½ \times (1.0 \times 3.6) \times 3.6 = 6.48t$

U = uplift acting upward against bottom slab which is
$uw_ohA = 0.4 \times 1.0 \times 3.6 \times 12.7$
$= - 20.83t$

(cf. structural calculation)

P_1 = weight of building supported by columns which is

$$\frac{4 \times 20.0}{3 \times 3.5} = 7.62t$$

P_2 = weight of backwall which is

$$\frac{2(4.42 + 4.63) \times 12.5}{3 \times 3.5} = 21.50t$$

	Weight	Arm	M moment	Remarks
P_1	7.62t	1.25m	-9.53t-m	
P_2	3.78	6.35	24.00	
P_3	21.50	-0.10	-2.15	
Ww	38.57	-0.10	-3.86	
W_1	6.38	5.00	31.90	
W_2	8.75	2.30	20.13	
W_5	15.24	0	0	
W_6	6.48	1.70	11.19	
U	- 20.83	0	0	
Total	H=6.48		71.99	$e = \frac{M}{V}$
	V=81.01			$= \frac{71.99}{81.01} = 0.89m$

- Stability against overturning

  Arm of eccentricity is 0.89m $< \frac{e}{6} = \frac{12.7}{6} = 2.12$m

Therefore, it is found that the resultant of these forces are within the middle third of the base.

- Stability against sliding

  total V = 81.01t m = Coefficient of friction between concrete and earth which is 0.5.

  total H = 6.48

  $m \geq \frac{2\Sigma H}{\Sigma V}$

  $= \frac{2 \times 6.48}{81.01} = \frac{12.96}{81.01} = 0.16 < 0.5$

- Bearing capacity of the earth

  total V = 81.01t

  e = 0.89m

  maximum and minimum compressive stresses are,

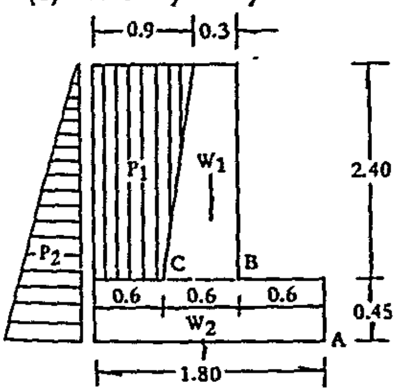
  max. min. $Pc = \frac{V}{bL}(1 \pm \frac{6e}{L}) = \frac{81.01}{1.0 \times 12.7}(1 \pm \frac{6 \times 0.89}{12.7}) = 63.79(1 \pm 0.42)$

  $\therefore$ max $Pc = 9.06t/m^2$ min $Pc = 3.70t/m^2$

The bearing capacity of the earth seems to be insufficient for this maximum compressive stress. Design of piling with a result of soil test will therefore be necessary.

8) Retaining wall for connecting canal

(1) Stability analysis



Moment at point of A

	Weight	Arm	Moment
P_1	2.88 t	1.43 m	4.10 t-m
P_2	1.95	0.95	-1.85
W_1	2.59	0.83	2.14
W_2	1.94	0.90	1.70
Total	V=7.41		M=6.14
	H=1.95		

Conditions (No water in canal

Bulk density of concrete $W_c = 2.4t/m$

Bulking density of soil $We = 1.6t/m$

$P_1 = ½(0.9 + 0.6) \times 2.4 \times 1.6 = 2.88t$

$P_2 = ½ \times 0.3 \times (2.85)^2 \times 1.6 = 1.95t$

$W_1 = ½(0.3 + 0.6) \times 2.4 \times 2.4 = 2.59t$

$W_2 = 0.45 \times 1.8 \times 2.4 = 1.94t$

* Stability against overturning

$\therefore e = \frac{L}{2} - x = \frac{1.80}{2} - 0.83 = 0.07m < \frac{L}{6} = \frac{1.80}{6} = 0.30m$

Moment of forces

Mf = 4.10 + 2.14 + 1.75 = 7.99 t-m

Moment of resistance

Mr = 1.85 t-m Mf/Mr = 7.99/1.85 = 4.3 > 1.5

Therefore this wall is safe.

* Stability against sliding
  This wall is safe because htc bed of the canal will be paved between these walls.

* Bearing capacity of earth

$$\begin{matrix}\text{max.}\\ \text{min.}\end{matrix}\ Pc = \frac{V}{bL}(1 \pm \frac{6e}{L}) = \frac{7.41}{1.0 \times 1.8}(1 \pm \frac{6 \times 0.07}{1.8}) = 4.12\,(1 \pm 0.023)$$

$\therefore$ max. $Pc = 5.07t/m^2$ min. $Pc = 3.17t/m^2$

(2) Structural calculation

Design for wall

P = earth pressure at the back of wall which is $\frac{1}{2} \times 0.3 \times (2.4)^2 \times 1.6 = 1.38t$

M = moment at the point of B which is $1/3\ Ph = 1/3\,(1.38 \times 2.4) = 1.104t\text{-}m = 110{,}400$ kg-cm

N = axial force which is W_1 (=2.59t = 2,590kg)

$Ms = M + NC$

$C = \frac{t}{2} - d' = \frac{60}{2} - 6 = 24$ cm

(t = 60cm

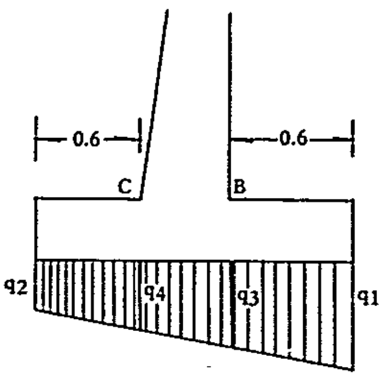
d' = 6cm $\therefore$ $Ms = 110{,}400 + 2{,}590 \times 24 = 172{,}600$ kg-cm

As = area of reinforcement bars,which is

$$\frac{C_1C_2Ms}{d} - \frac{N}{Psa} = \frac{0.00096 \times 1.726}{54} - \frac{2{,}590}{1{,}200} = 0.91cm^2$$

($d = 60 - 6 = 54$cm) $\therefore$ diameter and spacing of bars will be d = 9mm (= 3/8" ctc 25cm (=10")

* Design for toe



q_3 = reaction force at B which is

$3.17 + \frac{(5.07 - 3.17)}{1.8} \times 1.2 = 3.17 + 1.27 = 4.44\ t/m^2$

M_1 = moment due to reaction force of ground which

is $4.44 \times 0.6 \times \frac{0.6}{2} + (5.07 - 4.44) \times \frac{0.6}{2} \times \frac{0.6 \times 2}{3} = 0.88t\text{-}m$

M_2 = moment due to toe which is

$0.45 \times 0.6 \times 2.4 \times \frac{0.6}{2} = 0.19$ t-m

$M_B = M_1 - M_2 = 0.88 - 0.19 = 0.69$ t-m

$As = \frac{MB}{Psa\,jd} = \frac{69{,}000}{1{,}200 \times 7/8 \times 39} = 1.68cm^2$

($j = 7/8$) d = 9mm (=3/8") ctc 25cm (=10")

* Design for heel

q_4 = reaction force at C which is,

$3.17 + \frac{(5.07 - 3.17)}{1.8} \times 0.6 = 3.17 + 0.63 = 3.80t/m^2$

moment due to reaction force of ground

$M_1 = 3.80 \times 0.6 \times \frac{0.6}{2} + (3.80 - 3.17) \times \frac{0.6}{2} \times \frac{0.6}{3} = 0.72$ t-m moment due to heel and earth on it, (P_1 = 2.88t)

$$M_2 = (0.45 \times 0.6 \times 2.4 \times \frac{0.6}{2}) + (2.88 \times 0.225) = 0.84\text{t-m}$$

$$Mc = M_1 - M_2 = 0.72 - 0.84 = -0.12\text{t-m}$$

$$As = \frac{Mc}{Psa\,jd} = \frac{12{,}000}{1{,}200 \times 7/8 \times 39} = 0.29\text{cm}^2$$

d = 9mm (=3/8") ctc 25 cm (=10")
bars to be placed in the top section of hee.

9) Design for piling

Two borings and a penetrometer sounding had been carried out on the site for the proposed pumping station.

The results and records were received at the beginning of July. In this report, design of piling will be done with the use of the results.

According to the result of a penetrometer sounding, the resistance of earth deeper than 7m below the surface can be expected more than 4kg/cm^2 (=40t/m^2). Therefore, piling must be carried out deep enough to reach the harder layers.

(1) Type of soil: Clay
(2) Assumed bearing capacity of earth under the structural base by the result of a penetrometer sounding: 1.5 kg/cm^2 (=15t/m^2)
(3) Calculated maximum compressive stress of each structure:
 (a) at the footing of pier 18.0t/m^2
 (b) at the base of pump 9.06t/m^2
 (c) at the footing of retaining wall 5.07t/m^2
 As there is no necessity for piling in case of (c) designs of piling for (a) and (b) will be done below.
(4) Type of pile: Wooden friction and end-bearing pile.
(5) Formula to be used.
 (a) For determination of length and diameter.
 Dorr's formula:

$$R = \frac{\pi}{4} d^2 r_t\, l \tan^2 (\frac{\pi}{4} + \frac{\theta}{2}) + \frac{\pi}{2} d r_t l^2 U(1 + \tan^2\theta)$$

R: limiting bearing capacity of earth
r_t: mean bulk density of earth which is 1.53t/m^3 (=955 lbs/ft^3) according to the result of soil test.
l: length of pile required
d: diameter of pile required
θ: interior friction angle of earth which is averagely 3° according to the result of soil test.
U: coefficient of friction between pile and earth which is chosen to be 0.1

(b) For determination of interval of piles
Bierbaumer's formula modified by Mr. Minami

$$L = \sqrt{d^2 + 2\,ld \tan\left(\frac{\pi}{6} - \frac{\theta}{3}\right)}$$

L: interval of piles

(6) Calculation by above formulas

(a) At the footing of pier

Assuming that $l = 8.0m$ (=26.6')
$d = 0.3m$ (= 1.0')

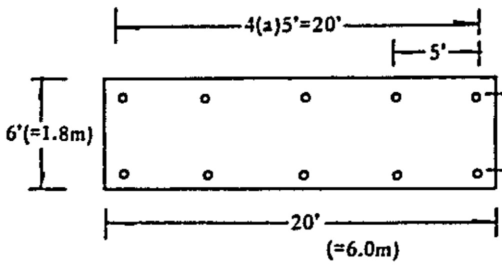
$$R = \frac{3.14}{4} \times (0.3)^2 \times 1.53 \times 8.0 \tan^2 (45^\circ + \frac{30}{2}) + \frac{3.14}{2} \times 0.3 \times 1.53 \times (8.0)^2 \times 0.1 \times (1 + \tan^2 3^\circ)$$

$$= 0.785 \times 0.09 \times 1.53 \times 8.0 \times (1.0538)^2 + 1.57 \times 0.3 \times 1.53 \times 64.0 \times 0.1 \times [1 + (0.0524)^2]$$

$$= 0.96 + 4.63 = 5.59\ t$$

$$L = \sqrt{(0.30)^2 + 2 \times 8.0 \times 0.3 \tan (30^\circ - \frac{3^\circ}{3})}$$

$$= \sqrt{0.09 + 4.8 \times 0.5548} = \sqrt{2.75} = 1.66m\ (=5.5' \doteqdot 5.0')$$



Piles can be disposed as in figure at left. Therefore bearing capacity of piles will be,

$$\frac{5.59 \times 10}{1.8 \times 6.0} = \frac{55.9}{10.8} = 5.18t/m^2$$

Thus, total bearing capacity will be,
$15.00 + 5.18 = 20.18t/m^2 > 18.0t/m^2$

(b) At the base of sump

Assuming that $l = 6.0m$ (=20')
$d = 0.2m$ (= 8")

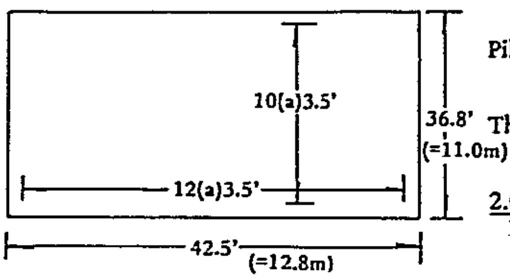
$$R = \frac{3.14}{4} \times (0.2)^2 \times 1.53 \times 6.0 \tan^2 (45^\circ + \frac{3^\circ}{2}) + \frac{3.14}{2} \times 0.2 \times 1.53 \times (6.0)^2 \times 0.1\ (1 + \tan^2 3^\circ)$$

$$= 0.785 \times 0.04 \times 1.53 \times 6.0 \times (1.0538)^2 + 1.51 \times 0.2 \times 1.53 \times 36.0 \times 0.1 \times (1 + 0.0524)^2$$

$$= 0.32 + 1.73 = 2.05t$$

$$L = \sqrt{(0.2)^2 + 2 \times 6.0 \times 0.2 \tan (30^\circ - \frac{3^\circ}{3})}$$

$$= \sqrt{0.04 + 2.4 \times 0.5543} = \sqrt{1.37} = 1.17m\ (=3.9' \doteqdot 3.5')$$



Piles can be disposed as in figure at left.

Therefore, unit bearing capacity of piles will be,

$$\frac{2.05 \times (11 \times 13)}{11.0 \times 12.8} = \frac{298.2}{140.8} = 2.08t/m^2$$

Although the earth itself seems to have sufficient bearing capacity against the maximum compressive stress by sump, piling is advisable for safety.

Thus total bearing capacity will be,

$$15.00 + 2.08 = 17.08t/m^2 > 9.06t/m^2$$

Computation for quantity of materials

1. SectionA - A
   (1) Volume of cut $(1.75 \times \frac{11.0 + 46}{2}) \times 34.5 = 772.8m^2$
   (2) Volume of fill $[\ (1.25 \times \frac{1.8 + 8.0}{2}) + (1.50 \times \frac{1.8 + 9.4)}{2}\ ] \times 34.5 = 501.3m^3$

2. Section B - B
   (1) Wingwalls
      (a) Volume of concrete $2\ [\ (1.5 \times 1.5) + (1.5 \times 2.1) + (2.7 \times 2.25) + (1.5 \times 0.45) + (1.5 \times 0.9) + (1.60 \times 1.35)\ ] \times 0.45 = 14.10m$
      (b) Weight of reinforcement bars $14.10 \times 0.0021 \times 7.8 = 0.231t$
      (c) Area of forms $4\ [(1.5 \times 1.5) + (1.5 \times 2.1) + (2.7 \times 2.25) + (0.5 \times 1.5)\ ] = 48.92m^2$
   (2) Toewall
      (a) Sheet piles $3.0 \times 12.3 = 36.9m^2$
      (b) Volume of concrete on sheet piles $0.75 \times 0.45 \times 12.3 = 4.15m^3$
      (c) Area of forms $2 \times 0.75 \times 12.3 = 18.45m^2$

3. Section C - C
   (1) Volume of cut $(1.30 \times \frac{12.3 + 14.9}{2})\ \times 6.0 = 106.1m^3$
   (2) Volume of fill and refill $[(1.2 \times \frac{7.0 + 10.0}{2}) + (1.5 \times \frac{7.0 + 11.0}{2}) + (1.30)^2 \times 6.0\ ] = 152.4m^3$
   (3) Intake - Piers, footings and gate framing
      (a) Volume of concrete
         Footings $4 \times (1.8 \times 0.45 \times 6.0) = 19.44m^3$
         Piers $4\ [(2,1 \times 0.5 \times 4.05) + (\frac{2.1 + 2.4}{2} \times 0.5 \times 1.95)\ ] = 25.76m^3$
         Frames $[\ (\ 0.45 \times 11.0) + (6 \times \frac{0.15 \times 0.45)}{2} + (\ 4 \times 2.55 \times 0.5)\ ] \times 1.5 = 15.38m^3$
         Bases between footings $3 \times 1.7 \times 0.45 \times 6.0 = 13.77m^3$
      (b) Weight of reinforcement bars $(19.44 + 25.76 + 15.38) \times 0.0045 \times 7.8 = 2.126t$
      (c) Area of forms
         Footings $8[(0.45 \times 6.0) + (0.45 \times 1.8)] = 28.08m^2$
         Piers $8[(2.1 \times 4.05) + (\frac{2.1 + 2.4}{2} \times 1.95)] + 4[(2.1 \times 0.5) + (2.4 \times 0.5)] = 112.20m^2$
         Frames $2[(11.0 \times 0.45) + (6 \times \frac{0.45 \times 0.15}{2}) + (4 \times 2.55 \times 0.5)] + [(6 \times 2.4) + (2 \times 3.0) + (3 \times 3.2)] \times 1.5 = 68.70m^2$
      (d) Gates, winches and attachments -- 3 sets (3.24m x 2.1m) (= 11' x 7')

4. Section D - D

(1) Volume of cut $(1.70 \times \frac{12.4 + 15.8}{2}) \times 9.0 = 215.7m^3$

(2) Volume of fill and refill $[(1.10 \times \frac{7.2 + 11.0}{2}) + (1.50 \times \frac{7.2 + 10.2}{2}) \times (1.70)^2] \times 9.0 = 233.6m^3$

(3) Canal

(a) Volume of concrete

Retaining walls $2 \times [(2.4 \times \frac{0.3 + 0.6}{2}) + (0.45 \times 1.8)] \times 9.0 = 34.02m^3$

Invert $[(0.3 \times 9.0) + (\frac{0.6 + 0.9}{2} \times 0.24) + \frac{3.4 + 10.0}{2} \times 0.3] \times 8.8 = 43.03m^3$

(b) Weight of reinforcement bars $34.02 \times 0.0021 \times 7.8 = 0.557t$

(c) Area of forms

Retaining walls $4 [(\frac{0.3 + 0.6}{2} \times 2.4) + (0.45 \times 1.8)] + 2[(2.4 \times 9.0) + (2.5 \times 9.0)] + 4 \times 0.45 \times 9.0 = 111.96m^2$

5. Section E - E

(1) Volume of cut $(2.65 \times \frac{11.0 \times 16.3}{2}) \times 3.90 = 141.1m^3$

(2) Volume of fill and refill $[(1.20 \times \frac{7.0 + 10.3}{2}) + (1.50 \times \frac{7.0 + 11.2}{2}) + (2.65)^2] \times 3.90 = 121.1m^3$

(3) Inlets of sumps

(a) Volume of concrete $[4 \times (3.65 \times 0.5) + 3 \times (3.0 \times 0.5)] \times 3.90 = 46.02m^3$

(b) Weight of reinforcement bars $46.02 \times 0.0050 \times 7.8 = 1.795t$

(c) Area of forms $2[4 \times (3.65 \times 0.5) + 3 \times (3.0 \times 0.5)] + [(6 \times 3.15) + (2 \times 3.65)] \times 3.90 = 125.78m^2$

(4) Screens 3 sets of 3.0m x 3.45m

6. Section F - F

(1) Volume of cut $(3.65 \times \frac{11.0 + 18.10}{2}) \times 8.85 = 457.1m^3$

(2) Volume of fill and refill $[(1.10 \times \frac{7.0 + 10.5}{2}) + (1.20 \times \frac{7.0 + 10.6}{2}) + (3.55)^2] \times 8.85 = 290.2m^3$

(3) Sumps

(a) Volume of concrete $[(4 \times 4.45 \times 0.5) + (3 \times 3.0 \times 0.45) + (3 \times 3.0 \times 0.50) + 12 \times (\frac{0.3}{2})^2 \times 8.40 + (11.0 \times 1.45 \times 0.45) + (6 \times (\frac{0.3}{2})^2 \times 3.50) - [3 \times \frac{3.14}{4} \times (1.0)^2 \times 0.45] = 173.04m^3$

(b) Weight of reinforcement bars $173.04 \times 0.0050 \times 7.8 = 6.749t$

(c) Area of forms $(4 \times 4.45 \times 0.5) + (3 \times 3.0 \times 0.45) + (3 \times 3.0 \times 0.50) + 12 \times (\frac{0.3}{2})^2 + (11.0 \times 4.45) + 3 \times [3.28 \times 3.5 - 4 \times (\frac{0.3}{2})^2] + (2 \times 4.45 \times 8.85) + (6 \times 3.78 \times 8.40) = 370.12m^2$

(4) Pumps, motors, reduction gears, suction and delivery pipes etc.

(3 sets of pumps ---------------- axial flow type d = 800mm

(3 sets of motors ----------------- 60kW each

(suction pipes ---------------------- d = 800mm x 2.85m

(delivery pipes --------------------- d = 800mm x 8.30m

7. Section G - G

(1) Volume of cut $(1.3 \times \frac{11.0 + 13.6}{2}) \times 1.6 = 25.6\ m^3$

(2) Volume of fill and refill $(0.8 \times \frac{25.2 + 31.0}{2}) \times 3.2 + [(1.20 \times \frac{7.0 + 9.3}{2})$

$+ (1.40 \times \frac{7.0 + 11.0}{2}) + (1.3)^2] \times 1.6 = 110.4\ m^3$

(3) Structure under delivery pipes

(a) Volume of concrete

Side walls $4 [(1.05 \times 2.70) + (½ \times 1.95 \times 2.70)] \times 0.45 = 9.85 m^3$

Base $5.85 \times 10.8 \times 0.45 = 28.43\ m^3$

(b) Weight of reinforcement bars $(9.85 + 28.43) \times 0.0021 \times 7.8 = 0.627t$

(c) Area of forms $4 [(1.05 \times 2.70) + (½ \times 1.95 \times 2.70)] = 21.88 m^2$

8. Section H - H

(1) Volume of cut $(1.5 \times \frac{10.8 + 13.8}{2}) \times 4.05 = 74.7\ m^3$

(2) Volume of fill and refill $[(1.3 \times \frac{7.15 + 9.30}{2}) + (1.4 \times \frac{7.15 + 11.2}{2}) + (1.5)^2] \times 4.05 = 104.5 m^3$

(3) Water tanks

(a) Volume of concrete $[(10.8 \times 0.45) + 2 \times (3.6 \times 0.45) + 2 \times (2.4 \times 0.45) + 6 \times (\frac{0.3}{2})^2] \times 3.0$

$+ [(3.6 \times 0.45) + (\frac{0.9 + 1.2}{2} \times 0.3) + (\frac{0.3}{2})^2] \times 10.8 = 53.08 m^3$

(b) Area of forms $[2 \times (4.05 + 3.30) + (4 \times 2.4) + (3 \times 2.4) + (6 \times 0.15) + 10.8] \times 3.0 = 129.60 m^2$

9. Section I - I

(1) Volume of cut $½ [(1.20 \times \frac{12.8 + 14.7}{2}) + 0.3 \times \frac{5.2 + 5.8}{2}] \times 7.5 = 66.9\ m^3$

(2) Volume of fill and refill $½[(1.20 \times \frac{7.2 + 9.5}{2}) + (1.45 \times \frac{7.2 + 11.3}{2}) + (1.20)^2$

$+ (1.10 + \frac{3.9 + 6.5}{2}) + (1.20 \times \frac{10.5 + 13.0}{2}) + (0.30)^2] \times 7.5 = 167.9\ m^3$

(3) Transition

(a) Volume of concrete

Retaining walls $[(\frac{0.3 + 0.6}{2} \times 3.0) + (1.8 \times 0.45) + (\frac{0.3 + 0.6}{2} \times 1.5) + (1.8 \times 0.45)] \times$
$7.5 = 27.38 m^3$

Invert $\frac{8.7 + 2.8}{2} \times 0.3 \times 7.5 = 12.94 m^3$

(b) Weight of reinforcement bars $27.38 \times 0.0021 \times 7.8 = 0.448\ t$

(c) Area of forms $2[(\frac{0.3 + 0.6}{2} \times 3.0) + (1.8 \times 0.45) + (\frac{0.3 + 0.6 \times 1.5)}{2} + (1.8 + 0.45)]$

$+ 2 [(\frac{3.0 + 1.5}{2} + (\frac{3.2 + 1.6}{2})] \times 7.5 + 4 \times 0.45 \times 7.5 = 90.55 m^2$

10. Section J - J

(1) Volume of cut $0.30 \times \frac{5.2 + 5.8}{2} \times 6.0 = 9.9m^3$

(2) Volume of fill and refill $[(1.10 \times \frac{3.9 + 6.5}{2}) + (1.20 \times \frac{10.5 + 13.0}{2}) + (0.30)^2 \times 6.0\ ] = 119.5m^3$

(3) Canal

(a) Volume of concrete $[\ (2 \times \frac{0.3 + 0.6}{2} \times 1.5\ ) + (5.2 \times 0.3) + (2 \times \frac{0.6 + 1.2}{2} \times 0.3)\ ] \times 6.0 = 20.70m$

(b) Weight of reinforcement bars $20.70 \times 0.0021 \times 7.8 = 0.339t$

(c) Area of forms $2\ (1.50 + 1.55 + 0.60) \times 6.0 = 43.60m^2$

11. Section K - K , L - L

(1) Volume of cut $1.6 \times \frac{5.5 + 8.7}{2} \times 12.3 = 139.7\ m^3$

(2) Volume of fill and refill $(1.95)^2 \times 7.8 + (1.60)^2 \times 4.5 = 41.2\ m^3$

(3) Culverts

(a) Volume of concrete

Inlets $[(0.45 \times 1.05) + (0.6 \times 0.15) + (0.45 \times 0.75)] \times 5.65 + [(1.5 \times 0.45) + (2 \times \frac{0.3 + 0.6}{2} \times 1.5)]$

$\times 1.05 = 7.22\ m^3$

Boxes $[(5.45 \times 0.3) + (5.45 \times 0.45) + (2 \times 1.2 \times 0.5) + (1.2 \times 0.45) + 8 \times (\frac{0.15}{2})^2\ ] \times 9.0 = 53.28m^3$

Outlets $[(2.25 \times 0.45) + (1.35 \times 0.45) + (0.6 \times 0.15)] \times 5.45 + (1.5 \times 0.45 \times 1.8) + 2 \times 0.5 \times$
$1.5 \times 2.25 = 13.92m^3$

(b) Weight of reinforcement bars $(7.22 + 53.28 + 13.92) \times 0.0021 \times 7.8 = 1.219t$

(c) Area of forms

Inlets $2[(0.45 \times 1.05) + (0.6 \times 0.15) + (0.45 \times 0.75)] + (1.2 + 0.6) \times 5.65 + 2\ (1.5 + 1.55) \times$
$1.05 + 2 \times 1.5 \times 1.25 = 22.13m^2$

Boxes $2[(5.45 \times 0.3) + (5.45 \times 0.45) + (2 \times 1.2 \times 0.5) + (1.2 \times 0.45) + 8 \times (\frac{0.15}{2})^2\ ]$
$+ [(2 \times 9.95) + (4 \times 0.9) + (2 \times 1.7) + (8 \times 0.2)] \times 9.0 = 124.34m^2$

Outlets $2[(2.25 \times 0.45) + (1.35 \times 0.45) + (0.6 \times 0.15)] + (1.8 + 1.2) \times 5.45$
$+ [(4 \times 2.25) + (2 \times 0.5)] \times 1.5 + (2 \times 2.0) \times 1.5 = 40.77m^2$

(d) Planks 2 sets x 2.2m x 1.5m

(e) Gabion baskets ½ (9.6 + 11.4) x 18.0 x 0.9 = 170.1 m

SUMMARY OF QUANTITY OF MATERIALS

No. of section	cut	fill & refill	concrete 1 : 2 : 4	concrete 1 : 3 : 6	reinforcement bars	form works	Others
	m^3	m^3	m^3	m^3	t	m^2	
A - A	772.8	501.3					
B - B		-	14.10	4.15	0.231	48.92	Sheet piles 3m x 12.3m
C - C	106.1	152.4	19.44	13.77	2.126	28.08 112.20	Wooden piles 60 x 8m (d=30cm) gates, winches and attachments 3 sets (=3.24m x 2.1m)
D - D	215.7	233.6	34.02	43.03	0.557	111.96	
E - E	141.1	121.1	46.02		1.795	125.78	Screens 3 sets (=3.00m x 3.45m)
F - F	457.1	290.2	173.04		6.749	370.12	Wooden piles 143 x 6m (d=20cm) pumps 3 sets motors (d=20cm) reduction gears (d=20cm) suction pipes d 800mm 2.85m delivery pipes d 800mm x 8.30m pump house
G - G	25.6	110.4	9.85 28.43		0.627	21.88	
H - H	74.7	104.5	53.08		0.869	129.60	
I - I	66.9	167.9	27.38	12.94	0.448	90.65	
J - J	9.9	119.5	20.70		0.339	43.80	
K - K	139.7	41.2	7.22		1.219	22.13	Planks 2 sets
L - L			53.28 13.92			124.34 40.77	(=2.2m x 1.5m) gabion baskets 170.1 m
Total	2,009.6 m^3 =2,628 yd^3	1,842.1 m^3 =2,409 yd^3	541.62 m^3 =708.4 yd^3	73.89 m^3 =96.6 yd^3	14,960t =32,981 lbs	1,357.28 m^2 =14,690.6 ft^2	

Total cost of works

(1) - 1	Cut by manual labour	$2,628yd^3 \times 0.3 \times \$2.00/yd^3 = \$1,577$
(1) - 2	Cut by dragline	$2,628yd^3 \times 0.7 \times 8¢/yd^3 = \$147$
(2) - 1	Fill and backfill by manual labour	$2,409yd^3 \times 0.3 \times \$2.20/yd^3 = \$1,590$
(2) - 2	Fill by dragline	$2,409yd^3 \times 0.7 \times 5¢/yd^3 = \$84$
(3) - 1	Concrete (1:2:4)	$708.4yd^3 \times \$37.-/yd^3 = \$26,211$
(3) - 2	Concrete (1:3:6)	$96.6yd^3 \times \$34.-/yd^3 = \$3,284$
	Reinforcement bars	32,981 lbs x 35¢ /lb = $11,543
(5)	Forms	$14,609.6ft^2 \times 36¢/ft^2 = \$5,259$
(6)	Sheet piles (10' in length)	12.3m = 40.4' 40.4ft x $30.-/ft = $1,212
(7)	Wooden piles 12' x 27'	60 piles x $104/pile = $6,240
	8' x 20'	143 piles x $44/pile = $6,292
		Total $12,532

(8) Gates (3 sets x 110' x 7.0') and attachment 3 sets x $5,000/set

(9) Screens (3 sets x 10' x 11.5') 3 sets x $260/set = $780

(10) Planks (2 x 7.3' x 5') = 10 planks x 12' x 6' x 7.3') 10 planks x $15 = $150

(11) Gabion baskets $170.1m^3$ = 58.5 baskets of 6' x6' x 3' 58.5 baskets x $102.-/basket = $5,967

Total cost	$85,386
Contingency (10% of above)	$ 8,464
Grand total cost	$93,800

(Remarks)

These unit costs used for the above calculations have detailed back data which were figured out in co-operation with Mr. C. Marshall engineer assistant (II) in Drainage Division.

The detailed back data are not described in this report.

The grand total cost does not include costs for pumping equipment, electricity supply and pump house.

They will have to be estimated subsequently.

Junichi Kitamura

PENETROMETER SOUNDING

Attach Table: 1 to 4
Boring No. 1
Location Northern end of gate
Observer 1 : Kirk
0-10 Dg/cm^2 Inclement weather caused delay
0-100/Dg/cm^2

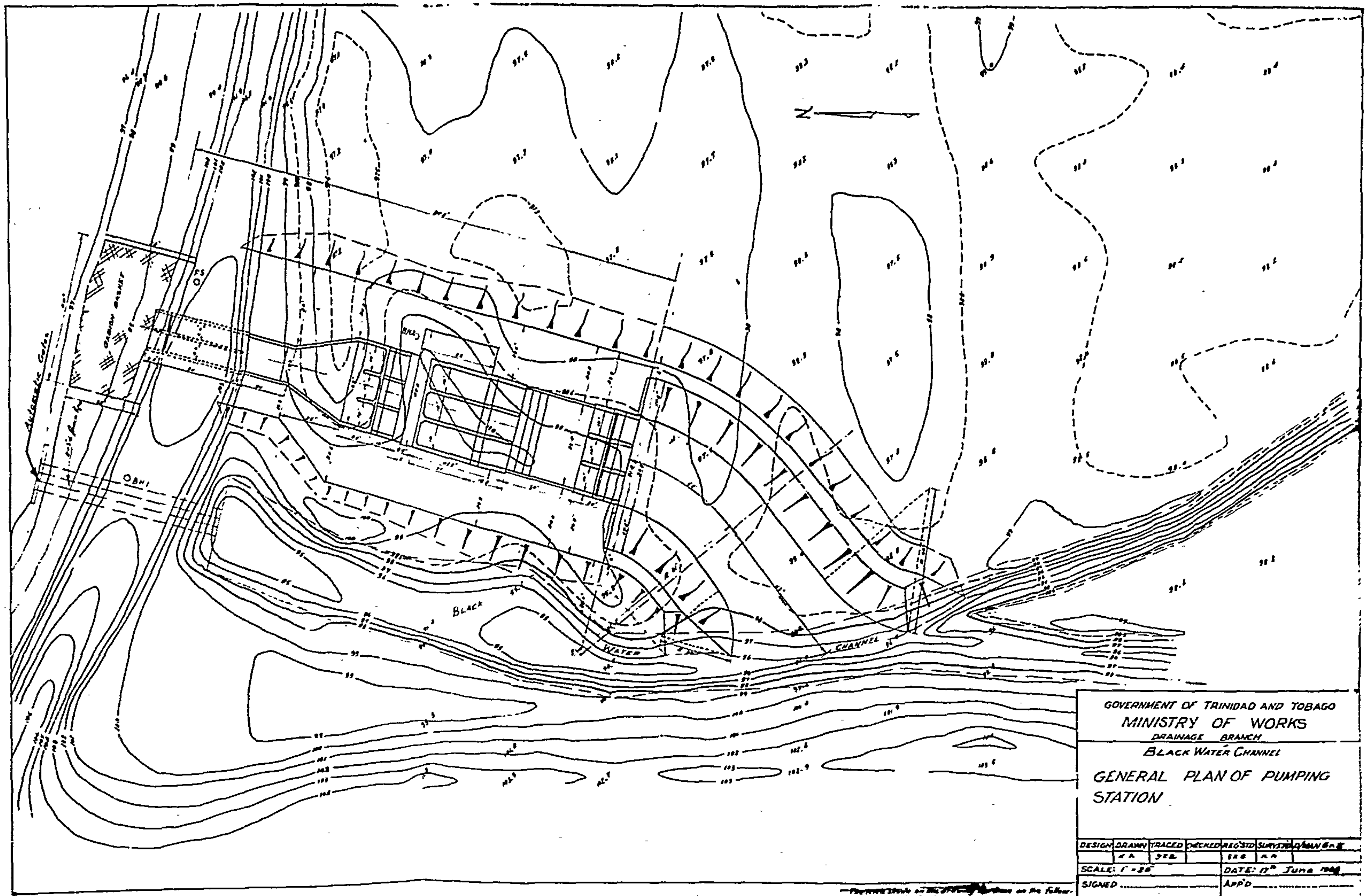
Date 27.5.68
Time started 10.30 a.m.
Time completed

Dep m	Resist kg/cm^2	Dep.m	Resist kg/cm^2	Dep.m	Resist kg/cm^2	Dep. m	Resist kg/cm^2	Dep. m	Resist kg/cm^2	Dep. m	Resist kg/cm^2	Dep. m	Resist kg/cm^2
0.0		3.2	2.0	6.4	2.0	9.6	6.0	12.8	18.0	16.0	22.0	20.0	
0.1		3.3	2.5	6.5	2.0	9.7	9.0	42.9	18.0	16.1	20.0	20.1	
0.2		3.4	2.5	6.6	2.0	9.8	9.0	13.0	15.0	16.2	24.0	20.2	
0.3		3.5	1.5	6.7	2.0	9.9	9.0	13.1	14.0	16.3	28.0	20.3	
0.4		3.6	1.0	6.8	3.0	10.0	10.0	13.2	18.0	16.4	30.0	20.4	
0.5		3.7	1.0	6.9	4.0	10.1	10.0	13.3	18.0	16.5	52.0	20.5	
0.6		3.8	1.0	7.0	4.0	10.2	11.0	13.4	15.0	16.6	84.0	20.6	
0.7		3.9	1.0	7.1	4.0	10.3	11.0	13.5	15.0	16.7	88.0	20.7	
0.8		4.0	1.0	7.2	4.0	10.4	12.0	13.6	18.0	16.8	100.0	20.8	
0.9		4.1	1.0	7.3	3.0	10.5	12.0	13.7	20.0	16.9		20.9	
1.0		4.2	1.0	7.3	4.0	10.6	11.0	13.8	18.0	17.0		21.0	
1.1		4.3	1.0	7.5	4.0	10.7	12.0	13.9	18.0	17.1		21.1	
1.2		4.4	1.0	7.6	4.0	10.8	14.0	14.0	16.0	17.2		21.2	
1.3		4.5	1.0	7.7	4.0	10.9	12.0	14.1	18.0	17.8		21.3	
1.4		4.6	1.0	7.8	4.0	11.0	14.0	14.2	16.0	17.4		21.4	
1.5		4.7	1.0	7.9	4.0	11.1	16.0	14.3	18.0	17.5		21.5	
1.6		4.8	1.0	8.0	4.0	11.2	18.0	14.4	18.0	17.6		21.6	
1.7		4.9	1.0	8.1	4.0	11.3	25.0	14.5	16.0	17.7		21.7	
1.8		5.0	1.0	8.2	4.0	11.4	17.0	14.6	16.0	17.8		21.8	
1.9		5.1	1.0	8.3	4.0	11.5	18.0	14.7	22.0	17.9		21.9	
2.0		5.2		8.4	4.0	11.6	18.0	14.8	22.0	18.0		22.0	
2.1		5.3		8.5	4.0	11.7	18.0	14.9	24.0	18.1		22.1	
2.2		5.4		8.6	4.0	11.8	18.0	15.0	22.0	18.2		22.2	
2.3		5.5		8.7	6.0	11.9	19.0	15.1	18.0	18.3		22.3	
2.4		5.6		8.8	6.0	12.0	16.0	15.2	18.0	18.4		22.4	
2.5		5.7		8.9	10.0	12.1	20.0	15.3	18.0	18.5		22.5	
2.6		5.8		9.0	6.0	12.2	22.0	15.4	16.0	18.6		22.6	
2.7		5.9		9.1	6.0	12.3	22.0	15.5	18.0	18.7		22.7	
2.8		6.0		9.2	6.0	12.4	22.0	15.6	20.0	18.8		22.8	
2.9		6.1		9.3	6.0	12.5	20.0	15.7	22.0	18.9		22.9	
3.0	4.0	6.2		9.4	6.0	12.6	20.0	15.8	22.0	19.0		23.0	
3.1	4.0	6.3		9.5	6.0	12.7	20.0	15.9	22.0	19.1			
										19.2			
										19.3			
										19.4			
										19.5			
										19.6			
										19.7			
										19.8			
										19.9			

SUMMARY OF LABORATORY RESULTS

* - Disturbed
U - Undisturbed
S - Sieve analysis   Job. Oropouche Pump House   L/A H. Maraj

Bore hole	Sample Identity	Type	Classification					Bulk density lb/cu.ft	Dry density lb/cu.ft.	Unconfined shear strength lb/sq.ft	Triaxial		Remarks
			M.C.%	L.L.%	P.I. %	P.I. %	Casagrande classification				C lb/sq.ft	Ø	
1	1/	D	22.6	,69.0	25.7	43.3							
	2/1	D	69.1										
	3/1	D	71.9	146.0	59.7	86.3							
	4/1	U	68.1	145.0	47.9	97.1		96.8	57.6	403.2	576	2.5	insitu.m/c Remoulded at
								92.7	55.1	342.7			insitu m/c Remoulded at 5% + insitu.
	5/	D	78.8										
	6/	D	207.3	222.0	73.5	148.5							
	7/1	D	106.7										
	8/1	D	105.7										
	9/1	D	76.9	100.0	40.3	59.7							
	10/1	D	54.0										
	11/1	D	43.6										
	12/1	D	58.9	104.0	12.9	61.1							
	13/1	U	54.8	100.0	38.5	61.5		99.2	64.1	342.7	403	3.5	insitu m/c. Remoulded at
								103.8	67.0	322.5			insitu. m/c. remoulded at 5% +
								91.8	57.4	126.0			insitu.
	14/1	D	51.4										
2	1/2	D	64.9	133.0	49.5	83.5							
	2/2	D	70.3										
	3/2	U	209.3	253.0	70.1	182.9		72.8	23.5	171.4	-	-	insitu - Peat
								73.0	25.4	151.2			Remoulded at insitu.
	4/2	D	230.3	239.0	82.7	156.3							
	5/2	D	82.5	99.0	40.1	58.9							
	6/2	D	76.3										



—PLAN—

The levels shown on this drawing are based on the following B.M. on ☐ Chiselled on head wall of automatic Gate on Black Water Channel of a value: 98.44

GOVERNMENT OF TRINIDAD AND TOBAGO

MINISTRY OF WORKS

DRAINAGE BRANCH

BLACK WATER CHANNEL

GENERAL PLAN OF PUMPING STATION

DESIGN	DRAWN	TRACED	CHECKED	REG'STD	SURV'D	D/MAN GR. II
	[illegible]	[illegible]		[illegible]	[illegible]	

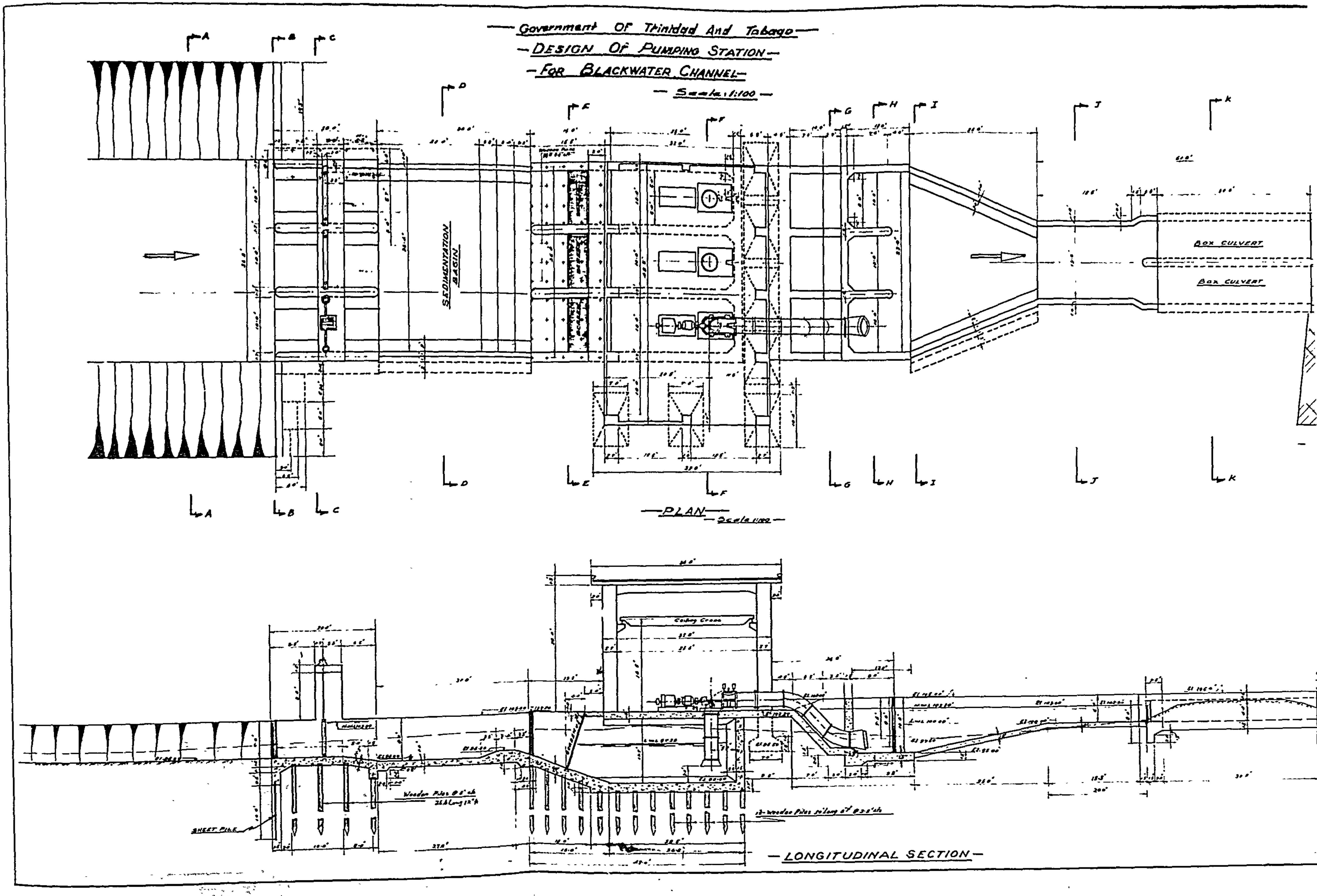
SCALE: 1" = 20' DATE: 17th June [illegible]

SIGNED .... Drainage Engineer

APP'D .... Director of Drainage

Date

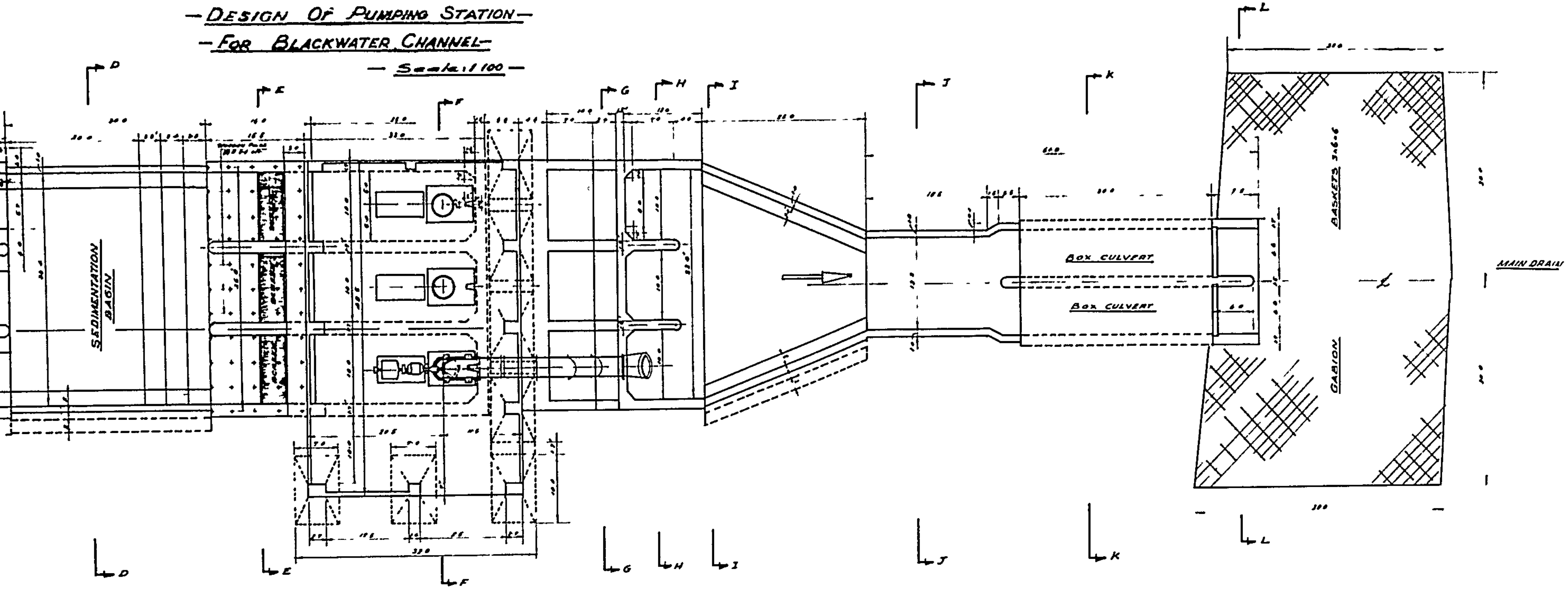
DRAWING No 20325/7/2



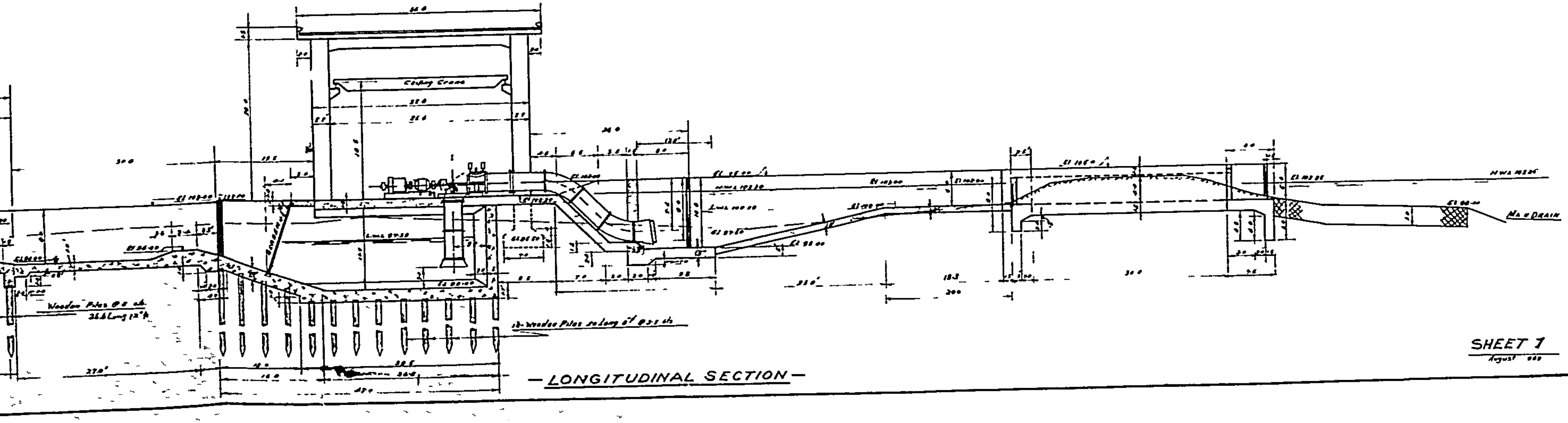
Government Of Trinidad And Tabago
DESIGN OF PUMPING STATION
FOR BLACKWATER CHANNEL
Scale: 1:100
SEDIMENTATION BASIN
BOX CULVERT
BOX CULVERT
PLAN
Scale 1:100
Gantry Crane
Wooden Piles
SHEET PILE
LONGITUDINAL SECTION

Government Of Trinidad And Tobago
Design Of Pumping Station
For Blackwater Channel

Scale 1:100

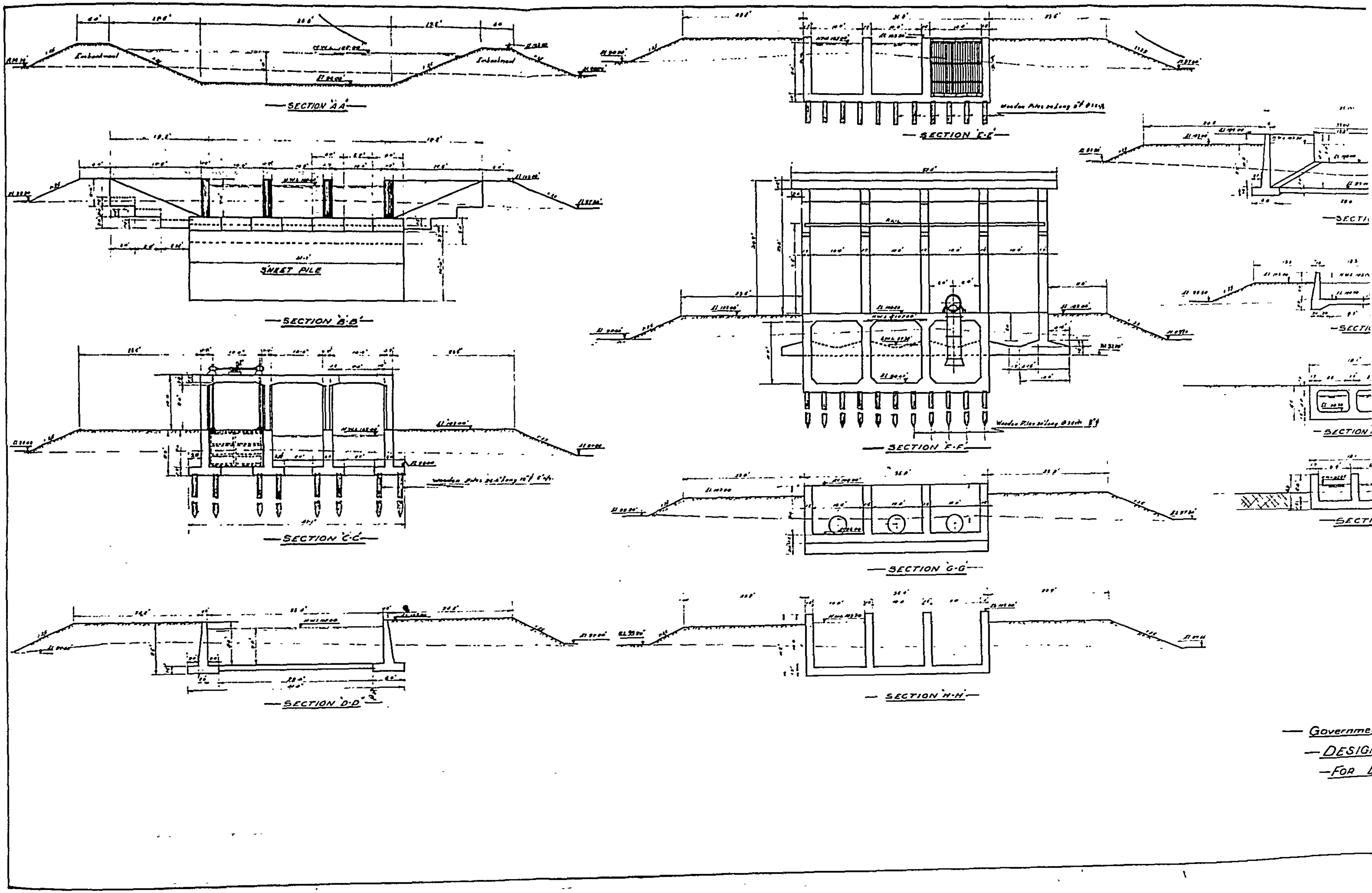


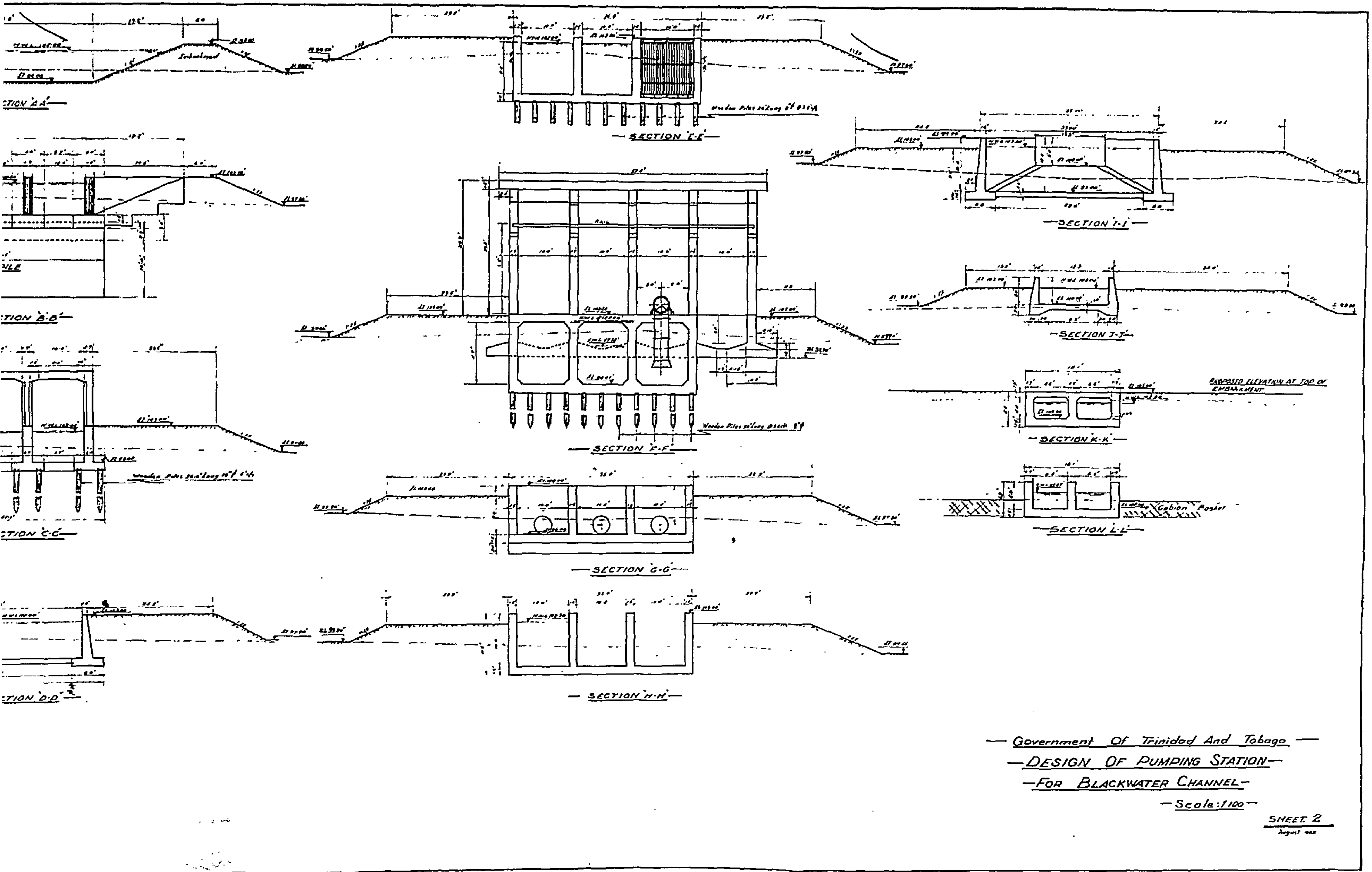
PLAN — Scale 1:100

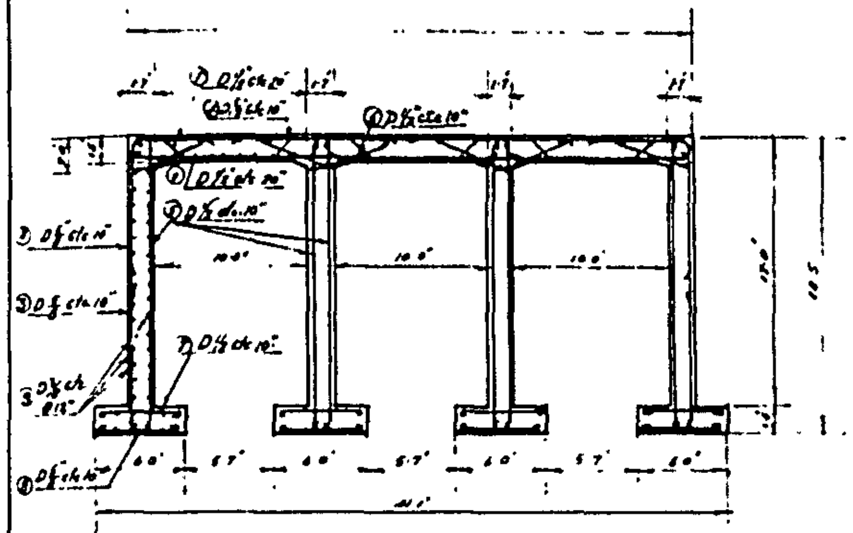


— LONGITUDINAL SECTION —

SHEET 7


Embankment
Embankment
SECTION 'A-A'
SECTION 'B-B'
SHEET PILE
SECTION 'C-C'
SECTION 'D-D'
SECTION 'E-E'
RAIL
SECTION 'F-F'
SECTION 'G-G'
SECTION 'H-H'
FOR


SECTION 'A-A'
SECTION 'B-B'
SECTION 'C-C'
SECTION 'D-D'
SECTION 'E-E'
SECTION 'F-F'
SECTION 'G-G'
SECTION 'H-H'
SECTION 'I-I'
SECTION 'J-J'
SECTION 'K-K'
SECTION 'L-L'
Embankment
Wooden Piles
RAIL
PROPOSED ELEVATION AT TOP OF EMBANKMENT
Gabion Basket
— Government Of Trinidad And Tobago —
—DESIGN OF PUMPING STATION—
—FOR BLACKWATER CHANNEL—
—Scale:1:100—
SHEET 2



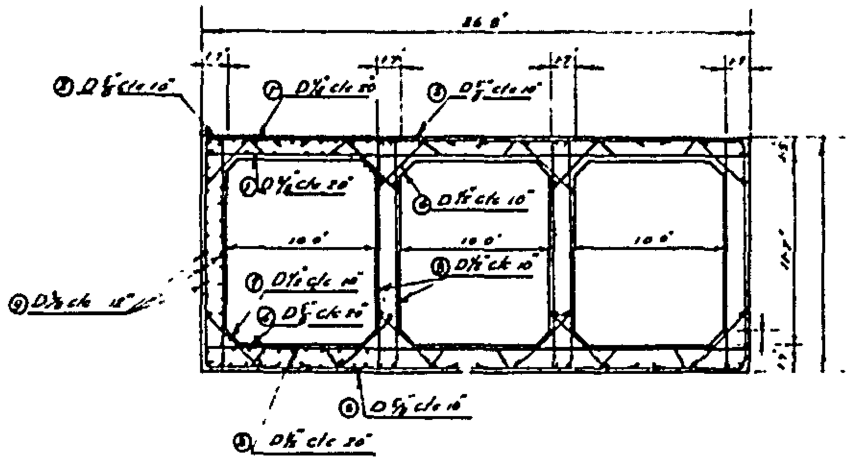
—DETAIL OF INTAKE—

Cross Sectional Area Of Concrete
[illegible]
= [illegible] = 17.49 m²

Cross Sectional Area Of Reinforcing Bar
0.6235/7.8 = 0.0799 m²

∴ Ratio Of Cross Sectional Area Of Reinforcing Bar To That Of Concrete P = 0.0799/17.49 = 0.00451/

UNIT REQUIREMENT OF REINFORCING BAR (Per/m depth)								
No	Diameter mm	Actual Length m	Length Of Hook m (2×8D)	Length Of Joint m (n=20D)	Total Length m	Unit Weight kg/m	Required Number	Weight kg
1	D 13	3.40	0.21	0.62	4.33	1.04	6	27.0
2	D 13	3.90	0.21	0.62	4.63	1.04	6	28.9
3	D 16	4.65	0.26	0.64	5.55	1.58	8	70.2
3	D 16	2.70	0.26	0.32	3.28	1.58	8	41.5
4	D 16	3.50	0.26	0.64	4.40	1.58	8	55.6
5	D 13	5.40	0.21	—	5.61	1.04	24	140.0
6	D 13	1.40	0.21	—	1.61	1.04	24	40.2
7	D 13	1.70	0.21	—	1.91	1.04	16	31.8
8	D 16	1.70	0.26	—	1.96	1.58	16	49.5
9	D 9	1.00	0.14	—	1.14	0.499	242	138.8
	D 9							138.8
	D 13							267.9
	D 16							216.8
	TOTAL							623.5



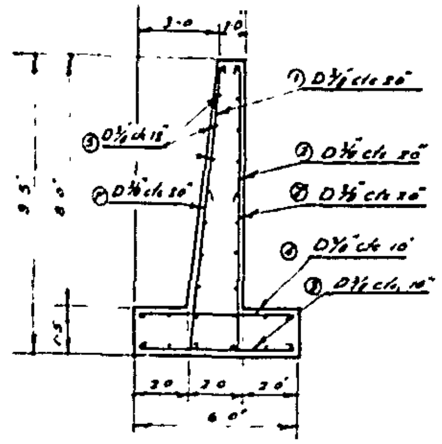
— DETAIL OF SUMP —

Cross Sectional Area Of Concrete
[illegible]

Cross Sectional Area Of Reinforcing Bar
0.697/7.8 = 0.0894

∴ Ratio Of Cross Sectional Area Of Reinforcing Bar To Concrete
P = 0.0894/78 = 0.00030//

UNIT REQUIREMENT OF REINFORCING BAR (Per/m depth)								
No	Diameter mm	Actual Length m	Length Of Hook m (2×8D)	Length Of Joint m (n=20D)	Total Length m	Unit Weight kg/m	Required Number	Weight kg
1	D 16	3.60	0.26	0.64	4.50	1.58	6	42.7
1	D 13	3.90	0.21	0.52	4.63	1.04	6	28.9
2	D 16	8.25	0.26	0.64	9.15	1.58	8	115.7
3	D 16	3.50	0.26	0.64	4.40	1.58	8	55.6
4	D 16	3.60	0.26	0.64	4.50	1.58	6	42.7
4	D 13	3.90	0.21	0.52	4.63	1.04	6	28.9
5	D 16	3.50	0.26	0.64	4.40	1.58	8	65.6
6	D 13	1.30	0.21	—	1.51	1.04	24	37.7
7	D 13	1.35	0.21	—	1.56	1.04	24	38.9
8	D 13	4.35	0.21	—	4.56	1.04	24	113.8
9	D 9	1.00	0.14	—	1.14	0.499	240	136.5
	D 9							136.5
	D 13							248.2
	D 16							312.3
	TOTAL							697.0



—DETAIL OF RETAINING WALL—

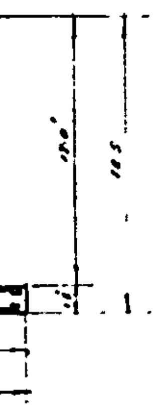
— Scale: 1:50 —

UNIT REQUIREME				
No	Diameter mm	Actual Length m	Length Of Hook m (2×8D)	Length Of Joint m (n=20D)
1	D 9	2.80	0.14	—
1	D 9	1.55	0.14	—
2	D 9	2.70	0.14	—
3	D 9	1.30	0.14	—
3	D 9	1.70	0.14	—
4	D 9	1.70	0.14	—
5	D 9	1.00	—	—
	TOTAL			

— Government Of Tri

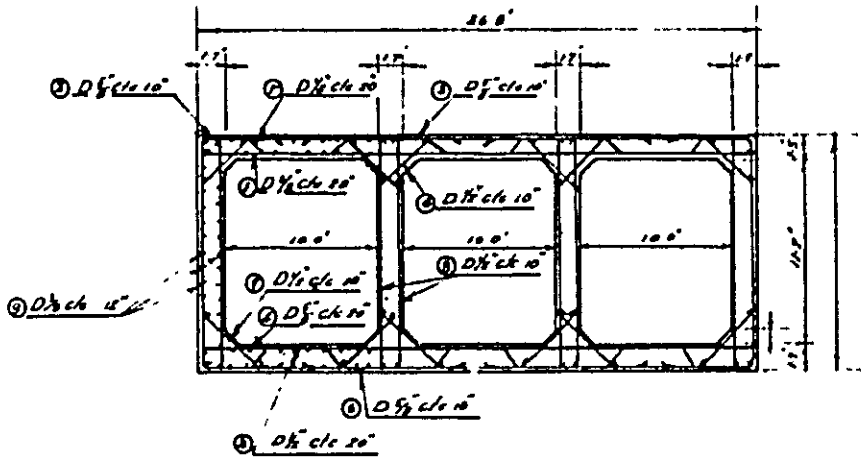
—DESIGN OF PU

—FOR BLACKWA



Of Cross Sectional Area Of Reinforcing Bar
Of Concrete $P = 0.0799/17.69 = 0.0045$//

...ength	Unit Weight Kg/m	Required Number	Weight kg
	1.04	6	27.0
	1.04	6	28.9
	1.58	8	70.2
	1.58	6	41.5
	1.58	8	55.6
	1.04	24	140.0
	1.04	24	40.2
	1.04	16	31.8
	1.58	10	49.5
	0.499	244	138.8
			138.8
			267.9
			216.8
			623.5



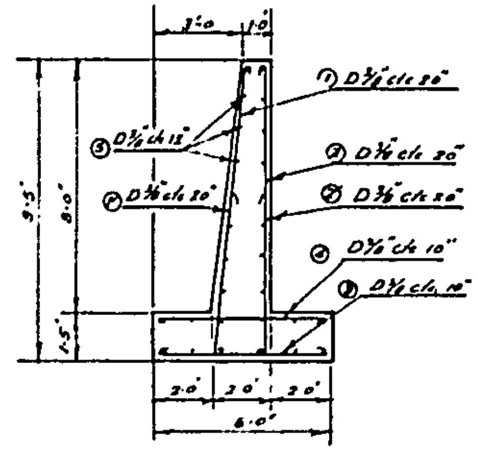
— DETAIL OF SUMP —

Cross Sectional Area Of Concrete
(11.0 × 0.45) + (11.0 × 0.60) + 4 × 0.5 × 3.5 + ½(12 × 0.3 × 0.3)

Cross Sectional Area Of Reinforcing Bar
$0.697/7.8 = 0.0894$

∴ Ratio Of Cross Sectional Area Of Reinforcing Bar To Concrete
$P = 0.0894/7.8 = 0.0050$//

UNIT REQUIREMENT OF REINFORCING BAR (Per/m in depth)								
No	Diameter mm	Actual Length m	Length of Hook m (2×8D)	Length of Joint m (π×20D)	Total Length m	Unit Weight Kg/m	Required Number	Weight Kg
①	D 16	3.60	0.26	0.64	4.50	1.58	6	42.7
①	D 13	3.90	0.21	0.52	4.63	1.04	6	28.9
②	D 16	8.25	0.26	0.64	9.15	1.58	8	115.7
③	D 16	3.50	0.26	0.64	4.40	1.58	8	55.6
④	D 16	3.60	0.26	0.64	4.50	1.58	6	42.7
④	D 13	3.90	0.21	0.52	4.63	1.04	6	28.9
⑤	D 16	3.50	0.26	0.64	4.40	1.58	8	65.6
⑥	D 13	1.30	0.21	—	1.51	1.04	24	37.7
⑦	D 13	1.35	0.21	—	1.56	1.04	24	38.9
⑧	D 13	4.35	0.21	—	4.56	1.04	24	113.8
⑨	D 9	1.00	0.14	—	1.14	0.499	240	136.5
	D 9							136.5
	D 13							248.2
	D 16							312.3
	Total							697.0



— DETAIL OF RETAINING WALL —
— Scale: 1/50 —

Cross Sectional Area Of Concrete
$\frac{1}{2}(0.3 + 0.6) \times 2.4 + (1.8 \times 0.45) = 1.08 + 0.81 = 1.89\ m^2$

Cross Sectional Area Of Reinforcing Bar
$0.0304/7.8 = 0.0039\ m^2$

∴ Ratio Of Cross Sectional Area Of Reinforcing Bar To That Of Concrete
$P = 0.0039/1.89 = 0.0021$//

UNIT REQUIREMENT OF REINFORCING BAR								
No	Diameter mm	Actual Length m	Length of Hook m (2×8D)	Length of Joint m (π×20D)	Total Length kg/m	Unit Weight kg/m	Required Number	Weight kg
①	D 9	2.80	0.14	—	2.94	0.499	2	2.9
①	D 9	1.65	0.14	—	1.69	"	2	1.7
②	D 9	2.70	0.14	—	2.84	"	2	2.8
③	D 9	1.50	0.14	—	1.64	"	2	1.6
③	D 9	1.70	0.14	—	1.84	"	4	3.7
④	D 9	1.70	0.14	—	1.84	"	4	3.7
⑤	D 9	1.00	—	—	1.00		28	14.0
	Total							30.4

— Government Of Trinidad And Tobago —

— DESIGN OF PUMPING STATION —

— FOR BLACKWATER CHANNEL —

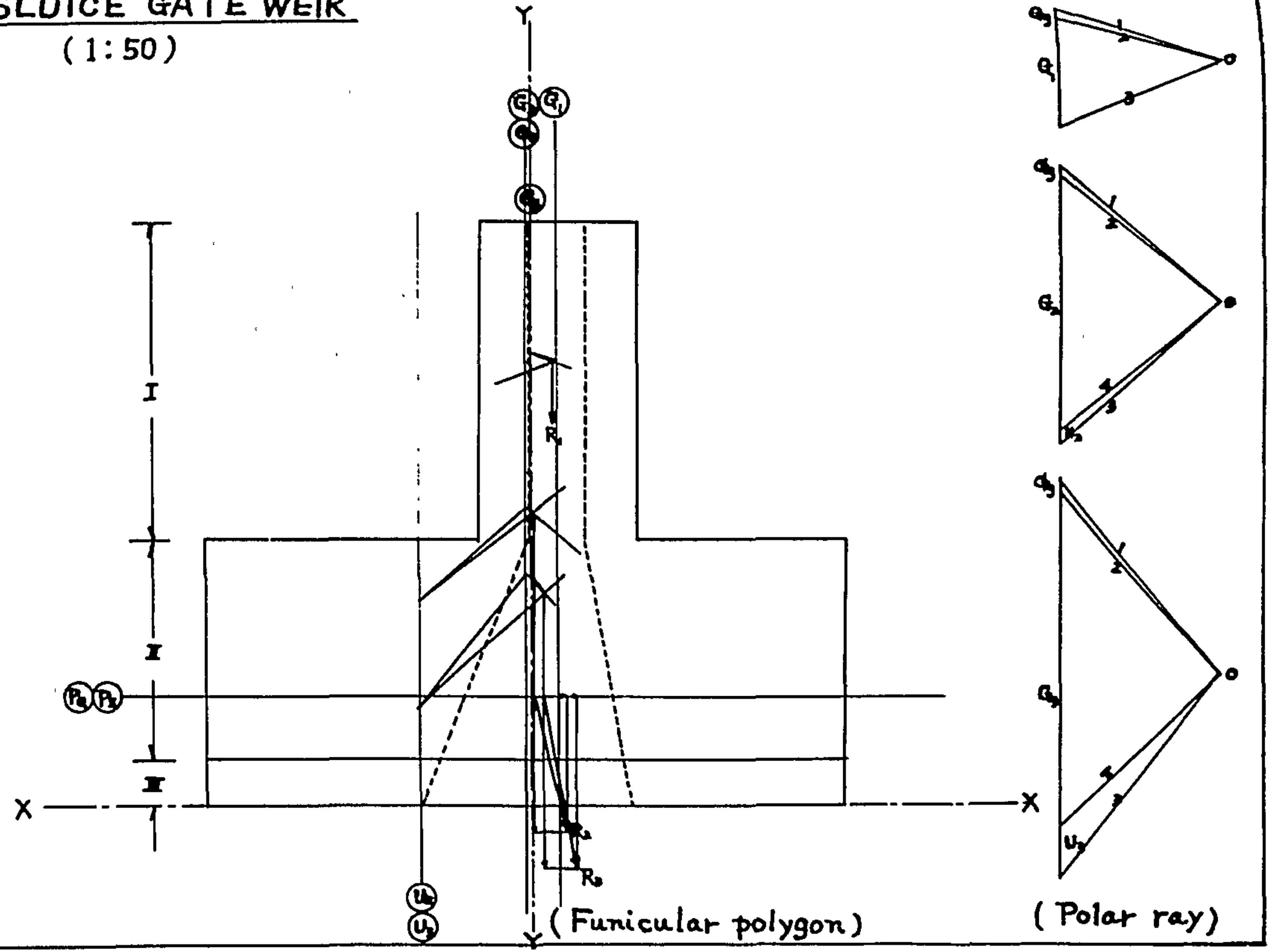
— Scale: 1:100 —

SHEET. 3

August 1968

SLUICE GATE WEIR

(1:50)



MOVEMENT

19, June '68 Received a reply about the electric supply for a pumping station on Blackwater channel, from Mr. M. R. Ramjohn, Area Superinendent, T & TEC, in San Fernando.

22 Aug. Received the estimate for the cost of pumps and machinery from Mr. H.F. Bushe, Neal & Massy Ltd.

10, Oct. Was given information on electric supply, - transmission line, transformer etc., by Mr. Massiah, Electrical Engineer T & TEC.

14, Oct. Received data from the design of the pump from Hitachi, Ltd. Tokyo, Japan.

4 & Feb. '69 Got information on the building cost for the pump house from Mr. Bruce Hamilton, a private architect.

Reference data

A) Dimensions designed by Hitachi Ltd. (per each pump)

Type, Form	AP-GH Horizontal axial flow pump
Suction Bore	900 mm (=35")
Discharge Bore	900 mm (=35")
Quantity	95m^3/min (=3,355 cusecs)
Total Head	2.6m (=8.5ft)
Rotation per Minute	310 r.p.m.
Driver	60kW (=80H.P) motor, 60 cy. 4P
How to be driven	driven by reduction gear

B) Dimensions designed by the reporter (cf Report (IV)) (per each pump)

Type, Form	Horizontal axial flow pump
Suction Bore	800mm
Discharge Bore	800mm
Quantity	93 m^3/min
Total Head	= 7.63 ft
Rotation per Minute	360 r.p.m.
Driver	60kW (=80HP) motor 60 cy 4P
How to be driven	driven by reduction gear

INTRODUCTION

In the report (IV), the pumping station on Blackwater Channel was designed and the cost for its civil works were estimated.

In this report (V), the costs for the machinery their transportation, installation, electric supply and the pump house will be estimated and all of the necessary costs will be totaled. Also, the running cost and the depreciation cost for the pumping operation will be calculated.

MACHINERY COST

According to the principls of Neal and Massy Ltd., the following pumps are recommended:-

Three 30" Axial Flow C.1 Pumps with flooded suction conditions, belt driven by 75 H.P. motors.

The price of each pump complete with belt drive and motor including suction and delivery pipes, would be in the region of $29,000 T.T. ex factory. For three sets of pumps,

3 x $29,000 = $87,000 T.T.

According to the information from Mr. H.F. Buche of Neal & Massy, the items are as follows:-

i)	Cost for pump itself	$18,000 T.T – $20,000 T.T. (for one set)
ii)	Cost for motor	$ 1,600 T.T – $ 2,000 T.T.
iii)	Cost for pipes	$ 6,000 T.T – $ 8,000 T.T.
	Total	$29,000 T.T.

With a view to checking the cost for pipes, the following calculation was carried out.

Weight of pipes:

Straight pipes (800mm) Dia	- 6.7m x 275kg/m = 1,840kg
Pipe Flanges	- 10 pieces x 70kg/piece = 700kg
Pipe Bends	- 2 pipes x 700 kg/pipe = 1,400kg
Bell-mouth pipe for suction (1,200mm)	480kg
Taper pipe with bell-mouth for delivery	(800mm → 1,150mm)
0.8m x ½(275kg/m + 525kg/m) = 240kg	
Check valve (1,150mm)	350kg
Total	5,010 kg (=11,022 lbs)

As the cost per lb of cast iron pipe can be twice as much as the cost per lb of reinforcing bar in ordinary size the approximate cost of the cast Iron Pipes will be

2 x 35¢ /lb = 70¢ /lb

11,022 lbs x 70¢ /lb = $7,715. T.T. ≑ $8,000 T.T.

As a result of above calculation, the estimated cost of pipes by Neal and Massy was approved to be reasonable.

Then, the machinery cost can finally be determined as follows:

(For one set)		(For three sets)
pump	$19,000 T.T.	$57,000 T.T.
motor	$ 2,000 T.T.	$ 6,000 T.T.
pipes	$ 8,000 T.T.	$24,000 T.T.
	$29,000 T.T.	$87,000 T.T.

2) Transportation cost

Conditions

i)	Distance (P.O.S. - Oropouche):	45 miles
ii)	Weight (for one set)	
	pump	4.0 t
	motor	1.1 t
	pipes	5.0 t
	Total	10.1 t
iii)	No. and scale of trucks	2 - 6t trucks for one set
iv)	Necwssary time for loading and unloading	4 hours
v)	Necessary time for driving the truck	4 hrs for going and returning.

2)-1 Wages (for one truck)

driver	1 person x $8.43 = $8.43
lorry loaders	3 persons x $7.40 = $22.20
Total	30.63

2)-2 Fuel (for one truck)

diesel fuel	4.5 l/hr x 4 hr	= 18 l	= 4.0 gals	
engine oil	0.3 l/hr x 4 hr	= 1.2 l	= 0.3 gals	
gear oil	0.015 l/hr x 4 hr	= 0.06 l	= 0.02 gals	
grease	0.05 kg/hr x 4 hr	= 0.2 kg	= 0.5 lb	
Diesel fuel		4.0 gals x 47¢ /gal		= $1.88
engine oil		0.3 gals x $1.14/gal		= $0.34
gear oil		0.02 gals x $1.47/gal		= $0.03
grease		0.5 lb x 45¢ /lb		= $0.23
			Total	$2.48

2)-3 Cost for maintenance and depreciation (for one truck) 6t-truck

i)	New price (Show window price	$9,095
	(Data are (Wooden body	$ 250
	from Neal (Purchase Tax (10%)	$909.50
	& Massy) (Other fees	$330
	Total	$10,494.50 ≑ $11,000

ii)	Durable time	5,000 hrs	
iii)	Daily adjustment ratio	0.40)	
iv)	Periodical adjustment ratio	0.80)	total 2.10
v)	Total depreciation ratio	0.90)	

Total cost for adjustment and depreciation per hour

$$\$11,000 \times \frac{2.10}{5,000\ \text{hrs}} = \$4.62/\text{hr}$$

Therefore, for 4 hours,

4 hours x $4.62/hr = $18.48

2)-4 Sum of the transportation cost for one truck

i)	Wages	$30.63
ii)	Fuel	$ 2.48
iii)	Maintenance and depreciation	$18.48
	Total	$51.59

2)-5 Sum of the transportation costs for three sets of pumps

2 trucks for one set of pump

Therefore 6 trucks for three sets of pumps

6 trucks x $51.69 = $309.54 ≑ $300

3) Installation cost

3)-1 For one set of pump

electrican	1 person x 2 days x $8.95 = $17.90	
crane operator	1 person x 2 days x $8.43 = $16.86	
mechanics	2 persons x 2 days x $8.95 =$35.80	
skill labourers	4 persons x 2 days x $7.93 =$63.44	
		$ 134.00
waste and grease		$ 4.00
	Total	$ 138.00

A ceiling crane must be used to install the pump.

3)-2 For one set of pipes

crane operator	1 person x 2 days x $8.43 = $16.86	
pipe fitter	1 person x 2 days x $8.95 = $17.90	
skill labourers	4 persons x 2 days x $7.93 =$63.44	
		$98.20
Waste and grease		$ 2.80
	Total	$101.-

Although the ceiling crane can be used in the house, a crane must be used on the outside of the house to set the pipes.

As the time to use the crane is very short, the cost for fuel, maintenance and depreciation of the crane can be omitted.

3)-3 Sum of the installation costs for three sets of pumps and pipes

$3 \times (\$138 + \$101) = \$717 \doteqdot \$700$

4) Transmission of electricity

Judging from the letter from M.R. Ramjohn, Area Superintendent, T & TEC, dated 19, June 1968, and the information received from Mr. Massiah, Electrical Engineer, T & TEC on October 1968, the best way for transmission will be to transmit 12KV from the nearest existing transmission line to the pumping station and stepping down from 12KV to 400V at the station by a suitable transformer.

In this case, the transformer and necessary electric meters will be provided by T & TEC, although rent for the transformer must be paid.

4)-1 Cost for transmission line

Distance of transmission line is about 0.7 mile i.e. from Suchit trace to the pumping station site at the mouth of Blackwater channel.

Unit cost for construction: $13,000/mile

Therefore, 0.7 mile x $13,000/mile = $9,100

4)-2 Cost for substation

Assuming that a cost for substation is about 10% of the cost for transmission line, it will be,

$1/10 \times \$9,100 \doteqdot \$900$

Total $10,000

(Note)

i) Total of the maximum load will be,
3 pumps x 60kW = 180kW
as the maximum load by one pump is regarded as 60kW
Therefore, the capacity of the transformer can be determined to be 200 K.V.A.

ii) Scale of the transmission line:
4/0 ..... (A.W.G. 0.739" dia x 7 strands ACSR) must be used for the purpose.

iii) Amperage:
the maximum amperage expected will be

$$\frac{180\text{kW}}{12\text{KV}} = 15.0\text{ A}$$

5) Building cost of pump house

According to the instruction given by Mr. B. Hamilton an architect, the building cost of a pump house can be estimated by (1) the unit cost of the reinforced concrete using form work. (2) the unit cost of the reinforced concrete using from work. (2) the unit cost of the wall (3) profits of the contractor which will be 15% of the total of the above (1 + 2).

5)-1 (1) Unit cost of the reinforced concrete using form work:

This unit cost consists of three parts, that is, cost of concrete, cost of reinforcement and cost of form work, which are always nearly equal.

As the unit cost of concrete itself is $32.00 T.T/cu.yd in average, the unit cost of the reinforced concrete using form work will be

3 x $32.00 T.T/cu.yd of concrete volume = $96.00 T.T/cu.yd of concrete volume.

(2) Unit cost of wall:

Plan 6" concrete block will be $8.00 T.T/sq.yd and plain 4" concrete block will be $4.70 T.T/sq.yd.

Unit cost of building (building area method)

As another standard, the unit cost for the building area of such a pump house is about $10.00 T.T./sq.ft of floor space.

In accordance with the above instruction, the building cost of the pump house will be estimated as follows.

5)-3 (1) Calculation for concrete volume of the reinforced concrete:
(cf. Figs. in the report IV and Fig. 2 in this report)

i) Slab 52.5' x 36.0' x 0.4' = 756.0 cu.ft

ii) Beam (Main 1.2' x 2.0' x (2 x 46.8' + 5 x 29.3') = 576.2 cu.ft.
(Sub 0.8' x 1.3' x 46.8' = 48.7 cu.ft.

iii) Column (2.7' x 1.7' x 23.0' x 10 = 1055.7 cu.ft.
(1.7' x 1.7' x 23.0' x 2 = 132.9 cu.ft.

Total 2569.5 cu.ft = 95.2 cu.yds.

(2) Calculation for wall area:
(100' x 23.0' x 8) + (12.5' x 23.0' x 4) x 60% = 1,794 sq.ft = 199.3 sq.yds

(3) Calculation of the total cost:

i) Reinforced concrete using form work
95.2 cu.yds x $96.00/cu.yd = $9,139

ii) Wall (made of plain 6" concrete block)
199.3 sq.yds x $8.00/sq.yd = $1,594

Total = $10,733 (cost based on labor and materials)

iii) 15% of above $1,610 (profit of contractor)
grand total $12,343

5)-4 Calculation in which the unit cost in 5-2) is used:

46.8' x 29.3' x $10.00/sq.ft = $13,712

5)-5 Determination of the building cost

As a result of the calculations in 5)-3 and 5)-4, the building cost can be determined to be $14,000T.T.

6) Total of the initial costs

i)	Civil work (cf. the report IV)		$ 93,800 T.T.
ii)	Machinery		$ 87,000
iii)	Transportation		$ 300
iv)	Installation		$ 700
v)	Transmission of electricity		$ 10,000
vi)	Pump house		$ 14,000
		Grand Total	$205,800 T.T.

7) Running cost for pumping operation

Assuming that each of three pumps is operated averagely for 100 hours a month.

7)-1 Wages (for one pump)

pump operator	1/3 x 1 person x 100/8 hours x $8.95	= $37.29
labourer	1/3 x 1 person x 100/8 hours x $7.35	= $30.63
electrician	1/3 x 0.5 x 100/8 x $9.43	= $19.65
	Total	= $87.57 = $0.88/hr.

7)-2 Electricity tariff (for one pump) (Large load, Rate D2)

i) Standing charge

1/3 x 200 K.V.A. x $7.15/K.V.A. = $476.67

ii) Rent for transformer (from T & TEC) (for one pump)

1/3 x $60 = $20

Total $1,216.67 = $12.17/hr

7)-3 Materials (for one pump)

2% of the 7)-2

$1,216.67 x 2/100 = $24.33 = $0.24/hr

Grand total $13.29/on pump hr.

(Note) Rent for transformer is based on the information from T & TEC
That is, \$60/month for 200 K.V.A.
\$72/month for 250 K.V.A.

8) Depreciation cost for pumping facilities (for one pump)

New price of pump and pipes:	\$19,000 + \$8,000 = \$27,000
New price of motor:	\$ 2,000
Durable time of pump and pipes:	8,000 hrs
Durable time of motor:	15,000 hrs
Ratio of depreciation:	0.9

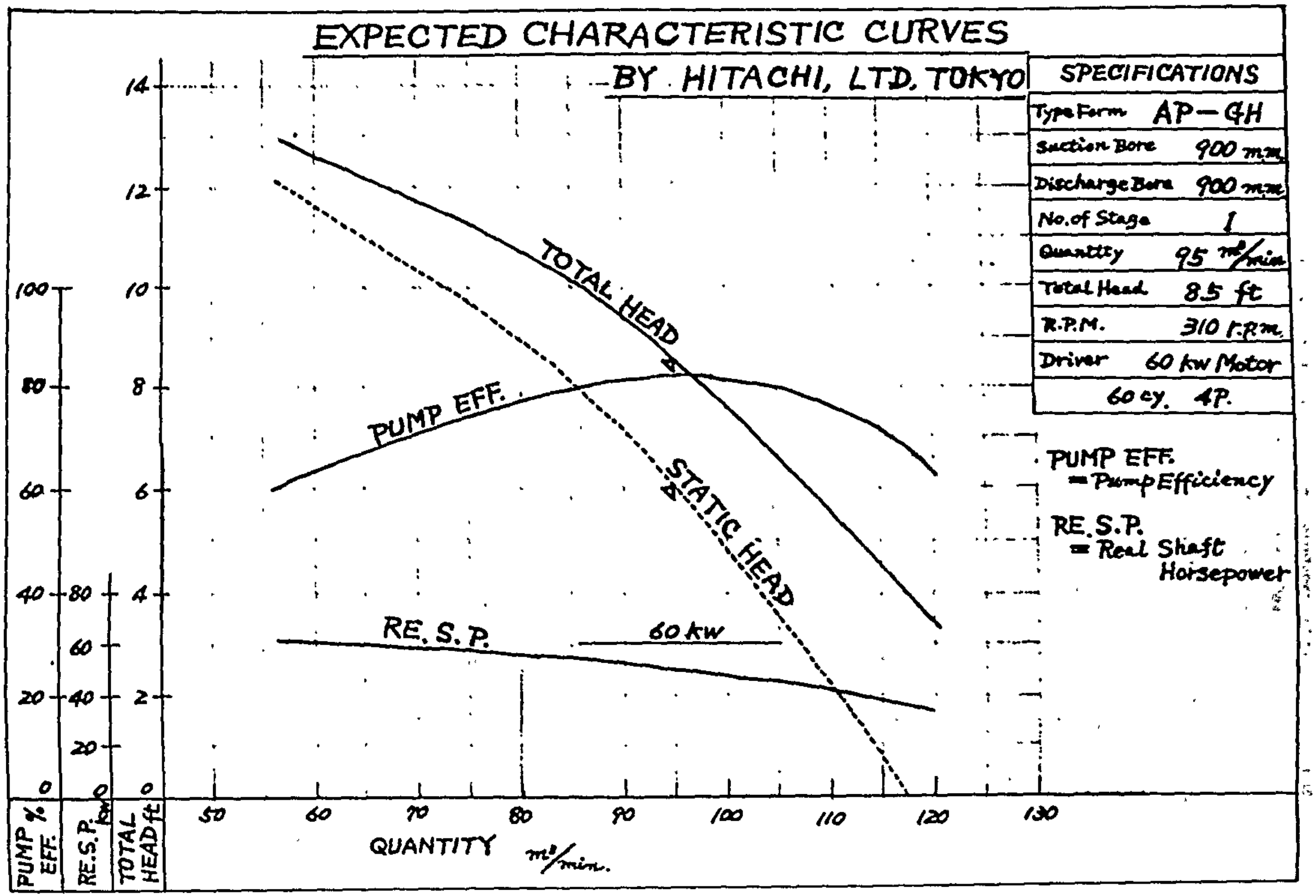
Therefore, depreciation cost will be,

for pump pipes	\$27,000 x 0.9/8,000 hrs = \$3.04/hr
for motor	\$ 2,000 x 0.9/15,000 hrs = \$0.16/hr
Total	\$3.20/hr.

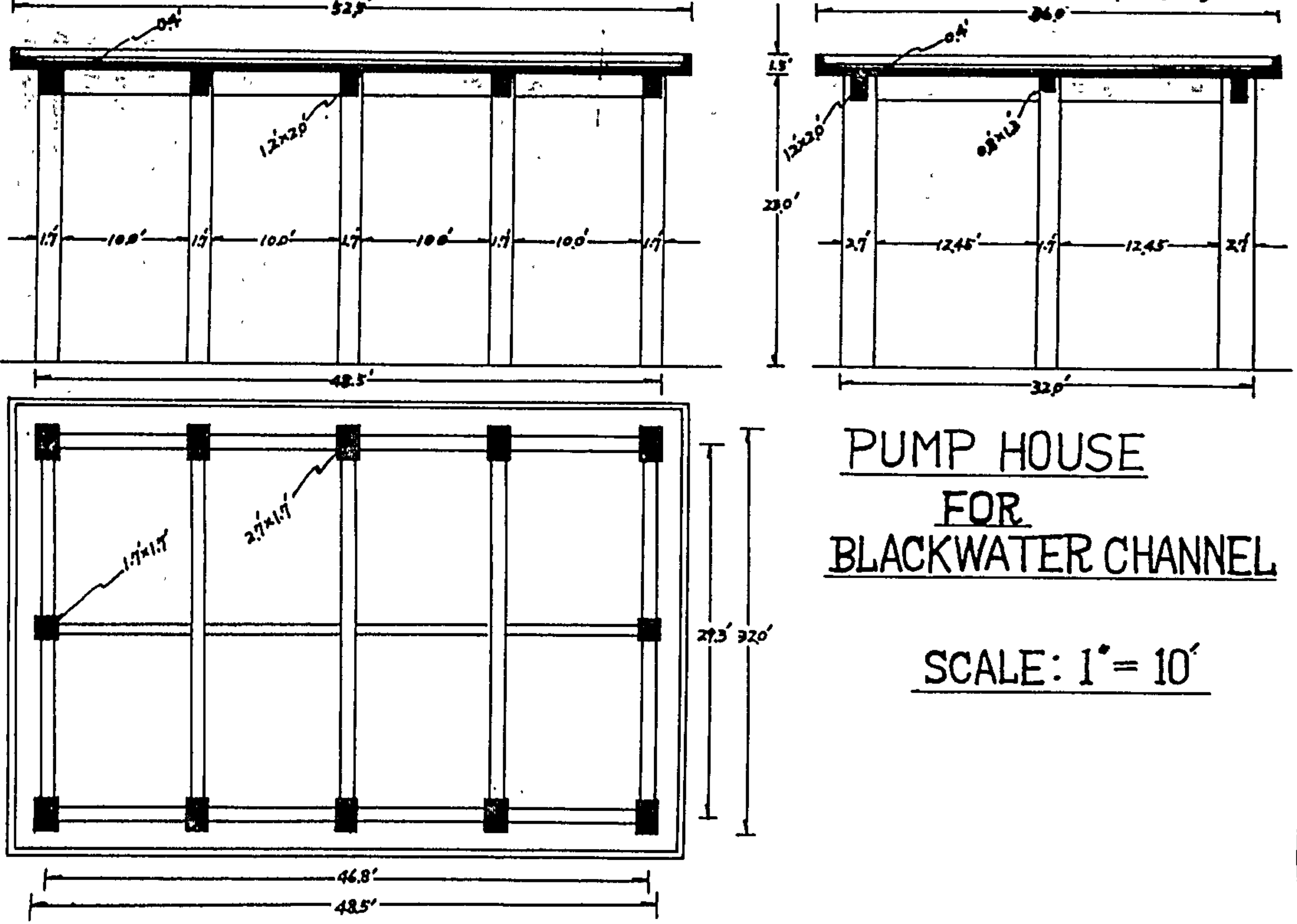
Junichi Kitamura

(Fig.1)

FOR BLACKWATER CHANNEL IN OROPOUCHE



(Fig.2)



PUMP HOUSE

FOR

BLACKWATER CHANNEL

SCALE: 1" = 10'

(2) DE BOULET VALLEY (I) November, 1967

MOVEMENT

15, Nov. Visited the area with Mr. Riley and was explained the prevailing condition by Mr. M. Antonio, Superintendent (South)

23, Nov. Visited there again and investigated the catchment area and the course of the river from the point near Iere High School to the confluence of this river.

DESCRIPTION OF THE AREA STUDIED

De Boulet Valley is situated in the uppermost reaches of the Morocupano River. San Fernando - Siparia-Erin Road goes thorugh east and west as the water-parting between this river in the north and the Coora River in the South.

The Morocupano River runs into the Oropouche Lagoon and courses down to the St John's Sluice.

The catchment area of De Boulet Valley is therefore limited in the south by the S.S. Erin Road.

The catchment area as far as the point of the confluence is estimated at 46 acres (7000 ft by 2000 ft).

And the catchment area as far as the point at the back of the High School, the area most affected, is nearly 5 acres (500 ft by 400 ft).

According to the plan, the river course goes north from Siparia-Erin Road and at a point of about 400 ft from the road turns to the west and goes, about 200 ft further. It curves to the north again and flows down the east side of the high school. It then reaches level land after flowing down about 1500 ft more.

The elevation is about 200 ft at the road and is 90 ft at the point 1300 ft away.

Therefore, the grade will be $\frac{200 - 90}{1,300} = \frac{1}{12}$ and it is found to be very steep.

Geologically, this area consists of red weathered sandy shale, which easily crumbles to red fine sand when squeezed by hand. This stratum easily collapses when squeezed by hand. This stratum easily collapses when saturated with moisture and subjected to water pressure.

The normal discharge mainly comes from the residential section along the raod and is very little The flood discharge, however quickly concentrates in valley from the catchment area which is however small. Consequently the concentrated water erodes both sides of the valley and conveys their soil to its lower reaches. The lower reaches thus became a sandy plain. This action will be continuously repeated and the valley will eventually become wider and deeper. As it is still very steep on both sides, it can be anticipated to continue collapsing until stable slopes are attained.

As to th four basket works which had been carried out as cross walls, the most upper one is still of use to keep the soil. The other ones, however, were destroyed by the large earth pressure. The pre-cast concrete drain which had been put down at the upper reaches, are in good condition, although the basic soil under the end of the drain was scoured by flood discharge.

ITEMS TO BE INVESTIGATED

1) Calculation of design flood

   a) Investigation to confirm the catchment area
   b) Drawing of the plan of the catchment area
   c) Collection of rainfall data
   d) Survey of the trace of the maximum flood stage

2) Preparing the detailed plan, profile and cross section of the river.

3) Investigation of the damage of the works which were carried out so far and study of their cause.

COUNTERMEASURE

1) Drainage Works

   a) Determination of the drain section at each point
   b) Determination of the longitudinal slope of the drain
   c) Study of the drop works - number, scale, materials and method of construction.

2) Slope-protection works

   a) Study of the most economical method by consideration of the conservation of the land along the valley.
      (Methods which are being considered)
      i) Filling up the valley with earth from both sides or/and from a borrow-pit. Calculation of the earth requirement. Study of the kind of earth. Location of suitable borrow-pit.
      ii) Cutting both sides to get a stable slope and paving with stone and cement-mortar. Measuring the area of lost land along the valley.
      iii) Construction of retaining wall with reinforced concrete. Stability analyses of the wall which must be done for overturning, sliding and bearing pressure of the basic soil.
      iv) Compromise method from the above.

3) Construction of adequate catchd-drains in some suitable place so as to control the flood into the valley and lead it to lower reaches.

Junichi Kitamura

DE BOULET VALLEY (II) 20th, March, 1968

MOVEMENT

7, Mar. Visited there to investigate the present condition and proposed borrow-pits on both sides.
Discussed the most economical and excellent countermeasure against erosion with Mr. M. Antonio.

DESCRIPTION OF THE AREA STUDIED AFTER THE PREVIOUS VISIT

Three months has passed since the previous visit. The present condition in De Doulet Valley has apparently been aggravated. The paths on both sides can no longer be seen and a fence nears the lere High School has partly fallen down. Moreover a part of the precast concrete channel on the upper left side is now in the air as the soil under the channel has been eroded completely.

This shows that erosion is still going on in this valley. The most urgent problem is of course the erosion on the side with the high school. This should be checked as soon as possible.

All the land along the left side is possessed by the Government and so the area just down from the ground of the high school can easily be used as a borrow-pit. This area which is next to the school ground cannot be cut deeply owing to the proximity of the school gorund. Earth quantity supplied from the area to fill up the valley will be therefore very small.

On the other hand the land along the right side is higher than the left side. If the earth of these primate lands can be used by cutting them down to allowable level the requirement to fill up the valley will be supplied to some extent. As some of the lands are scheduled as building lots, consideration for the purpose will be necessary.

The anchors of supporting wires for a tall antenna in the land must be moved. An existing private road may also need changing.

INTRODUCTION

The survey to arrange a detailed plan, profile and cross sections in the valley is being carried out. However, urgent works are needed before the rainy season starts. Thus there is no time to design in detail. One cannot help but to roughly examine the best method.

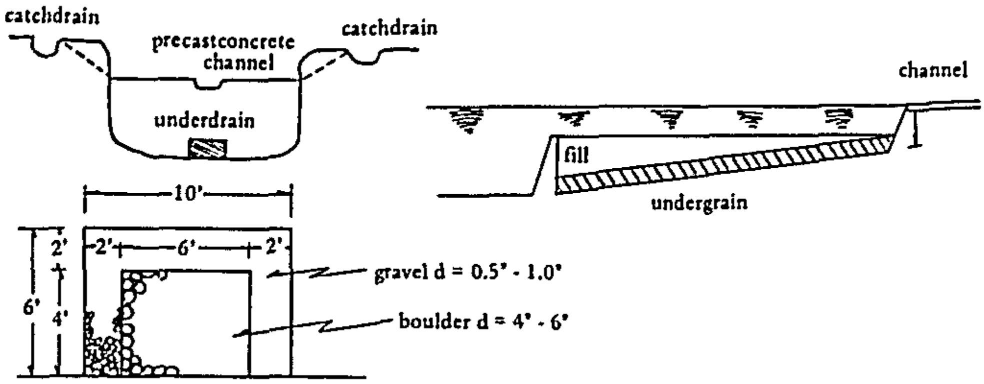
See methods in first report.

METHOD

1. Drainage Works

It is recommended that the water from the upper reaches is mainly discharged with underdrains made of boulders and gravel. Because the underdrain can function not only by-pass channel during filling works but also a collecting channel which carries seepage flow from both sides.

The dimensions are shown on the following sketch:



On the other hand, after filling, precast concrete channels are necessary for discharging surface runoff from both slopes and for leading water from both sides to lower reaches as catch drain.

2. Filling Works

It is most economical to fill the valley by buldozing the earth from both sides. Transportation of earth far from the valley by dump-truck will be very expensive. Underdrain-works must be carried out carefully by man power. Using a bulldozer is convenient to consolidate earth in this case. The ownerer's consents to use their lands as borrow-pits should be sought as early as possible.

3. Slope-protection Works

Upper parts of both sides will need the slopes which have stable grades even after filling up the valley, because the filling cannot be carried out up to the top of the sides owing to shortage of earth. On this slope appropriate vegetation is necessary for protecting the surface from erosion. Along the high school side hwich has no room to make slopea retaining walls may be needed. Anchored steel sheet piles can be substituted for the retaining walls as the case may be.

4. Drop Works

At a minimum two drop works are necessary, that is at the end of the present concrete channel, and at the end of the filling work. A cross wall must be built with concrete placing or rubble masonry. Weep holes which can discharge water sufficiently must be made carefully.

Junichi Kitamura

DE BOULET VALLEY (III) September, 1968

MOVEMENT

13, Sept. Visited De Boulet Valley with Mr. C. Taylor, Director of Drainage Division, Mr. W. Riley and Mr. C. Antoine to inspect the works undertaken to divert drains which flow into this river.

OBJECT OF THIS REPORT

The works undertaken this time, are intended to divert three drains which flow into the upper reaches of the river and to maintain the river temporarily - aligning the river-bed, taking out boulders which had been used for Gabion baskets (which were broken by floodwater) and backfilling the spots eroded by flood water.

At the next stage, filling the river itself must be carried out.

For the purpose, the crown land along the river west of the playground of the Iere High School was assumed as a borrow-pit.

The earth volume possible to be supplied from the borrow-pit and earth requirement for filling the river will be calculated in this report.

CALCULATION FOR EARTH VOLUME POSSIBLE TO BE SUPPLIED FROM THE BORROW-PIT

(Case I)

Conditions: As the river-bed level at section No. 7 (the left side of which is the highest part of the borrow-pit) is 130 ft, the borrow-pit surface after cutting is determined to be flat and to be 140 ft in elevation.

No. of cross section of borrow-pit	Distance	Cross sectional area	Mean cross sectional area	Earth volume	Remarks
(-) 2B	0 ft	0 ft^2	0 ft^2	0 ft^3	(cf. plan and cross
(-) 1B	100	480	240	24,000	sections)
0B	50	855	668	33,400	
1B	50	2,460	1,658	82,900	
2B	50	2,330	2,395	119,750	
3B	50	1,685	2,008	100,400	
4B	50	805			
		0	1,245	62,250	
5B	50	0	0	0	
6B	50	0	0	0	
7B	50	0	0	0	
8B	50	0	0	0	
9B	50	0	0	0	
Total	600			422,700ft^3	=15,656yd^3=11,970m^3

(Case II)

Conditions: As the river-bed level at section No. 7 (the left side of which is the highest part of the borrow-pit,) is 130 ft, the borrow-pit surface after cutting is determined to be flat and to be 130 ft in elevation.

No. of cross section of borrow-pit	Distance	Cross sectional area	Mean cross sectional area	Earth volume	Remarks
(-) 2B	0ft	0ft^2	0ft^2	0ft^3	(cf. plan and cross sections)
(-) 1B	100	1,880	940	94,000	
0B	50	2,595	2,238	111,900	
1B	50	4,770	3,683	184,150	
2B	50	5,165	4,968	248,400	
3B	50	4,150	4,658	232,900	
4B	50	2,620			
		0	3,385	169,250	
5B	50	30	15	750	
6B	50	250	140	7,000	
7B	50	670	460	23,000	
8B	50	370	520	26,000	
9B	50	150	260	13,000	
Total	600			1,110,350ft^3	=41,124yd^3=31,442m^3

In regard to the geological condition in the borrow-pit, judging from the both sides of the river, the top soil mostly consists of the weathered reddish brown silt-sand shale, which cakes as it dries, and easily crumbles as it contains water.

According to the planned cross sections, the maximum cutting height is about 30 ft.

It is through that soils within this 30 ft, however, are the same as mentioned above.

In this calculation for the borrow-pit there is no consideration of the pipelines to be set in diversion of the drains.

Therefore consideration will have to be given for the resetting of the pipes or not using the sections (in which the pipe-lines are to be set) as the borrow-pit.

Calculation for earth requirement for filling De Boulet Valley:

(Case I)	River-bed level at section No. 3 - 22.5 ft	.... 163 ft
	River-bed level at section No. 13	.... 83 ft
	Difference	80 ft

Assuming that a drop is 10 ft high, eight drops will be needed. It is determined that these drops are set up at the following sections:-

	No. 5 + 50
	No. 6
	No. 7
	No. 10
	No. 11
	No. 12
	No. 12 + 50
	No. 13
Total	8 sections

The earth volume required for filling De Boulet Valley will be calculated due to the above assumption, as case I

Calculation for case I

No. of cross section of borrow-pit	Distance	Cross sectional area	Mean cross sectional area	Earth volume	Remarks
No. 3-22.5	0 ft	0ft^2	0ft^2	0ft^3	(cf Longitudinal and
No. 3	22.5	101	51	1,148	cross sections)
No. 4	42.5	139	120	5,100	
No. 5	58	332	236	13,688	
No. 6	87	646			
		153	489	42,543	
No. 7	70	585			
		137	369	25,830	
No. 8	32	420	269	8,928	
No. 9	107	735	578	61,846	
No. 10	90	1,333			
		766	1,034	93,060	
No. 11	65	1,038			
		461	902	58,630	
No. 12	90	1,068			
		436	752	67,680	
No. 13	105	283	360	37,800	
		0			
No. 14	100	0	0	0	
				416,253ft^3	=15,417yd^3=11,787m^3

(Case II)

In addition to case I, filling must be considered so as to give slopes of 1:2 to both sides of the river.

According to this assumption, the calculation will be carried out on the following table.

In this case, the surface level of borrow-pit after cutting is assumed as 130ft.

Calculation for Case II

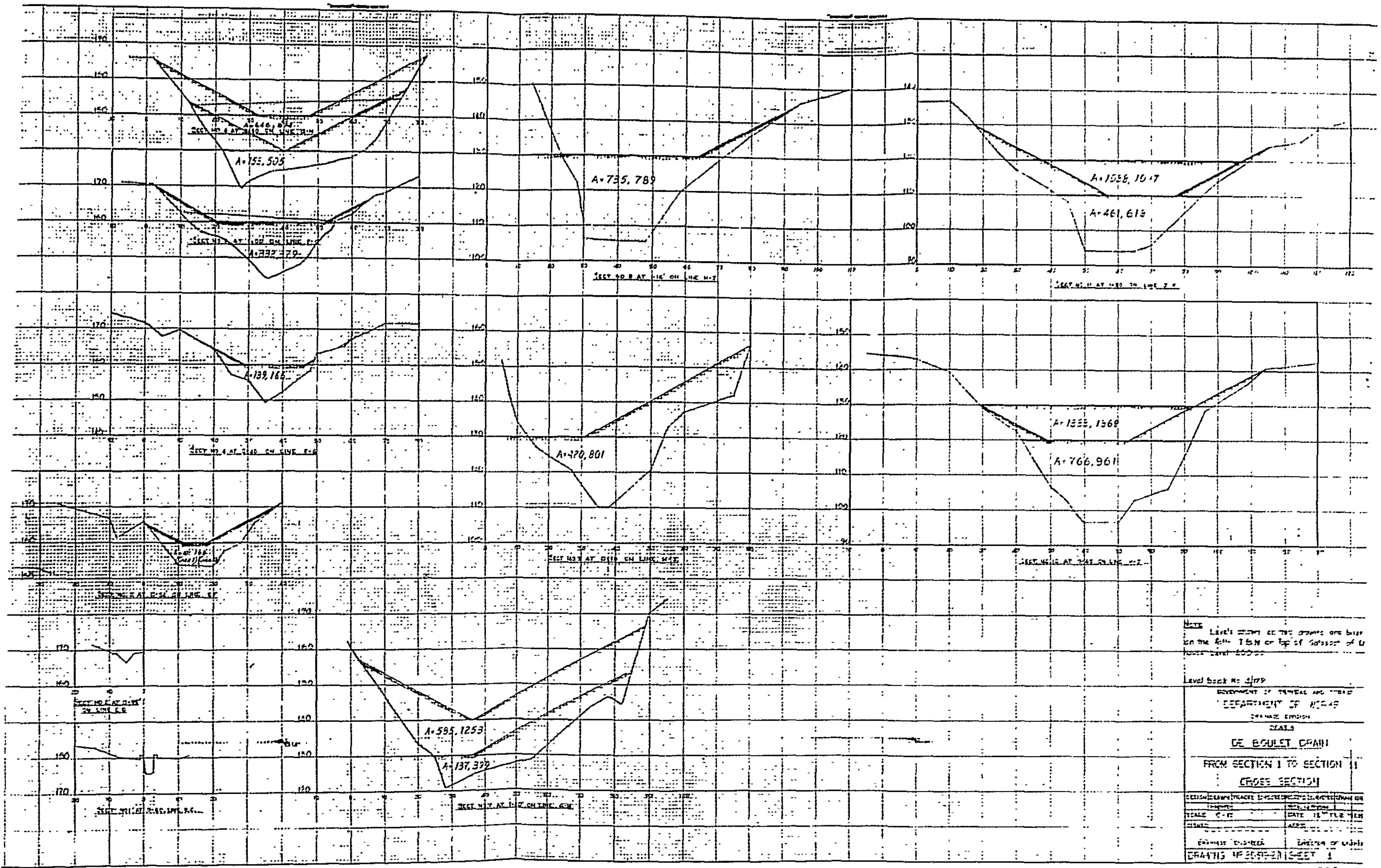
No. of cross section of borrow-pit	Distance	Cross sectional area	Mean cross sectional area	Earth volume	Remarks
No. 3-22.5	0ft	$0ft^2$	$0ft^2$	$0ft^3$	(cf. Longitudinal and cross sections)
No. 3	22.5	163	82	1,845	
No. 4	42.5	166	165	7,013	
No. 5	58	379	273	15,834	
No. 6	87	974			
		505	677	58,899	
No. 7	70	1,253			
		399	879	61,530	
No. 8	32	801	600	19,200	
No. 9	107	789	795	85,065	
No. 10	90	1,368			
		961	1,079	97,110	
No. 11	65	1,097			
		613	1,029	66,885	
No. 12	90	1,105			
		698	859	77,310	
No. 13	105	285			
		0	492	51,660	
No. 14	100	0	0	0	
				$542,351ft^3$	$=20,087yd^3=15,358m^3$

Conclusion:

If the case I is adopted to determine earth requirement for filling De Boulet Valley, the surface level of the borrow-pit after cutting can be 140ft.

If the case II is adopted to determine the whole earth requirement for filling the river, the surface level of the borrow-pit after cutting must be 2-3 ft (=137-138ft) lower than in case I.

Junichi Kitamura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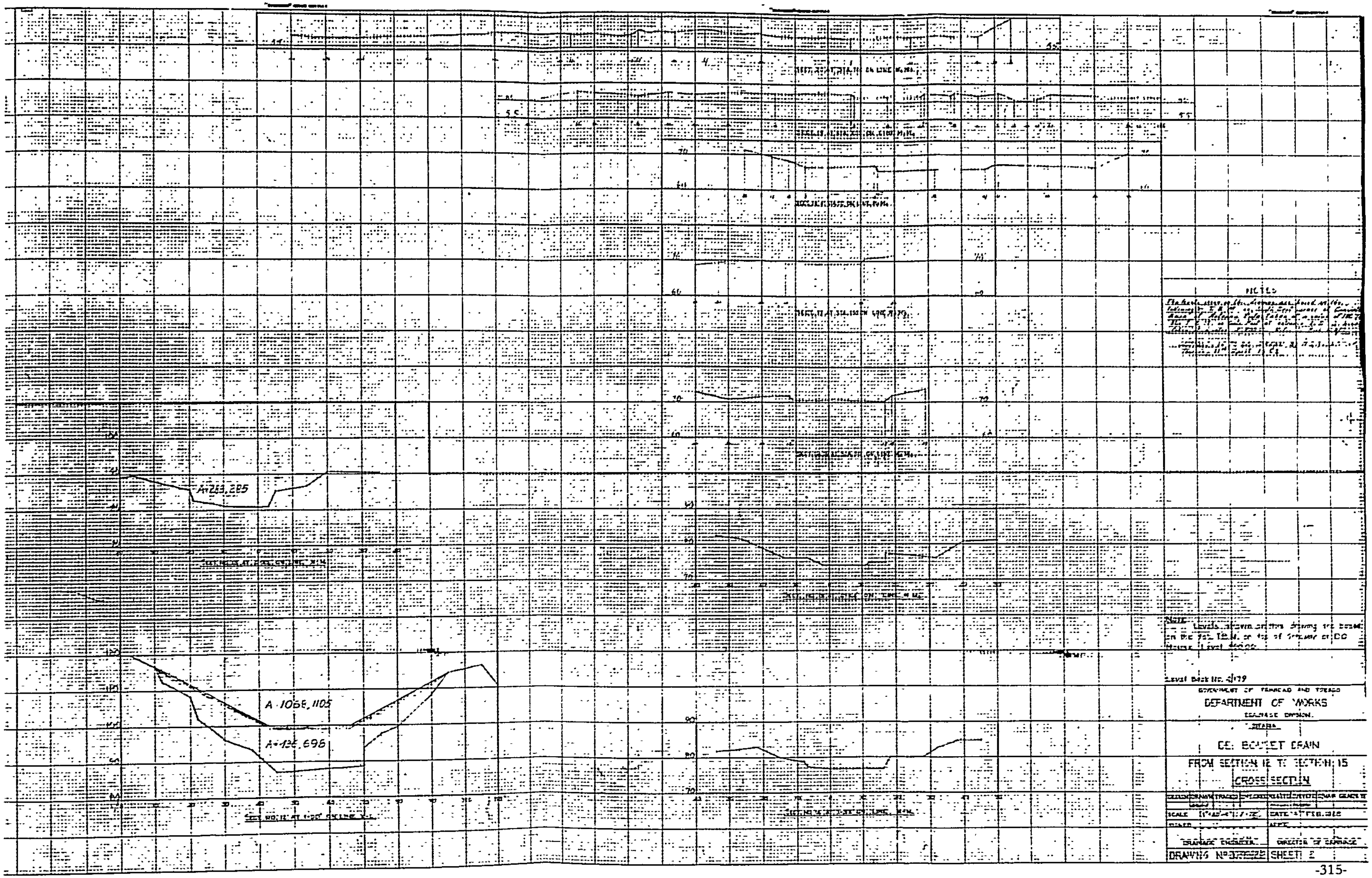
CROSS SECTION
SHEET 1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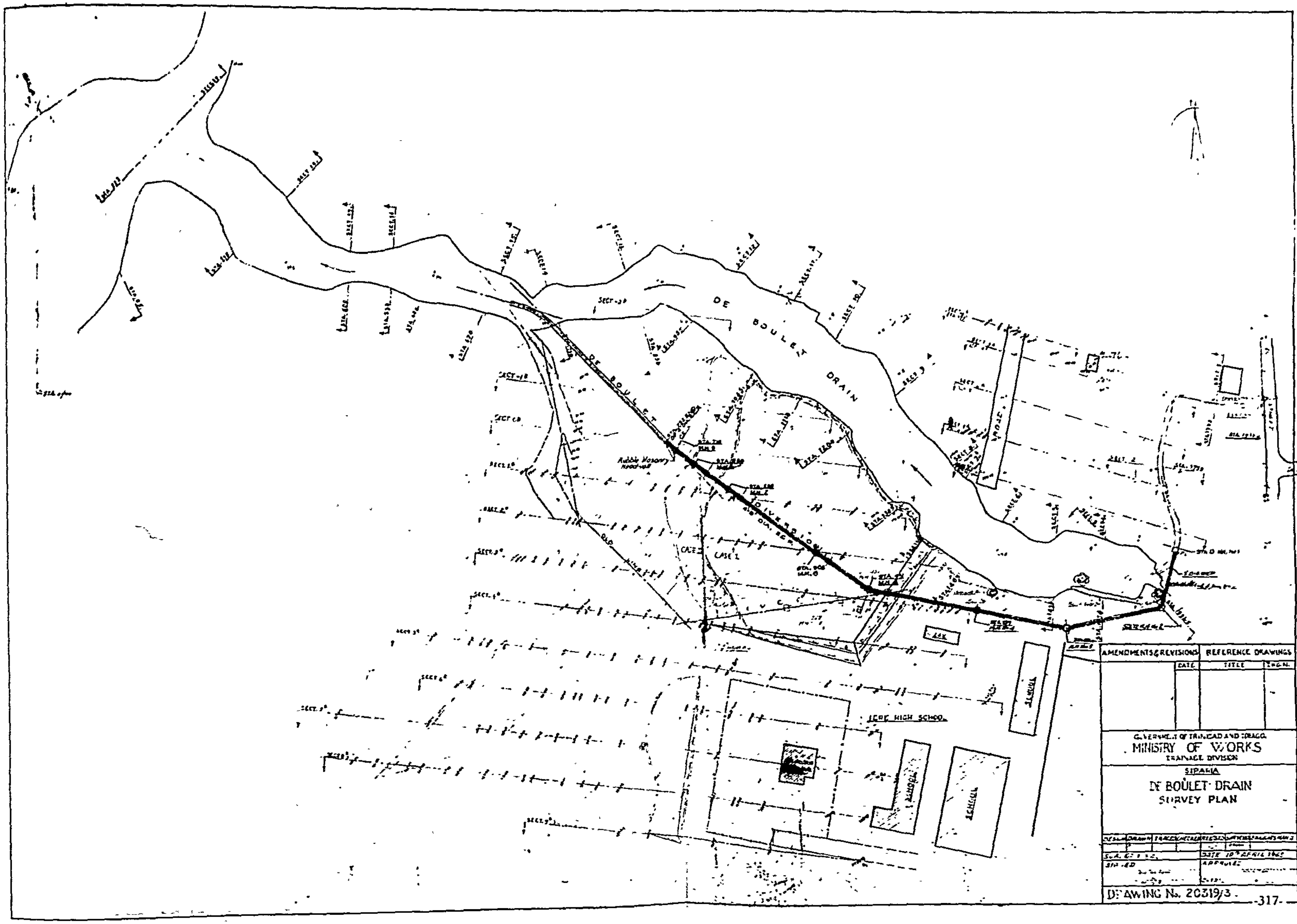
CROSS SECTION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DE BOULET DIVERSION
Rubble Masonry Headwall
CASE 2
CASE 1
OLD
SCHOOL
HIGH SCHOOL
AMENDMENTS & REVISIONS
DATE
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TITLE
MINISTRY OF WORKS
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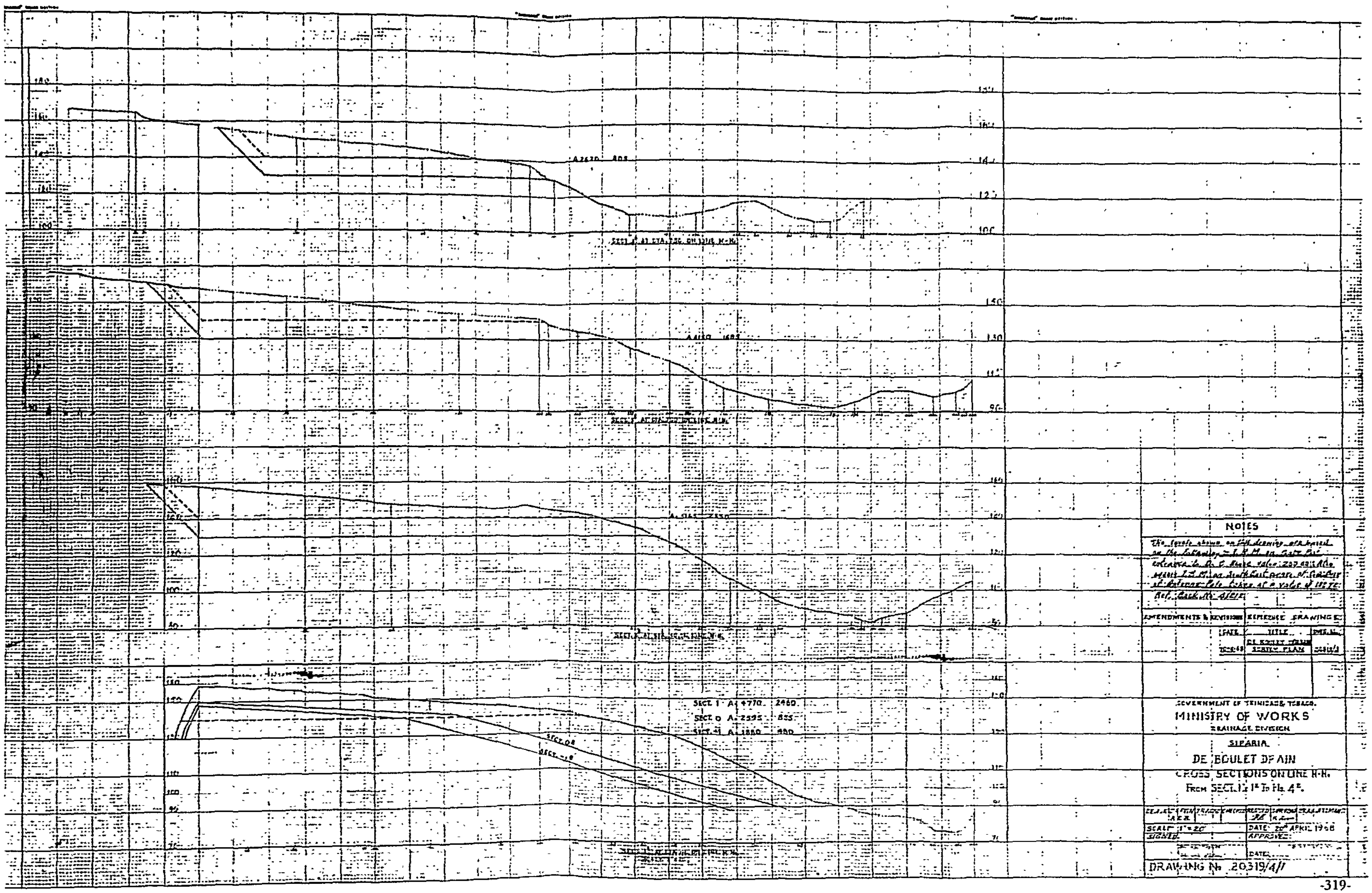

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SECT. 0 · A. 2595 · 835
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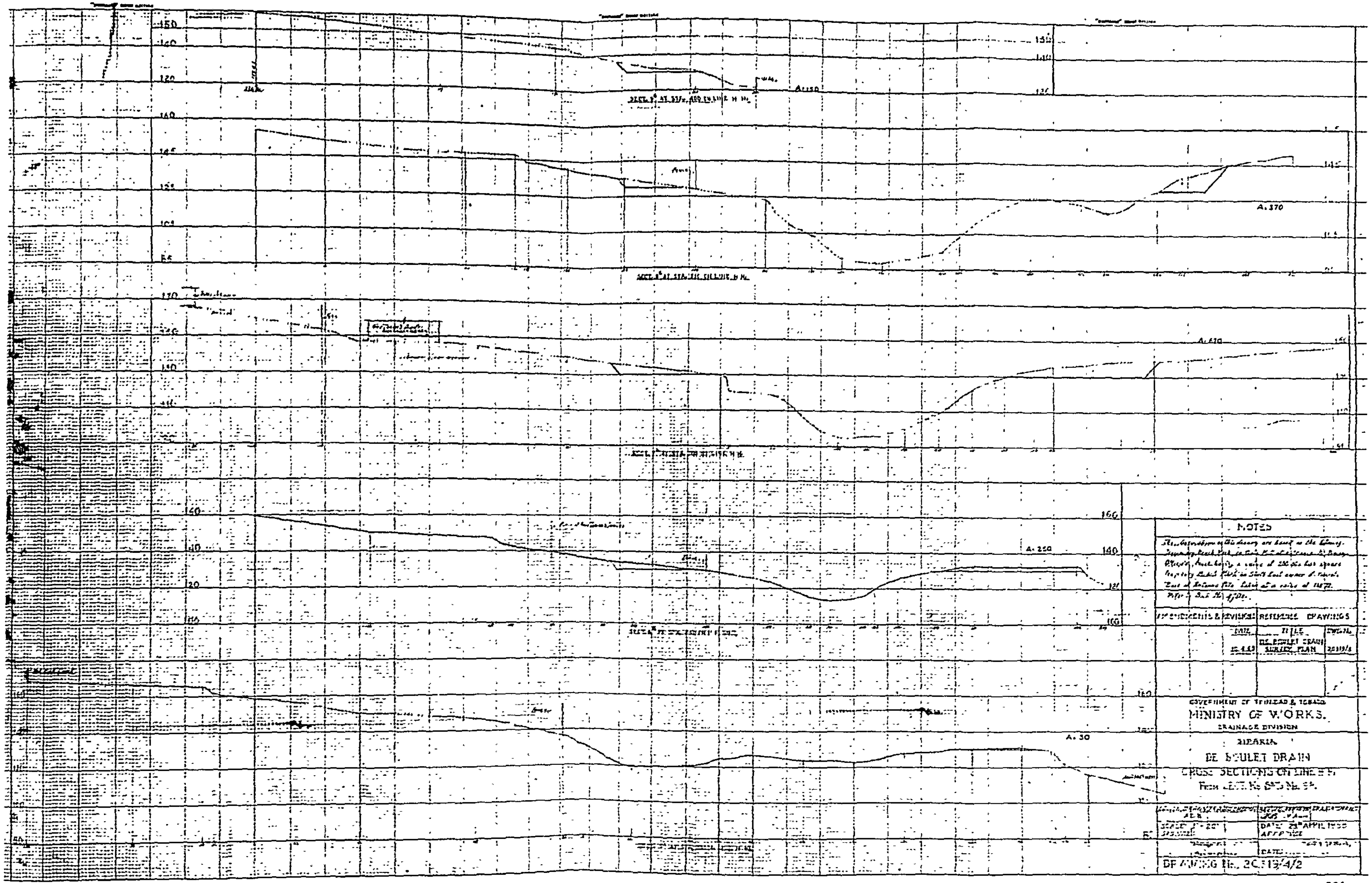
MINISTRY OF WORKS
DRAINAGE DIVISION
SIPARIA
DE BOULET DRAIN
CROSS SECTIONS ON LINE H-H.
From SECT. 1 To 4
SCALE 1" = 20'
SIGNED.
DATE 20th APRIL 1968
APPROVED:
DATE
DRAWING No. 20319/4/1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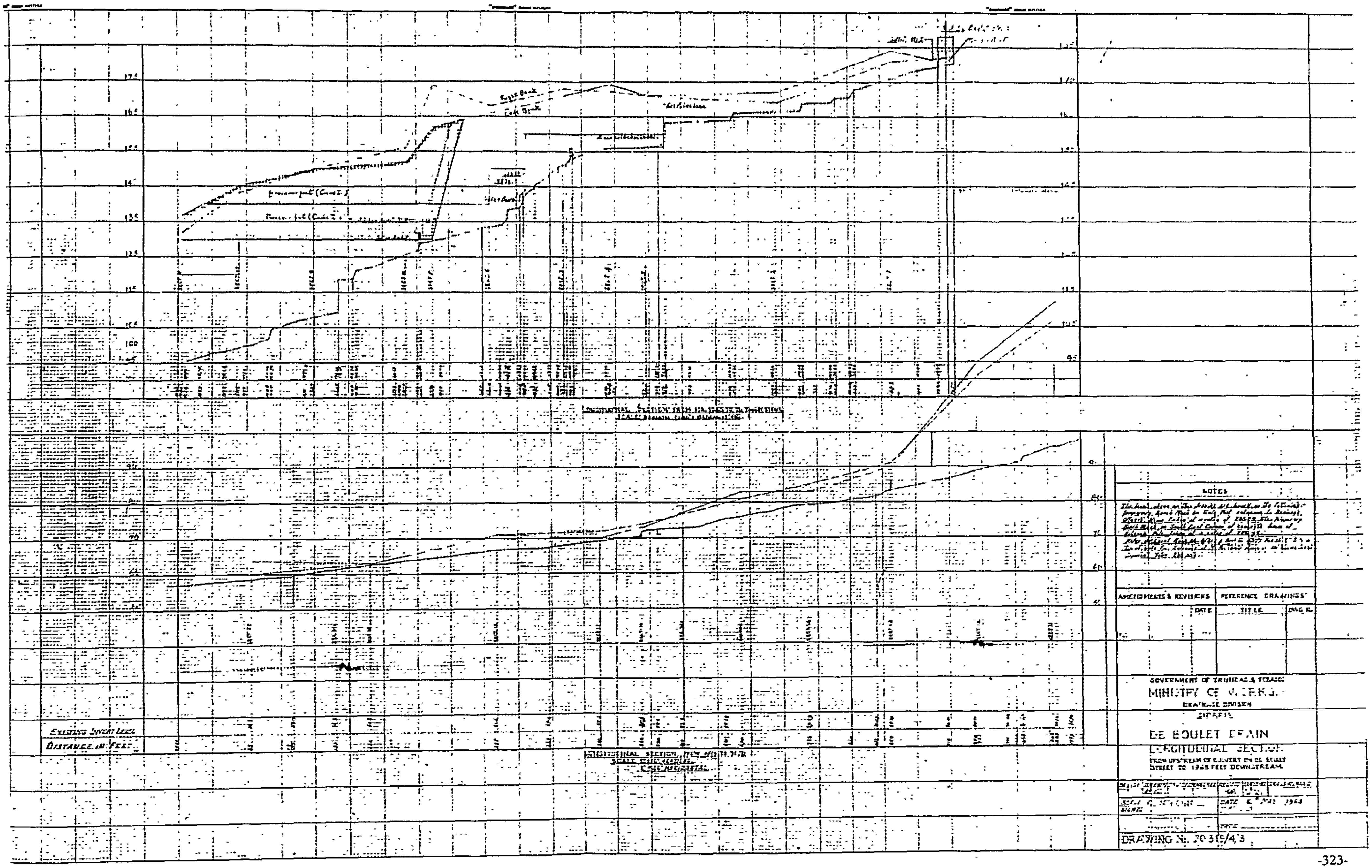

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REFERENCE DRAWINGS
DATE
TITLE
DRAINAGE DIVISION
DE BOULET DRAIN
STREET TO 1965 FEET DOWNSTREAM.
DATE

(3) LA HORQUETTE VALLEY November, 1967

MOVEMENT

10, Oct. Visited area with the Minister, Permanent Secretary, Mr. C. Taylor and others.

13, Nov. Visited area again to see the prevailing condition of the river with Mr. L. Grant, and was told about the damage caused by the overflow and silting up.

DESCRIPTION OF THE RIVER STUDIED

Paving was carried out 1,153.55 ft down from Bailey Bridge the preceding year. At time of visit sand and gravel extended to midway of the pavement. La Horquette Road to Bailey Bridge was covered with sand and gravel in spite of its having been paved with pitch. The investigation of the Cumana River was carried out from Bailey Bridge to the confluence of two main tributaries. The river bed was elevated somewhat, because of the sand and gravel mixed with boulders (1ft - 2ft in diameter) has accumulated.

The slope of the river bed, however, is still too steep, (Even the slope of the paved parts was designed 1 in 50) and at the bends the river side was scoured and had collapsed during flood conditions. As a result plenty sand and gravel came onto the river bed and were gradually carried down. The obulders had surely not been brought down from the upper reach but had tumbled down from nearby river side during scouring. The sand and gravel may of course have been brought down from the upper reaches.

Due to steep slope of river bed it can be supposed that the velocity of the river flow in flood is too fast and the scouring force is therefore tremendous.

Both sides of the river are terraces, or alluvial layers consisting of gravel and coarse sand mainly. They are easily collapsed by flood water.

The gravel comprises fragments of metamorphic rocks (quartzite, black schist and the like.)

At the curves in the upper reach of the river, Crystaline Schists are exposed and stand up well against scouring.

At the hilly area on the right bank of the river, building lots already graded, were observed.

Items to be investigated

1. Surveys from which data may be obtained for the plotting of longitudinal and cross section from Bailey Bridge to points 200 yards beyond the confluence of two tributaries.

2. Calculation of flood discharge with the rainfall precipitation and the trace of flood stage.

3. Check on areas where scouring was much in evidence along the river.

4. Check on those bare areas where vegetative cover is needed.

Countermeasures:

1. Building the cross-walls as fall works, to make the river bed slope mild. And determination of the positions, number and scale.

2. Building of retaining walls or basket work to protect the banks from collapsing. And selection of the corresponding sites.

3. Removal of gravel and sand on the river bed now.

4. Extension of pavement to allow smooth discharge of the water.

5. Expansion of river section its narrow parts to discharge the design flood.

6. Coveirng the exposed land with vegetation in order to delay arrival of flood water and thus prevent concentration of same.

Immediately after survey data is obtained, detailed calculations and design will be undertaken.

Junichi Kitamura

(4) WALLER FIELD April, 1968

MOVEMENT

4, March	Received a copy of the letter requesting a drainage study of the following areas from the Ministry of Planning and Development. - Waller Field, Carlson Field and Esmeralda.
14, Mar.	Visited Mr. M. Percy at the central Experimental Station, at Centeno to obtain specific information of the areas concerned.
18, Mar.	Got plans titled 'A proposed layout of Waller Field' and 'A proposed layout of Carlson Field' from Ministry of Agriculture.
19, Mar.	Got a contour map scaled 1/10,000 with 25 foot-contour lines from the mapping Section.
22, Mar.	Visited Waller Field with Mr. J. V. Smith on tour with Mr. M. Percy and Mr. B. Reynolds a dairy expert.
28, Mar., 3, Apr, 5, Apr.	Visited Waller Field to investigate topographical condition and main rivers.

INTRODUCTION

For purpose of this study the following is quoted from the letter from the Ministry of Planning and Development 'At a recent meeting of the Crown Lands Development Programme Committee, the view was expressed that a drainage study of the project areas was most desirable particularly in view of the drainage problems existing on the farms at present. It was stressed that corrective measures should be implemented forthwith to solve the matter but that such action would be more effectively carried out if there was some master plan to follow.

As a matter of urgency, a drainage study of the entire area, that is. Waller Field; Carlsen Field; and Esmeralda in that order of priority was requested to be undertaken. Because of this the drainage study for Waller Field will be described in this report.

DESCRIPTION OF THE AREA STUDIED

The drainage areas concerned comprise a portion of the lands which had been leased to the U.S. Government and lately returned. These areas are being developed mainly for dairy farms. The acreage concerned is as follows. (by Mr. M. Percy).

Kind of farm	Block of farm	Acreage	Remarks
Dairy Farm	Extension Farm I	1,183	In the catchment area of the Guanapo R. and El Mano River
Dairy Farm	Extension Farm II	600	In the catchment area of the Aripo River
Dairy Farm	Block I	484	Developed
Dairy Farm	Block II	135	Developed
Dairy Farm	Block III	880	Underdevelopment
Dairy Farm	Block IV	261	Underdevelopment
Dairy Farm	Block V	546	Underdevelopment
Pig Farm	Block I	110	Underdevelopment
Pig Farm	Block II	100	Underdevelopment
Pilot Farm	Block I	93	Developed in 1963
Total		4,392	

'Heights of Guanapo' which comprises El Cerro Del Aripo (3,085 ft) Morne Blau (2,781 ft) and Chaguaramal (2,817 ft) is situated in the north of the catchment areas of the rivers which run through Waller Field.

The Guanapo River and the Aripo River, rise in these mountains and flow down from the North to the South.

Aripo Savanna situated in the east of the area is a watershed between the water system of the Caroni which flows down to the west and pours into the Gulf of Paria, and, the Oropouche which flows-down to the East and pours into the Atlantic Ocean.

There are hilly lands 300 - 175 ft in elevation in the north of Waller Field and mild sloped lands 175-70 ft in elevation follow them.

The dimensions of the rivers are shown on the following table.

Name of river	At the confluence			At the crossing of Eastern Main Road		
	Catchment area	Length of river	Difference of elevation	Catchment area (A)	Length of river (L)	Difference of elevation (H)
ARIPO R.	47.3 km^2 18.24miles	20.6 km 12.8miles	2,830-70ft 863-21m	26.1km^2 10.07miles	10.9km 6.8miles	2,830-145ft 863-44m
EL MAMO R.	12.3 4.74	9.5 5.9	1,410-70ft 430-21	3.7 1.43	4.7 2.9	1,410-140 430-43
GUANAPO R.	43.7 16.85	20.4 12.7	2,950-50 899-15	34.3 13.24	12.6 7.8	2,950-140 899-43
MATURITA R.	6.6 2.54	7.2 4.5	1,400-95 427-29	4.3 1.66	4.2 2.6	1,400-180 427-55
Tributary of the CARONI R.	6.0 2.32	2.7 1.7	110-70 34-21	- -	- -	- -
ARIMA R.	28.9 11.16	18.0 11.2	2,250-70 686-21	23.9 9.23	11.6 7.2	2,250-160 686-49

Four main rivers provide drainage from Waller Field - The Aripo, the El Mamo, the Guanapo and the Maturita. The El Mamo and the Maturita which are one of the tributaries of the Aripo and the Guanapo respectively, originate from low hilly lands and have small catchment areas and river-lengths.

The lower land which is surrounded by the catchment area of the Gunapo and the El Mamo is a direct catchment area of the Caroni, Water in the area is discharged into the Caroni via a small gully. Although the routes of these rivers are meandering, the slopes are still very steep. Generally these rivers are flowing down under natural conditions without dikes.

The cross sections of these rivers are shown on Fig. 3. Geological condition studied - Hilly land, in northern part of the area consists of schist and its top soil is made of the weathered sandy schist. A part of the catchment area of the Aripo River consists of sandy shale. The greater part of Waller Field, however, consists of greyish white or brownish white calcareceous silt which cakes easily as it dries and becomes muddy as it gets wet.

Rainfall precipitation - There are five rainfall stations in the vicinity as shown on Fig. 1.

However data collected is only of ten years duration.

Maximum daily rainfall every year at the stations are shown below:-

(inches)

Name of station	1958	1959	1960	1961	1962	1963	1964	1965	1966	1967
Quare Dam Hollis Reservoir	5.06	3.23	3.90	3.43	3.06	5.16	3.24	4.01	3.59	3.03
El Naranjo Aripo	4.04	3.09	4.66	4.03	2.25	3.40	3.15	3 60	3.67	3.95
Grasslands Aripo	-	-	-	-	-	-	-	-	4.07	2.84
Torrecilla Arima	2.91	2.36	2.60	3.87	3.00	3.40	2.43	2.41	3.55	3.03
El Suzan Cumuto	-	-	-	-	-	5.20	3.22	3.94	5.30	3.10

Master Plan of Drainage

1) Outline of the plan

As a designed flood, the flood which occurs once in ten years must be adopted.

Natural drainage is possible for this area, because it is situated at the upmost reaches of the Caroni River and it is a terrace 175-70 ft in elevation (above the mean sea level) as mentioned above.

Although main rivers have insufficient sections to discharge the maximum desinged flood, there will be hardly any occasion that the inflow of water from outside the area would damage the subject area. (Improvement of the rivers, that is, making them straight, widening and deepening, will however be necessary so as to eradicate possibility of damage due to flooding). Cross sections of the main rivers were surveyed in order to determine the possible discharge. The study is shown below. (cf. Fig. 3).

2) Study of respective blocks

i) Extension farm 1 (1,183 acres)

There is no problem in the Eastern part of this block, because the El Mamo River runs down through this area.

In the central part there are a few gullies which have no definite sections in their lower reaches. Thus the condition swampy and unsuitable for drainage.

In the western part, there is a watershed between the Guanapo River and the El Mamo River. Therefore the drains which run down southwest wards and pour into the El Mamo River before C. R. Highway are very necessary.

In the catchment area of the Guanapo River, there is no problem.

ii) Extension farm 11 (600 acres)

This block extends on both sides of the Aripo River and is limited by the Eastern Main Road.

On the left side of the river there is no problem, because water can easily be discharged into the river.

On the right side three catchdrains are necessary, the first one which goes down south along the Western boundary, the second which goes West from Antigua Road along the Eastern Main Road and the last which goes East from Antigua Road to Aripo River along the Eastern Main Road.

iii) Block 1 (484 acres)

Water in this block has only to be discharged into the El Mamo River and the drains which come down from the Extension farm 1.

The Western side of Bermuda Road belongs to a catchment area of the Guanapo River. One drain must therefore be led to the Guanapo River.

iv) Block 11 (135 acres)

This Block which is situated along the Maturita River has no problem in particular.

However the Maturita River itself must be made straight as it is meandering remarkably.

v) Block 111 (880 acres)

The Eastern half of this block belongs to the catchment area of the El Mamo River. Water can tehrefore be led into the river by new drains which run down southwards.

The Western half of this block is in the direct catchment area of the Caroni River. Water must therefore be led through Block V into the river by the drains which it is desirable to unite with the existing gullies.

vi) Block IV (261 acres)

Water on the Eastern side of Demerara Road can easily be discharged into the Maturita River. On the other hand, water on the western side must be caught by a drain which is built along the Western boundary of this block before being led to the lower reaches of the Maturita River.

vii) Block V (546 acres)

This block which is 100 to 70 feet in elevation is the lowest of all. It belongs to the direct catchment area of the Caroni River and is situated near the river.

Water in the greater part of the block must be discharged into the Caroni River by drains which come from Block 111 and are united with existing gullies like Block 111.

On the other hand water in the Western part must be discharged into the Guanapo River.

viii) Pig Farm 1 and 11 (210 acres)

These blocks are situated along the El Mamo River as well as Amazon Road. Drainage is very easy by arranging the tributaries of the EL Mamo River.

ix) Pilot Farm (93 acres)

This block which was developed in 1963 is situated on both sides of the Cenza River one of the tributaries of the Aripo River.

There seems to be no difficulty in effecting drainage.

3) Calculation of designed flood discharge

Designed flood discharge at the points where the Eastern Main Road crosses over each river will be calculated in order to examine the discharging capacity of the rivers.

i) Determination of the designed basic rainfall precipitation which is River. The maximum daily rainfall in these ten years at the station, El Naranjo which is 4.66 inches is adopted as the one which represents the intensity of the mountainous zone in the catchment areas.

ii) Determination of 'T' the arrival time of flood at Eastern Main Road:
Velocity of flood = V = 72 $(H/L)^{0.6}$ (km/hr.) T = (L/V) (hr.) (by Rziha's formula)

	Name of River	H	L	(H/L)	$(H/L)^{0.6}$	V	T
1.	Maturita River	372 m	4,180 m	0.0890	0.234	16.8 km/hr.	0.25 hr.
2.	Guanapo River	856	12,550	0.0682	0.200	14.4	0.87
3.	El Mamo River	387	4,670	0.0829	0.224	16.1	0.29
4.	Aripo River	818	10,940	0.0748	0.211	15.2	0.72
5.	Cenza River	398	4,670	0.0852	0.228	16.4	0.28

iii) Determination of 'r' the mean hourly rainfall intensity up to arrival of flood. The arrival time of flood which is 1 hour must be assumed for this calculation, as all the calculated arrival times of flood are within 1 hour.

R = 4.66 inches = 118 mm

$r = R/24 \times (24)/T^{2/3} = 118/24 \times (24)/1^{2/3} = 40.9$ mm/hr.

iv) Determination of 'Q' max. the max. flood discharge.

$Q_{max.} = 1/3.6\ frA$

f: run-off coefficient at flood peak

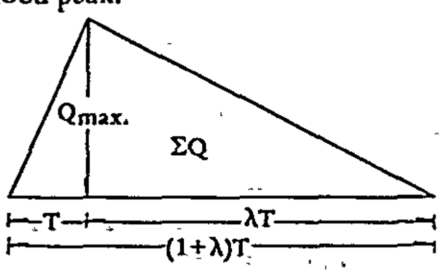
	Name of River	1/3.6	f	r	A	$Q_{max.}$
1.	Maturita River	0.2778	0.45	40.9	4.3 km^2	22.0 m^3/s
2.	Guanapo River	0.2778	0.50	40.9	34.3	194.9
3.	El Mamo River	0.2778	0.45	40.9	3.7	18.9
4.	Aripo River	0.2778	0.50	40.9	20.7	117.6
5.	Cenza River	0.2778	0.50	40.9	5.4	30.7

v) Determination of λT, the discharging time after flood peak.

$\lambda = Rf'A/1.8TQmax - 1$

f: total run-off coefficient in a flood

$\Sigma Q = 1,000\ Rf'A = ½Q_{max.}(1+\lambda)T.3,600$



	Name of river	R	f	A	$Rf A$	T	Qm^3/s	1.8 TQ	$\frac{Rf A}{1.8\ TQ}$	λ	λT hr
1	Maturita R.	118mm	0.75	4.3km²	380.6	0.25hr	22.0	9.90	38.44	37.44	9.36
2	Guanapo R.	118	0.80	34.3	3,237.9	0.87	194.9	305.2	10.61	9.61	8.36
3	El Mamo R.	118	0.75	3.7	327.5	0.29	18.9	9.87	33.18	32.18	9.62
4	Aripo R.	118	0.80	20.7	1,954.1	0.72	117.61	132.4	12.82	11.82	8.51
5	Cenza R.	118	0.80	5.4	509.8	0.28	30.7	15.5	32.89	31.89	8.93

vi) Determination of ΣQ, the total flood discharge

$$\Sigma Q = 1{,}000\ RF'A$$

	Name of river	RF'A	ΣQ
1	Maturita R.	380.6	380,600 m³
2	Guanapo R.	3,237.9	3,237,900
3	El Mamo R.	327.5	327,500
4	Aripo R.	1,954.1	1,954,100
5	Cenza R.	509.8	509,800

4) Calculation of river discharging capacity (cf. Fig. 3)

From Fig. (3) Cross sectional areas of the rivers were measured with a planimeter and wetted perimeters of the cross sections with a scale. Subsequently the actual areas and lengths were determined, using the scale of 1/240.

$$Q = AV$$

$$V = \frac{N.R.}{D+\sqrt{R}} \quad \text{(by Kutter's ND formula)}$$

Section No.	Name of River	Measured Area	Actual Area (A)	Measured Length	Actual Wetted Perimeter (P)	Hydraulic Radius (R)	$\sqrt{R}$	Estimated River slope (I)	Rough-ness coeffi. (n)	N	D	Velocity (V)	Discharge (Q)
1	Maturita R	1.7cm^2	9.79m^2	3.8cm	9.12m	1.073m	1.036	$\frac{1}{110}$	0.050	4.116	1.159	2.01m/s	19.7m^3/s
2	Guanapo R.	4.5	25.92	11.2	26.88	0.964	0.982	$\frac{1}{310}$	0.050	2.469	1.174	1.10	28.5
3	El Mamo R.	1.4	8.06	3.4	8.16	0.988	0.994	$\frac{1}{140}$	0.050	3.653	1.161	1.67	13.5
4	El Mamo R.	1.7	9.79	8.2	19.68	0.497	0.705	$\frac{1}{100}$	0.050	4.316	1.158	1.15	11.3
5	Aripo R.	1.9	10.94	4.2	10.08	1.085	1.042	$\frac{1}{200}$	0.050	3.062	1.166	1.50	16.4
5	Cenza R.	1.5	8.64	3.2	7.68	1.125	1.061	$\frac{1}{110}$	0.050	4.116	1.159	2.09	18.1
6	Aripo R.	3.0	17.28	5.2	12.48	1.385	1.176	$\frac{1}{200}$	0.050	3.062	1.166	1.81	32.3
7	Gaunapo R.	3.9	22.46	12.8	30.72	0.731	0.855	$\frac{1}{200}$	0.050	3.062	1.166	1.11	24.9
8	El Mamo R.	1.6	9.22	3.3	7.92	1.164	1.079	$\frac{1}{160}$	0.050	3.419	1.162	1.78	16.4
9	El Mamo R.	3.4	19.58	5.64	13.44	1.457	1.207	$\frac{1}{700}$	0.050	1.666	1.204	1.01	19.8

Comparison between these river discharging capacities and the maximum flood discharges calculated before, at the crossings of Eastern Main Road are shown in the following table:

Section No.	Name of River	River discharging capacity	Maximum flood discharge	B/A
1	Maturita R.	19.7 m^3/s	22.0 m^3/s	1.1
2	Guanapo R.	28.5	194.9	6.8
3	El Mamo R.	13.5	18.9	1.4
5	Aripo R.	16.4	117.0	7.1
5	Cenza R.	18.1	30.7	1.7
6	Aripo R.	31.3	147.7	4.7

The smaller rivers, the Maturita, the El Mamo and the Cenza can discharge between two thirds of the maximum discharge and the full maximum flood discharge.

On the oter hand the larger rivers, the Guanapo and the Aripo can discharge only one fifth to one seventh of the maximum flood discharge. This means that flooding temporarily cannot be helped. The period of flooding, however, will be within one day judging from the facts that flood arrival time on the rivers are less than one hour due to steep river-bed slopes and short lengths of the river, and flood continuing times are only eight to nine hours. The larger rivers have wide lower wild lands along them as flood basins. If cross sections inclusive of these lands are adopted for discharging the flood, they will be sufficient.

Consequently widening the river sections and making the routes straight will be very effective for the smaller rivers because the flood immediately damages the farms along them; and will be desirable but not urgent for the larger rivers.

5) Determination of drainage discharge from farms concerned. The maximum daily rainfall in these ten years at the stations. Grasslands, Aripo and Torrecilla, Arima is 4.07 inches (=103mm).

This is adopted as the rainfall which represents conditions in open fields in the catchment areas.

$$Q = \frac{ARn\ f.10}{T \cdot 3{,}600}$$

Q:	drainage discharge (m^3/sec)
A:	catchment area (ha)
Rn:	maximum rainfall in n hours (mm)
f:	run-off coefficient which is 0.4 to 0.6 in flat land.
T:	time for drainage which is now adopted as 24 hrs.

Block	Area		R24	F	T	Q		Remarks
	acres	ha	mm		hrs	m^3/s	cusecs	
Extension farm I	1,183	479	103	0.45	24	2.57	90.8	
Extension farm II	600	243	103	0.50	24	1.45	51.2	
Block I	484	196	103	0.50	24	1.17	41.3	
Block II	135	55	103	0.60	24	0.39	13.8	
Block III	880	356	103	0.45	24	1.91	67.5	
Block IV	261	106	103	0.50	24	0.63	22.2	
Block V	546	221	103	0.50	24	1.32	46.6	
Pig Farm I	110	45	103	0.60	24	0.32	11.3	
Pig Farm II	100	40	103	0.60	24	0.29	10.2	
Pilot Farm	93	38	103	0.60	24	0.27	9.5	

6) Determination of drain sections in farms concerned.

i) Land Slope

Block	Contour line	Difference of contour		Interval of contour	Land slope	Remarks
Extension Farm I	200ft-175ft	25 ft	7.62m	400m	7.62/400=1/52	
	175 - 150	25ft	7.62m	1,300	7.62/1,300=1/171	
Farm II	225 - 200	25ft	7.62m	600	7.62/600=1/79	
Block I	150 - 125	25ft	7.62m	1,500	7.62/1,500=1/197	
Block II	175 - 150	25ft	7.62m	700	7.62/700=1/92	
Block III	125 - 100	25ft	7.62m	1,200	7.62/1,200=1/157	
Block IV	200 - 175	25ft	7.62m	600	7.62/600=1/79	
	175 - 150	25ft	7.62m	700	7.62/700=1/92	
Block V	100 - 75	25ft	7.62m	1,500	7.62/1,500=1/197	

Thus, the designed slope of drains is determined as 1/200 to be on the safe side.

ii) Drain sections

I: Slope which is $\frac{1}{200}$

N: Coefficient of roughness which is assumes as 0.030
side slope which is 1 : 1 is adopted in this case due to the soil type.

By Kutter's N.D-formula

$$Q = A.v$$
$$A = (b+h)h$$
$$V = \frac{N.R}{D+\sqrt{R}}$$

b: width of bed
h: depth
N = 4.005
D = 0.6993

Type of drain	h	b	A	V	Q
No. 1	0.50m	1.50m	$1.00m^2$	1.071m/s	$1.071m^3/s$
No. 2	0.50	1.00	0.75	0.991	0.743
No. 3	0.50	0.75	0.625	0.937	0.585
No. 4	0.50	0.50	0.50	0.866	0.433

(cf. Fig. 4)

Drains for the extension farms I and II must have sufficient capacity to also accommodate the inflow from the higher areas.

(iii) Locations and dimensions of drains

The Locations are shown on Fig. 4. The other dimensions are shown in the following table which is titled Summary of proposed drains.

7) Estimate of cost for Works

(i) For drains

Cross sectional area for cutting : A = (b + h')h'

Type No. 1	b = 1.5m h' = 0.7	$A = (1.5 + 0.7) \times 0.7 = 1.54m^2$
Type No. 2	b = 1.0 h' = 0.7	$A = (1.0 + 0.7) \times 0.7 = 1.19m^2$
Type No. 3	b = 0.75 h' = 0.7	$A = (0.75 + 0.7) \times 0.7 = 1.02m^2$
Type No. 4	b = 0.5 h' = 0.7	$A = (0.5 + 0.7) \times 0.7 = 0.84m^2$

Earth volume for cutting

a) New drain

	(A)		(L)		(V)
Type No. 1	$1.54m^2$	x	3,030m	=	$4,666m^3$
Type No. 2	1.19	x	6,760	=	8,044
Type No. 3	1.02	x	13,520	=	13,790
Type No. 4	0.84	x	5,110	=	4,292

L: total length of drains

b) Drain to be improved

Type No. 1	$1.54m^2$	x	1,380m	$\times 1/3 = 708m^3$
Type No. 2	1.19	x	5,460	$\times 1/3 = 2,166$

Type No. 3	$1.02m^2 \times 3,020 \times 1/3 = 1,027$	
Type No. 4	$0.84 \quad \times 2,210 \times 1/3 = \quad 619$	

The mean earth volume for cutting of drain to be improved is regarded as one third as much as new drain.

Total cost for works. ($2.20/yds = $2.88/m³)

Type No. 1	$(4,666m^3 + 708m^3) \times \$2.88/m^3 =$	$ 15,477.-
Type No. 2	$(8,044 + 2,166) \times \$2.88/m^3 =$	$ 29,405.-
Type No. 3	$(13,790 + 1,027) \times \$2.88/m^3 =$	$ 42,673.-
Type No. 4	$(4,292 + 619) \times \$2.88/m^3 =$	$ 14,144
		$101,699
Contingency about 10% of above total		$ 9,301.-
(for crossing road-works)		$110,000.-

(ii) For rivers to be improved

the EL Mamo River:	4km long upstream from C.R. Highway
the Maturita River:	3km long downstream from Eastern Main Road
Total	7km = 7,000 m

Assuming that $1.00 per meter is necessary for improvement, the earth volume to be cut will be about $0.35m^3/m$.

7,000 m x $1.00/m = $7,000.-

(iii) Grand total

$110,000 + $7,000 = $117,000.-

SUMMARY OF PROPOSED DRAINS

Block	No. of drain	Classification of drain	Length of drain	Details		Remarks
				Improvement	Newcut	
Extension Farm I	1	Section (3)	850m/2,590ft	450m/1,370ft	400m/1,220inch	
	2 - 1	(4)	500/1,520	500/1,520	- / -	
	2 - 2	(2)	1,400/4,270	1,400/4,270	- / -	
	2	(2)	440/1,340	440/1,340	- / -	
	3 - 1	(3)	1,400/4,270	1,400/4,270	- / -	
	3 - 2	(3)	500/1,520	500/1,520	- / -	
Extension Farm II	17	(4)	650/1,980	- / -	650/1,980	
	18	(3)	370/1,130	- / -	370/1,130	
	19	(3)	1,080/3,290	- / -	1,080/3,290	
Block I	1	(2)	790/2,410	- / -	790/2,410	

Block	No. of drain	Classification of drain	Length of drain	Details		Remarks
				Improvement	Newcut	
	2	(1)	1,120/3,410	90/270	1,030/3,140	
	3	(2)	1,580/4,820	750/2,290	830/2,530	
	4 - 1	(3)	950/2,900	- / -	950/2,900	
	4 - 2	(4)	410/1,250	- / -	410/1,250	
	4	(3)	500/1,520	- / -	500/1,520	
Block II	12 - 1	(3)	1,480/4,510	- / -	1,480/4,510	
	12 - 2	(4)	850/2,590	- / -	850/2,590	
	12	(3)	250/760	- / -	250/760	
Block III	5	(3)	1,050/3,200	- / -	1,050/3,200	
	6	(3)	650/1,980	350/1,070	300/910	
	6	(2)	1,260/3,840	850/2,590	410/1,250	
	7	(3)	1,380/4,210	- / -	1,380/4,210	
	8	(3)	400/1,220	- / -	400/1,220	
	8	(2)	950/2,900	700/2,130	250/770	
	9	(3)	800/2,440	- / -	800/2,440	
	9	(2)	1,140/3,470	220/670	920/2,800	
	10	(3)	900/2,740	- / -	900/2,740	Total length
Block IV	13 - 1	(3)	1,500/4,570	- / -	1,500/4,570	Improvement
	13 - 2	(3)	1,660/5,060	- / -	1,660/5,060	(1) 1,380m 4,200ft
	13.3	(2)	600/1,830	- / -	600/1,830	(2) 5,460 16,650
	13	(2)	700/2,130	- / -	700/2,130	(3) 3,020 9,210
						(4) 2,210 6,730
Block V	8	(1)	1,670/5,090	870/2,650	800/2,440	Newcut
	9	(1)	1,620/4,940	420/1,280	1,200/3,660	(1) 3,030m 9,240ft
	9 - 1	(4)	620/1,890	- / -	620/1,890	(2) 6,760 20,600
	9 - 2	(4)	720/2,190	- / -	720/2,190	(3) 13,520 41,200
	10	(2)	1,600/4,880	560/1,710	1,040/3,170	(4) 5,110 15,570
	11	(2)	1,760/5,360	540/1,650	1,220/3,710	Total
Pig Farm I	14	(4)	1,460/4,450	900/2,740	560/1,710	(1) 4,410m 13,440ft
						(2) 12,220 37,250
	15	(4)	1,280/3,900	550/1,680	730/2,220	(3) 16,540 50,410
						(4) 7,320 22,300
Pig Farm II	16	(4)	830/2,530	260/790	570/1,740	Gr. total ft
						40,490 123,400
Pilot Farm	20	(3)	820/2,500	320/980	500/1,520	

8) Remarks:

A five foot-contour map is necessary to determine the route of a drain exactly. However, as only a twenty-five foot-contour map reduced to one in ten thousand was available at this stage, the route could only roughly be determined.

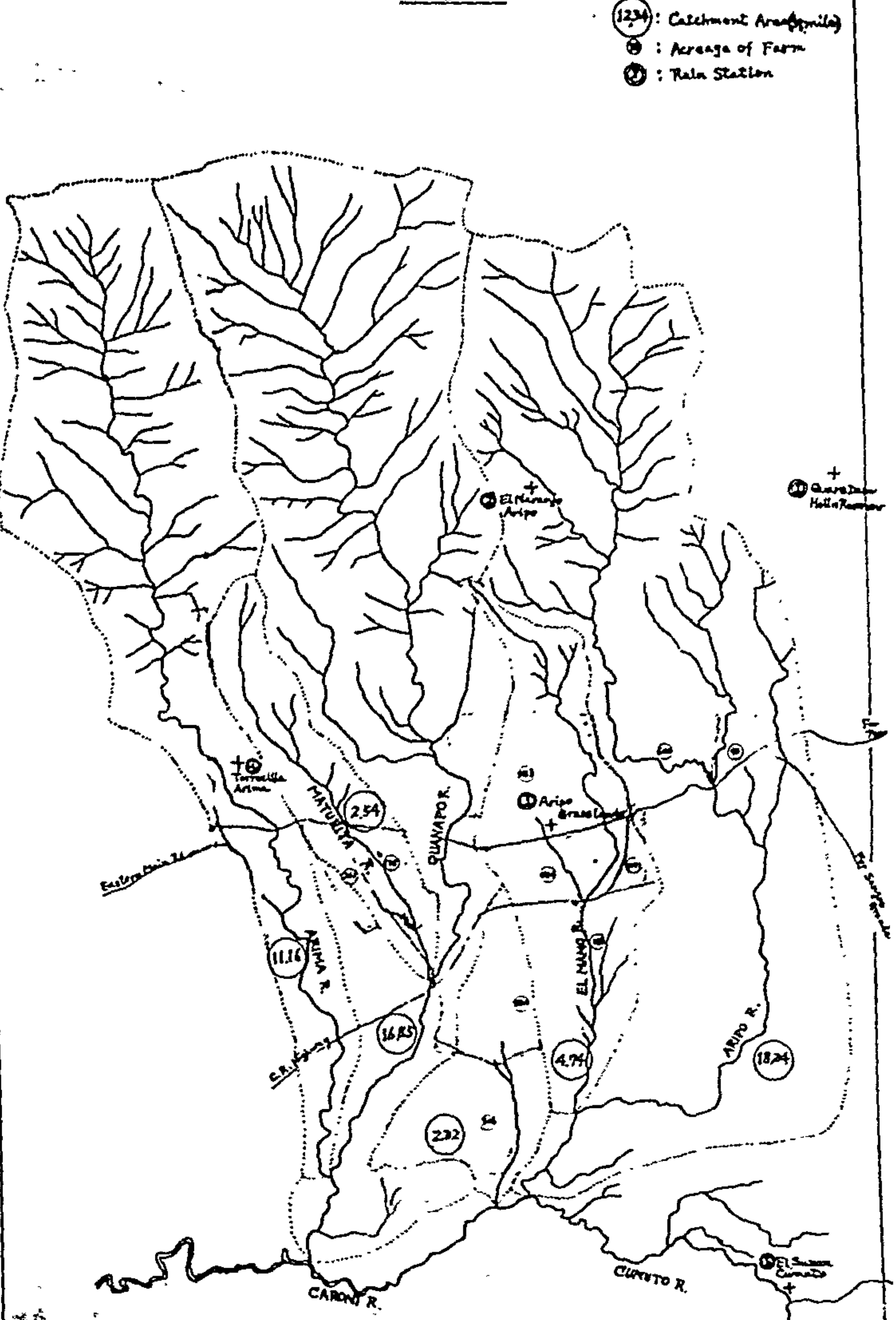
Although a more detailed study is therefore necessary to carry out this scheme, the outlined, scheme and the estimated cost may be satisfactorily determined on the basis of this report. The finely and effective implementation of this report is strongly advised and cannot be over-emphasized.

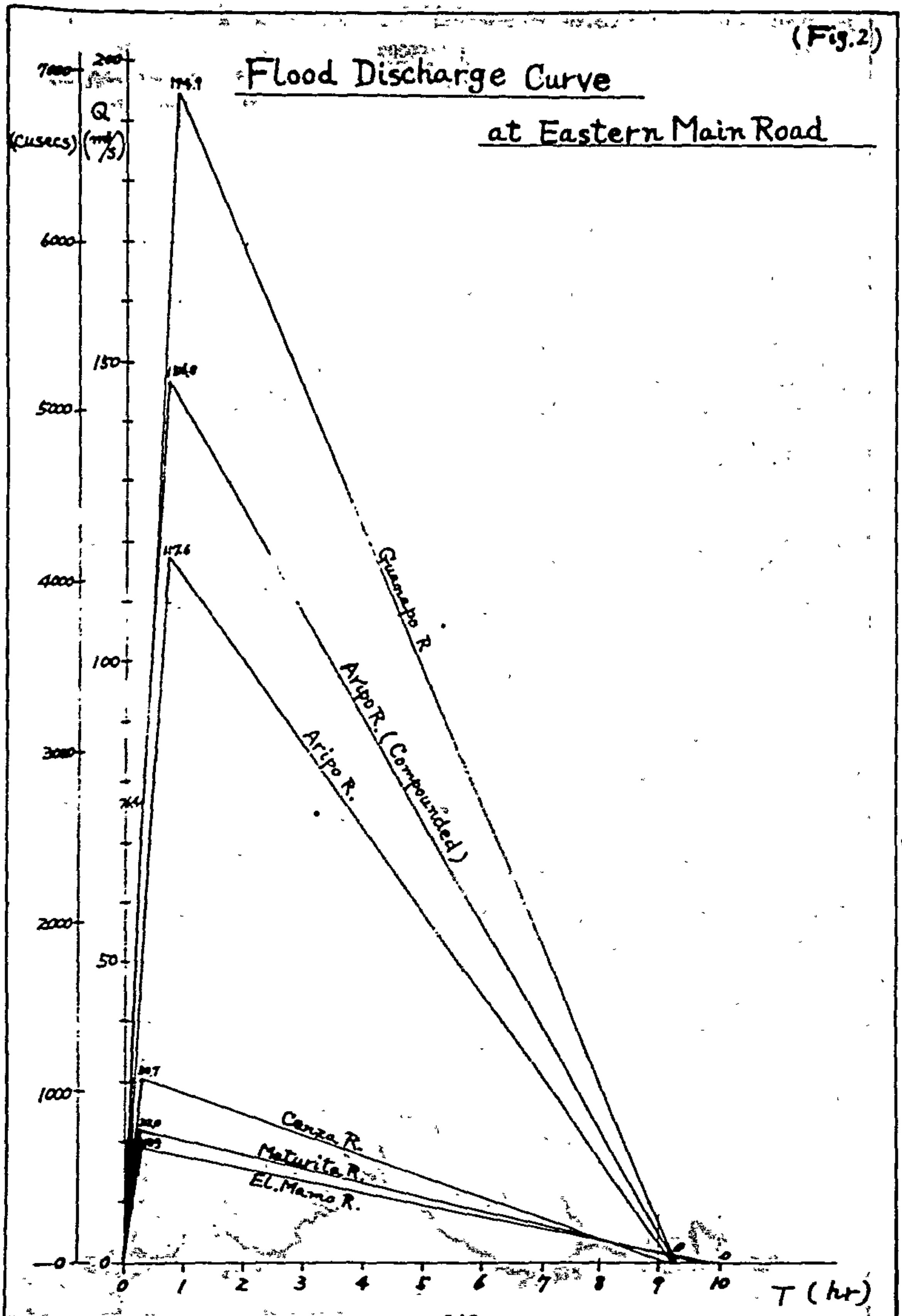
Junichi Kitamura

CATCHMENT AREA OF RIVERS
(WHICH GO THROUGH WALLER FIELD)

(Fig. 1)

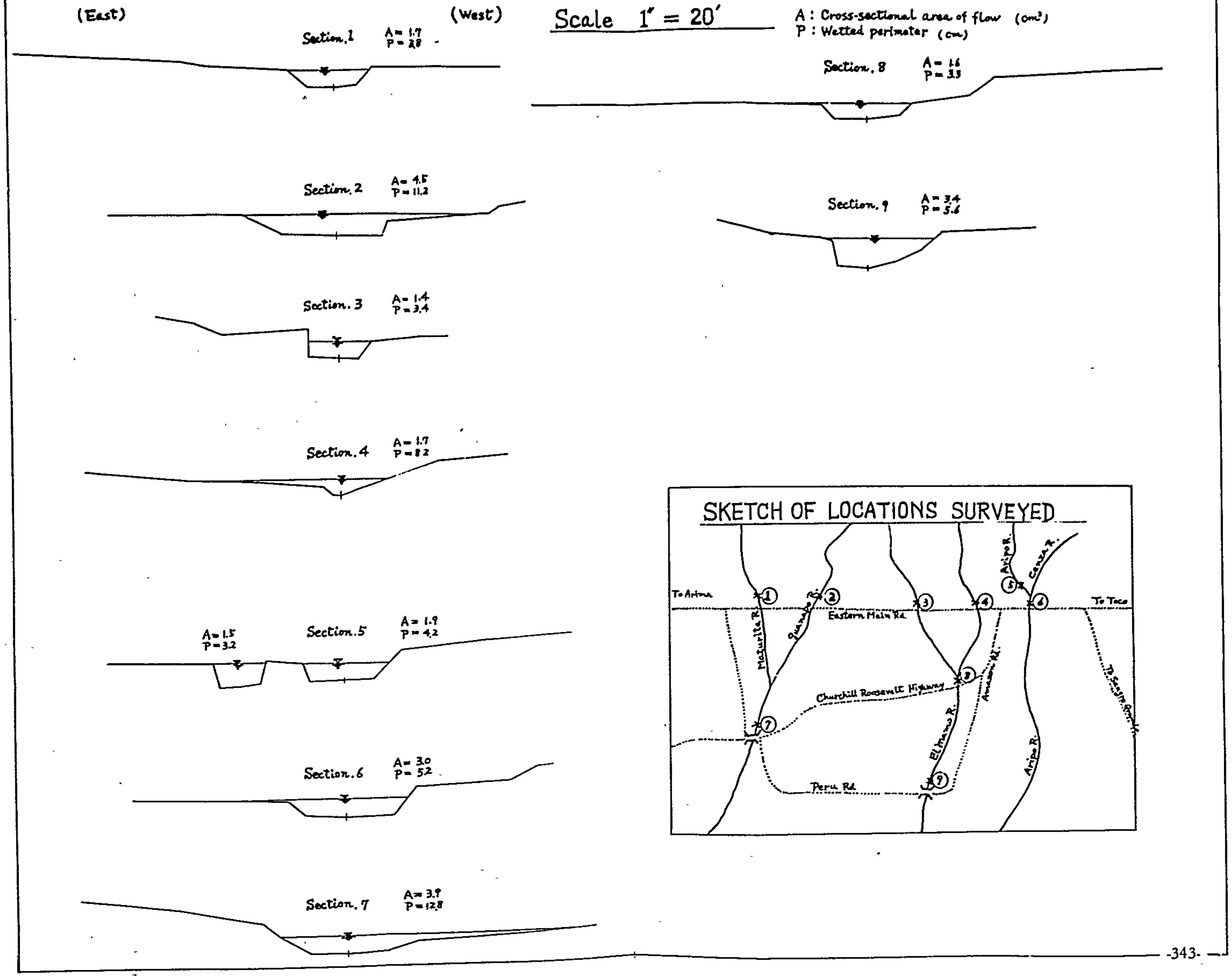
1 : 50000




(Fig.2)
Flood Discharge Curve
at Eastern Main Road
Q
(cusecs)
7000
6000
5000
4000
3000
2000
1000
0
200
150
100
50
0
Guanapo R.
AripoR.(Compounded)
Aripo R.
Cuaza R.
Maturita R.
El.Mamo R.
0
1
2
3
4
5
6
7
8
9
10
T (hr)

CROSS SECTIONS OF RIVERS IN WALLER FIELD

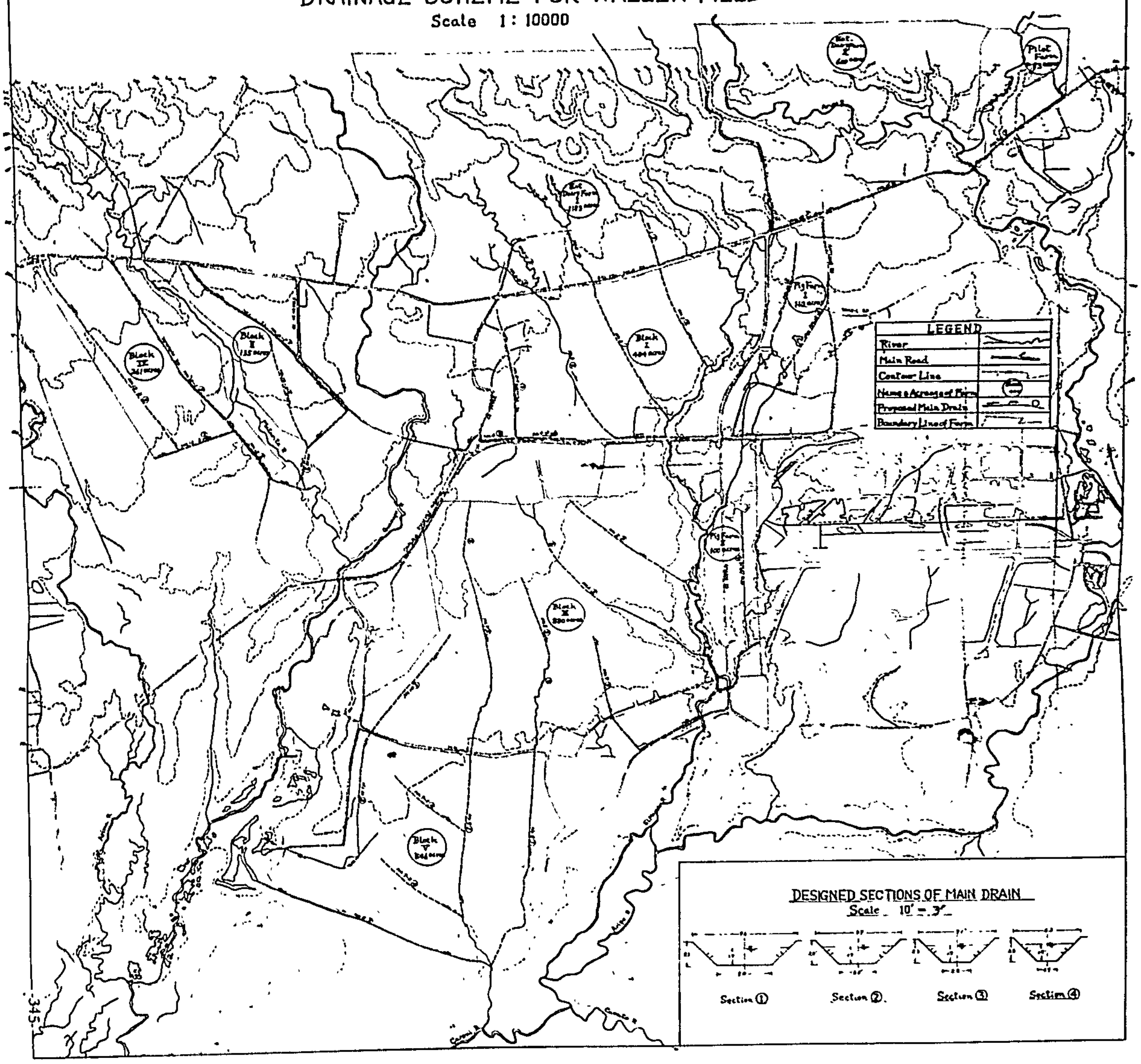
(Fig.3)



(Fig. 4)

DRAINAGE SCHEME FOR WALLER FIELD

Scale 1 : 10000



(5) CARLSEN FIELD September 1968

MOVEMENT

30, May	Visited a branch office of the Ministry of Agriculture in Carlsen Field to investigate the existing drainage condition with Mr. J. V. Smith, Drainage Superintendent (north).
19, June	Visited again with Mr. C. Taylor, Director of Drainage and was taken on tour by Mr. M. Percy of Central Experimental Station who explained the dairy farm scheme to us.
3, July	The requested survey for river cross section was completed.
15, July	Visited there to investigate the flooding condition in rainy season.
18, July	The soil map of Carlsen Field was sent by Mr. M. Percy.
10, Aug.	The information on soils was sent by Mr. M. Percy

INTRODUCTION

For puprose of this study, the following is quoted from the letter from the Ministry of Planning and Development dated, February 29th in 1968;

'At a recent meeting of the Crown Lands Development Programme Committee, the view was expressed that a drainage study of the project areas was most desirable particularly in view of the drainage problems existing on the farms at present.

It was stressed that corrective measures should be implemented forthwith to solve the matter but that such action would be more effectively carried out if there was some master plan to follow.'

As a matter of urgency, a drainage study of the entire area, that is, Waller Field, Carlsen Field and Esmeralda in that order of priority had been requested to be undertaken. According to the priority, the drainage study for Waller Field was completed in April.

Therefore, in this report, the drainage study for Carlsen Field will be carried out.

DESCRIPTION OF AREA STUDIED

The area to be drained in Carlsen Field is about 2,000 acres. The details which were explained by Mr. M. Percy, are shown as follows:-

(1) completed dairy farms 34 x 16 acres = 544 acres
(2) scheduled dairy farms
- 6 blocks ... 180 + 115 + 180 + 210 + 225 + 105= 1,015 acres
(3) scheduled pig farms 14 x 5 acres = 70 acres

Total		1,629 acres
Others	about	400 acres

According to a topographical map of $\frac{1}{10,000}$ with 25 feet contours the area is 60 ft to 130 ft above the sea level and mildly slopes to the west.

The Uquire River and the Arena River which are branches of the Chandernagore River, originate from low hills of 220 ft and 310 ft above the sea level, respectively, and meander through Carlsen field. After passing through the area, they meet together and become the Chandernagore River to flow into the Gulf of Paria. River slopes in Carlsen Field are mostly 1/700 to 1/900. On the other hand, a slope of the Chandernagore River (which is the lower reaches of the Uquire and Arena River) is about 1/500. This means that the slope of the lower reaches of the river is steeper than that of the upper reaches.

Almost all the parts of these rivers, however, are still in natural condition, and have many tributaries and bendings.

The cross sectional areas of the rivers are too small to discharge the flood water. Consequently flooding occurs very easily and frequently for short periods in rainy season.

The dimensions of the rivers are shown on the following table and Fig. 1. The cross sections of the rivers are also shown on Fig. 2.

Location	Catchment km^2	Area $miles^2$	Length of River km	Length of River miles	Difference of Elevation m	Difference of Elevation ft.
at (3) of Uquire R.	9.31	3.59	5.95	3.70	20.4 - 67.1	67 - 220
at (9) of Uquire R.	20.89	8.07	9.85	6.12	15.8 - 67.1	52 - 220
at (6) of Arena R.	15.10	5.83	10.00	6.21	21.3 - 94.5	70 - 310
at (9) of Arena R.	20.54	7.93	14.70	9.13	15.8 - 94.5	52 - 310
at mouth of Chandernagore R.	54.43	21.02	17.65	10.97	0 - 67.1	0 - 220
			22.50	13.98	0 - 94.5	0 - 310

As to rainfall in this area, daily rainfall precipitation have been observed since 1955. The maximum daily rainfall in each month is shown on the following table. This table shows that the most frequent occurrence of the maximum daily rainfall is in June, that is, six in thirteen years. The maximum daily rainfall recorded is 4.80" in June 1964. Calculation for probability of the maximum daily rainfall was carried out by this data. It is shown on the following table on page 4 and Fig. 3.

Maximum Daily Rainfall in Inches each month

STATION: Carlsen Field

DATE ESTABLISHED: 14.9.49 - Recording

13.7.55 - Non-recording

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Total	Remarks
1967	0.77	0.24	0.70	1.04	0.57	0.96	1.10	2.08	1.68	2.15	2.24	1.08		
1966	0.40	0.75	0.36	0.79	0.66	3.30	2.70	1.25	2.00	1.61	0.80	2.25		
1965	1.65	0.48	0.23	1.45	1.63	2.03	1.58	0.92	1.29	1.41	1.62	1.39		
1964	1.36	0.17	0.35	1.12	1.05	4.80	2.27	1.02	2.10	2.49	1.11	1.24		
1963	0.68	0.59	0.59	0.35	1.32	1.23	1.31	3.67	2.40	1.46	1.40	0.70		
1962	0.46	0.41	0.30	0.50	0.53	3.17	2.03	1.81	0,80	0.49	1.85	1.56		
1961	1.03	0.30	0.37	0.25	0.25	1.30	1.30	0.83	2.13	0.93	1.73	0.63		
1960	0.32	0.13	0.10	1.91	0.74	3.18	1.07	1.49	0.60	1.83	1.12	1.95		
1959	0.26	0.14	0.48	0.31	0.83	1.84	0.76	0.89	1.68	1.10	1.09	0.83		
1958	1.27	0.66	0.09	2.09	1.58	(3.58)	(1.14)	(1.23)	(0.73)	(1.60)	(1.30)	(0.99)		June to Dec. fig. taken from T.C.P., Longdonville
1957	0.39	1.47	0.19	1.16	0.88	1.07	1.87	0.96	0.72	0.62	2.41	1.89		
1956	1.37	1.09	0.57	1.31	1.67	2.15	0.79	2.66	1.77	2.05	1.12	1.40		
1955	0.64	1.48	0.68	0.83	0.51	1.60	2.92	2.81	3.84	2.34	2.05	0.89		

i	i/n x 100	max. daily rainfall
1	7.69	4.80"
2	15.38	3.84
3	23.08	3.67
4	30.77	3.58
5	38.46	3.30
6	46.15	3.18
7	53.85	3.17
8	61.54	2.66
9	69.23	2.41
10	76.92	2.24
11	84.62	2.13
12	92.31	2.03
13	100.00	1.84
n = 13		

Probability	Max. daily rainfall
Once in 2 years	2.95"
Once in 5 years	3.85
Once in 10 years	4.30
Once in 20 years	4.90
Once in 50 years	5.60
Once in 100 years	6.00

Geological condition can be studied by the soil map (fig. 4) which was sent by Mr. M. Percy.

The areas along the Uquire River and the arena River are flat lands which consist of clayey alluvium.

The other areas are terraces which consist of quarternary silty clay and terrace and levee sand.

As to vegetation surrounding Carlsen Field, sugarcane fields stretch out in the north, broken forests to forests in the east and south-east, cultivated lands, sugar-cane fields to forest in the south-west, and sugar-cane field and rice field in the west.

On the other hand, as to vegetation in Carlsen Field, grass lands take the greater part, and swampy wild lands and bare lands like borrow-pits along the Highway take some part.

MASTER PLAN OF DRAINAGE

1) Outline of the plan

For a designed flood, the flood which occurs once in ten years must be adopted. The designed flood must be calculated due to the maximum daily rainfall once in ten years, because of no flood records in this area.

Natural drainage is possible for this area because the land is situated 60 - 130 ft above the mean sea level and the rivers have sufficient slopes (which are 1/500 - 1/900) to discharge flood-water.

As stated below the Chandernagore River which unites the Uquire River and the Arena River as well as the Arena River however, has insufficient river-section to discharge the maximum designed flood. Therefore, it occurs very often that the water from the area are impeded by the narrow section and flood in the area.

Improvement of the rivers, that is, making them straight, widening and deepening and taking off weeds and shrubs on the slope of the banks, will be inevitably necessary. Cross sections of the main rivers were surveyed for checking the discharge capacity.

2) Study of respective blocks

No. 1 Block (Drairy farm 544 acres + 180 acres)

This block is situated in the catchment of the tributaries of the Uquire River No. 1-1, No. 1-2, No. 1-3 and No. 1-4 (cf. Fig. 6)

The tributary No. 1-1 from the bridge on Yaraba Road to the confluence with the Uquire River is now in good condition by having been dredged and the weeds and shrubs having been cutlassed.

The others, however, are in natural condition (shallow by deposit, meandering, weedy and shrubby).

In this area, the 34-16 acre dairy farms (total = 544 acres) were first made and the southern part of about 180 acres are being ploughed to be also made into dairy farms.

No. 2 Block (Dairy farm 105 acres)

This block which is surrounded by the Chaguanas-San Fernando Highway, Connector Road and the Uquire River, has no main drains. As a result, this block is very swampy and had partly been used as a borrow-pit for the Highway.

Dairy farms have not yet been established in this block.

No. 3 Block (Dairy farm 225 acres)

This block is situated in the catchment area of the Uquire River and its tributaries No. 3-1, No. 3-2 and No. 3-3 next to No. 2 block and further upstream on the Uquire River. It seem to be impossible for these tributaries to drain the designed flood water immediately, because of very small cross-sections.

No. 4 Block (Dairy farm 210 acres)

This block is situated in the catchment area of the Uquire River and its tributaries No. 4-1 and No. 4-2; it is a terrace 60-100 ft in elevation, further upstream, next to No. 3 block. There seems to be no problem except the Uquire River meanders.

No. 5 Block (Dairy Farm 180 acres)

This block which lies on the south of No. 4 block, is a terrace 70-130 ft in elevation, where many streams flow down, joining together in the lower reaches and flow alongside of Uquire Rd. into Uquire River.

The tributary after joining together must be made straight and wide.

No. 6 Block (Dairy farm 115 acres)

This block surrounded by the proposed Highway (to be extended), Balai Trace and Uquire Road is in the catchment area of the Arena River and its tributary No. 6-2.

In the western part along the Highway, to be extended, drain No. 6-1 will be needed to be changed to take water from the tributaries of Arena River before they cross the Highway.

No. 7 Block (Pig Farm 70 acres)

This block which lies between Carlsen Runway and No. 6 Block, is a strip of land near the border line between the catchment of the Uquire River and that of the Arena River.

Eventually, this area belongs to the catchment area of the Arena River and is under good condition for drainage.

3) Calculation of designed flood discharge:

Designed flood discharge at the points of No. 3, No. 6, No. 9 and No. 10 (near the mouth of the Chandernagore River) will be calculated in order to examine the discharging capacity of the rivers.

i) Determination of the designed basic rainfall precipitation (R). The maximum daily rainfall which occurs once in ten years, at Carlsen Field is adopted as the one which represents the intensity in whole catchment areas.

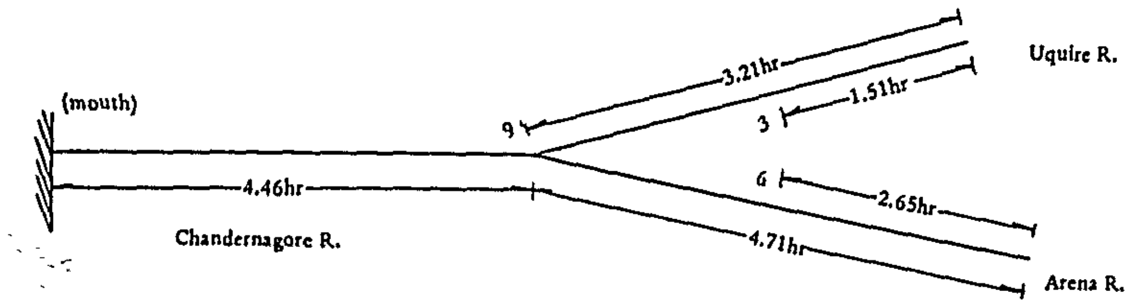
(i.e) $R = 4.30"$

ii) Determination of the arrival time of flood from the upmost to each point (T) by Rziha's formula

Velocity of flood: $V = 72\ (H/L)\ 0.6$ (km/hr)
H: Difference of elevation of riverbed from the upmost to the point
L: Length of river from the upmost to the point. Arrival time of flood: $T = L/V$ (hr)

	Location	H	L	H/L	(H/L)0.6	V	T
1	at (3) of Uquire R.	46.7m	5.950m	0.00785	0.05457	3.93 km/hr	1.51 hr
2	at (9) of Uquire R.	51.3	9.850	0.00521	0.04267	3.07	3.21
3	at (6) of Arena R.	73.2	10.000	0.00732	0.05233	3.77	2.65
4	at (9) of Arena R.	78.7	14.700	0.00535	0.04335	3.12	4.71
5	at the mouth of Chandernagore R.	15.8	7.800	0.00203	0.02424	1.75	4.46

(from (9) to the mouth)



iii) Determination of the mean hourly rainfall intensity during T

$$R = 4.30" = 109.2\text{mm}$$
$$r = \frac{R}{24}\left(\frac{24}{T}\right)^{2/3}$$

	Location	R/24mm	24/T	$(24/T)^{2/3}$	r mm
1	at (3) of Uquire R.	4.55	15.89	6.32	28.8
2	at (9) of Uquire R.	4.55	7.48	3.83	17.4
3	at (6) of Arena R.	4.55	9.06	4.35	19.8
4	at (9) of Arena R.	4.55	5.10	2.96	13.5
5	at the mouth of Chandernagore R.	4.55	5.38	3.07	14.0

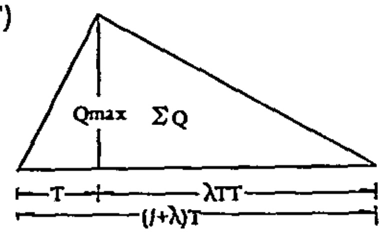
iv) Determination of the maximum flood discharge (Qmax.)

$$Q\text{max.} = \frac{1}{3.6} frA$$

f: run-off coefficient at flood peak
A: catchment area $(\text{km})^2$

	Location	$\frac{1}{3.6}$	f	r mm	$A\text{km}^2$	Qmax m^3/s
1	at (3) of Uquire R.	0.2778	0.40	28.8	9.31	29.8
2	at (9) of Uquire R.	0.2778	0.40	17.4	20.89	40.4
3	at (6) of Arena R.	0.2778	0.40	19.8	15.10	33.2
4	at (9) of Arena R.	0.2778	0.40	13.5	20.54	30.8
5	at the mouth of Chandernagore R.	0.2778	0.35	14.0	13.00	17.7

v) Determination of the discharging time after flood peak (λT)



$$\Sigma Q = 1{,}000\, Rf'A = \tfrac{1}{2}Q\text{max.}\,(1+\lambda)T.3{,}600$$

1. $\lambda = \frac{Rf'A}{1.8Q\text{max.}} - 1$

f': total run-off coefficient in a flood

	Location	R mm	f	A km²	RfA	T hr	Qmax m³/s	1.8T Qmax	RfA / 1.8TQmax	λ	λT hr.
1	at (3) of Uquire R.	109.2	0.60	9.31	610.0	1.51	29.8	81.0	7.53	6.53	9.86
2	at (9) of Uquire R.	109.2	0.55	20.89	1,254.7	3.21	40.4	233.4	5.38	4.38	14.06
3	at (6) of Arena R.	109.2	0.60	15.10	989.4	2.65	33.2	158.4	6.25	5.25	13.91
4	at (9) of Arena R.	109.2	0.55	20.54	1,233.6	4.71	30.8	261.1	4.72	3.72	17.52
5	at the mouth of Chandernagore R.	109.2	0.50	13.00	709.8	4.46	17.7	142.1	5.00	4.00	17.84

vi) Determination of the total flood discharge (ΣQ)

$$\Sigma Q = 1{,}000\ RfA$$

	Location	RfA	ΣQ m³
1	at (3) of Uquire R.	610.0	610,000
2	at (9) of Uquire R.	1,254.7	1,254,700
3	at (6) of Arena R.	989.4	989,400
4	at (9) of Arena R.	1,233.6	1,233,600
5	at the mouth of Chandernagore R.	709.8	709,800

4) Calculation of river discharging capacity (cf. Fig. 2)

From Fig. 2 cross sectional areas of the rivers were measured with a plainimeter and wetted perimeters of the cross sections with a scale. Subsequently the actual areas and lengths were determined by using the scale of $\frac{1}{240}$.

The river slopes were estimated on the map of $\frac{1}{10{,}000}$ with 25 feet countours.

$$Q = AV$$

$$V = \frac{N.R}{D+\sqrt{R}} \qquad \text{(by Kutter's ND formula)}$$

Table of calculation for river discharging capacity

Section No.	Name of River	Measured Area (cm^2)	Actual area A (m^2)	Measured Wetted Perimeter (cm)	Actual Wetted Perimeter P(m)	Hydraulic Radius R (m)	$\sqrt{R}$	Estimated River slope I	Coefficient of Roughness n	N	D	Velocity V(m/s)	Discharge Q(m^3/s)
1	Tributary of Uquire River	4.7	27.07	6.4	15.4	1.758	1.326	$\frac{1}{700}$	0.050	1.666	1.204	1.16	31.4
2	Tributary of Uquire River	4.2	24.19	8.2	19.7	1.228	1.108	$\frac{1}{700}$	0.050	1.666	1.204	0.88	21.3
3	Uquire River	6.3	36.29	8.9	21.4	1.696	1.302	$\frac{1}{850}$	0.050	1.520	1.216	1.02	87.0
4	Uquire River	6.1	35.14	7.3	17.5	2.008	1.416	$\frac{1}{850}$	0.050	1.520	1.216	1.16	40.8
5	Uquire River	4.6	26.50	5.3	12.7	2.087	1.444	$\frac{1}{850}$	0.035	1.814	0.851	1.65	43.7
6	Arena River	3.5	20.16	5.9	14.2	1.420	1.192	$\frac{1}{850}$	0.050	1.520	1.216	0.90	18.1
7	Arena River	3.4	19.58	5.3	12.7	1.542	1.241	$\frac{1}{850}$	0.050	1.520	1.216	0.95	18.6
8	Arena River	3.3	19.01	5.3	12.7	1.497	1.223	$\frac{1}{850}$	0.050	1.520	1.216	0.93	17.7
9	Chandernagore River	3.7	21.31	7.7	18.5	1.152	1.073	$\frac{1}{500}$	0.050	1.958	1.189	1.00	21.3
10	Chandernagore River	4.2	24.19	6.2	14.9	1.623	1.274	$\frac{1}{500}$	0.050	1.958	1.189	1.01	24.4

5) Comparison between the river discharging capacities calculated in paragraph 4) and the maximum flood discharge calculated in paragraph 3) - iv).

Section	(A) River discharging capacity	Location	(B) Maximum flood discharge	B/A
No. 3	37.0 m^3/sec	at (3) of Uquire R.	29.8 m^3/sec	0.81
No. 5	43.7	at (9) of Uquire R.	40.4	0.92
No. 6	18.1	at (6) of Arena R.	33.2	1.83
No. 8	17.7	at (9) of Arena R.	30.8	1.74
No. 9	21.3	at (9) of Chandernagore R.	66.8	3.14
No. 10	24.4	at the mouth of Chandernagore R.	79.8	3.27

* Refer to Fig. 5

maximum flood discharge at (9) of Chandernagore River = 30.8 (Arena R.) + 36.0 (Uquire R.) = 66.8m^3/s

As a result of comparison between the river discharging capacities and the maximum flood discharge, it was found out that the Uquire River can discharge the maximum flood discharge, but the Arena River can only discharge about a half of maximum flood discharge and the Chandernagore River only about a third of it.

It means that temporary flooding along the Arena River and the Chandernagore River cannot be helped.

Under the existing condition the period of flooding, however will be within one and a half days judging from the following calculation.

Total flood discharge at the mouth of the Chandernagore River:

	at (9) of Uquire R.	1,254,700m^3	
(cf. 3) - vi))	at (9) of the Arena R.	1,233,600	
	at the mouth of the Chandernagore R.	709,800	(Between (9) and the mouth of the river)
	Total	3,198,100m^3	

River discharging capacity at (10) is 24.4 m^3/sec

∴ Time required for discharge will be

$$\frac{3,198,100}{24.4 \times 60 \times 60} = 36.4 \text{ hrs.}$$

The Chandernagore River below the limits, the Uquire and the Arena River does not directly affect Carlsen Field.

However, if the flooding occurs in lower lands along the Chandernagore River for lack of its discharging capacity, discharge from the upper reaches will be impeded and the influence of the backwater will reach well inside of Carlsen Field.

It is therefore suggested that widening the river section and re-aligning the route for the Chandernagore River as well as the Arena River will be necessary.

6) Determination of Sections of drains (tributaries) from the concerned farms:

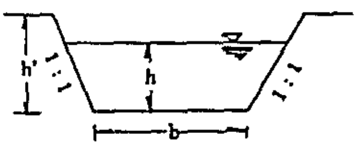
Drainage scheme cannot be made for only these farms, because many tributaries flow into these areas from outside.

Accordingly, the sections of the drains must be determined based on their catchment areas and not the areas of the farms.

However, there is no time to measure the catchment areas individually. At least three standard sections will be desinged and will be applied for all the cases.

These sections are shown in the table below.

Type	h m	b m	A m^2	V m/s	Q m^3/s
A	0.75	1.50	1.688	0.604	1.018
B	1.00	2.00	3.000	0.748	2.244
C	1.00	3.00	4.000	0.805	3.220



$n = 0.030$ ..... coefficient of roughness

$I = \frac{1}{1,000}$ .... slope of drain.

The sections which have discharges of about $1m^3/s$, $2m^3/s$, and $3m^3/s$ are determined by the following reasons.

1) Comparison between the catchment area of drain and the one of its main river.

2) Difference of the maximum flood discharge between at (3) and at (9) on the Uquire River. which is $10.6m^3/sec$.

(∵ the maximum flood discharge at (9) .... $40.4m^3/s$
at (3) .... 29.8 "
difference $10.6\ m^3/s$

This discharge of $10.6m^3/s$ comes from the drains concerned.

On the other hand, the maximum flood discharge of $30.8m^3/s$ at (9) on the Arena River, is rather less than one of $33.2m^3/s$ at (6) on the river due to arrival time of flood.

7) Study for the sections of the main river

As mentioned on 5), the Uquire River can accommodate maximum flood discharge the Arena River can only discharge about a half of it, and the Chandernagore river can only discharge about a third of it, provided that the maximum daily rainfall may be discharged during a day. In other words, the flood in the area with a day can be allowed, however, the required discharge of the Arena River will be, (cf 3) - vi))

$$\frac{1,233,600\ (m^3)}{24 \times 60 \times 60} \fallingdotseq 14.3\ m^3/sec < 17.7\ m^3/sec \text{ at } (8)$$

$$\frac{989,400\ (m^3)}{24 \times 60 \times 60} \doteqdot 11.5\ m^3/sec < 18.1\ m^3/sec \text{ at } (6)$$

Therefore, the sectional area of the Arena River is sufficient, and the required discharge of the Chandernagore River will be

$$\frac{3,198,100\ (m^3)}{24 \times 60 \times 60} \doteqdot 37.0m^3/sec. > 24.4m^3/s \text{ at } (10)$$

Therefore the sectional area of the Chandernagore River must be widened at least 1.5 times as wide as the existing section.

Summary of concerned drains: (cf Fig. 6)

Block	River System	No. of Drain	Length of New short cut	Length of Drain to be improved	Type of Drain
No. 1	The Uquire	No. 1 - 1	100m	1,400m	C
		1 - 2	200	1,600	C
		1-2-1	-	300	A
		1-2-2	-	100	A
No. 2	Theuquire	2 - 1	1,600	-	B
		2 - 2	-	600	B
No. 3	Theuquire	3 - 1	-	400	A
		3 - 2	100	1,350	B
		3 - 3	100	700	A
		1 - 3	-	200	A
		1 - 4	-	150	A
No. 4	Theuquire	4 - 1	100	-	C
		4 - 2	100	-	C
		5 - 1	-	500	B
No. 5	Theuquire	5 - 1	-	400	A
		5 - 2	-	500	B
		5 - 3	100	100	B

Block	River System	No. of Drain	Length of New short cut	Length of Drain to be improved	Type of Drain
No. 6	The Arena	No. 6 - 1	750	-	B
		6 - 2	-	1,150	C
		6 - 2 - 1	150	250	A
		6 - 2 - 2	-	150	B
Total	A		250	2,500	
	B		2,550	3,200	
	C		500	4,150	

Improvement of drain means clearing, excavating and widening in some cases.

Summary of the Main Rivers (cf. Fig. 6)

Name of River	Range	Length of New Short Cut	Length of River to be Improved	(A) Existing River Discharging Capacity	(B) Required Discharge	(B-A) Shortage of Discharging Capacity
The Uquire	Upstream from (3)	900m	- m	37.0 m^3/s	7.1m^3/s	- m^3/s
	(3) - (4)	1,000	-	40.8	≑10	-
	(4) - (5)	600	-	43.7	≑13	-
	(5) - (9)	150	-	43.7	14.5	-
	Total Length	2,650	-			
The Arena	Upstream from (6)	650	100	18.1	11.5	-
	(6) - (7)	550	400	18.6	≑12	-
	(7) - (8)	300	900	17.7	≑13	-
	(8) - (9)	1,150	950	17.7	14.3	-
	Total Length	2,650	2,350			
The Chandernagore	(9) - the mouth	2,000	5,000	24.4	37.0	12.6
	Total Length	2,000	5,000			

The discharges calculated on condition that the flood within a day can be allowed are shown as the discharge required for the rivers.

at (3) on the Uquire River (cf. 3) -vi)) $\dfrac{610{,}000m^3}{24 \times 60 \times 60} \fallingdotseq 7.1\ m^3/s$

at (9) on the Uquire River $\dfrac{1{,}254{,}700\ m^3}{24 \times 60 \times 60} \fallingdotseq 14.5m^3/s$

8) Estimate of Cost for Works

(i) For drains

a) Cross sectional area: $A = (b+h')\,h'$

Type A: $b = 1.5m$

$h = 0.75m \quad \therefore\ h' = 1.00m \quad A = (1.5 + 1.0) \times 1.0 = 2.50m^2$

Type B:	b = 2.00m		
	h = 1.00m	∴ h' = 1.30m	A = (2.0 + 1.3) x 1.3 = 4.29m^2
Type C:	b = 3.00m		
	h = 1.00m	∴ h' = 1.30m	A = (3.0 + 1.3) x 1.3 = 5.59m^2

b) Length of side slopes L = 2 x 1.41h'

Type A: L = 2.82 x 1.0 = 2.82m

Type B: L = 2.82 x 1.3 = 3.67m

Type C: L = 2.82 x 1.3 = 3.67m

c) Earth Volume for cutting (New short cut)

	(L)	(A)	(V)
Type A:	250m	x 2.50m^2	= 625m^3
Type B:	2,550m	x 4.29m^2	= 10,940m^3
Type C:	500m	x 5.59m^2	= 2,795m^3
	Total		14,360m^3

L: total length of drains

Drains to be improved:

Assuming that the base is to be dredged 0.3m in depth.

	(b)	(d)	(A)	(L)	(A)	(V)
Type A:	1.50m	x 0.3m	= 0.45m^2	2,500m	x 0.45m^2	= 1,125m^3
Type B:	2.00m	x 0.3m	= 0.60m^2	3,200m	x 0.60m^2	= 1,920m^3
Type C:	3.00m	x 0.3m	= 0.90m^2	4,150m	x 0.90m^2	= 3,735m^3
			Total			6,780m^3

d) Area of side slopes to be cleared

	(Length of slope)	(Length of Dr.)	(Area)
Type A:	2.82m x	2,500m =	7,050m^2
Type B:	3.67m x	3,200m =	11,744m^2
Type C:	3.67m x	4,150m =	15,231m^2
		Total	34,025m^2

e) Estimate of cost for drains

New short cuts	
For cutting	14,360m^3 x \$2.88/m^3 = \$41,357.-
Drains to be improved	
For dredging	6,780m^3 x \$2.88/m^3 = \$19,526
For clearing	34,025m^2 x 14.4¢/m^2 = \$ 4,900.-
Total	\$65,783.-

(For cutting \$2.20/yd^3 = \$2.88/m^3)

(For clearing 12¢ /yd^2 = 14.4¢/m^2)

(ii) For Main Rivers

a) The Uquire River

. Total length of new short cut = 2,650m

Mean required sectional area = $20m^2$ (cf. Table on P. 9)

Volume of cut = 2,650m x $20m^2$ = $53,000m^3$

b) The Arena River

. Total length of new short cut = 2,650m

Mean required sectional area = 20 m^2

Volume of cut = 2,650m x $20m^2$ = $53,000m^3$

∴ Total length of river to be improved = 2,350m

Assuming that the river-bed is to be dredged 0.3m in depth, cross sectional area to be dredged will be,

6m x 0.3m = $1.8m^2$

(mean width of the river bed to be dredged is 6.0m)

∴ Volume of dredging = 2,350m x $1.8m^2$ = $4,230m^3$

. Total area of side slopes to be cleared will be,

7m x 2,350m = $16,450m^2$

(mean length of side slopes is 7.0m)

c) The Chandernagore River

. Total length of new short cut = 2,000m

Mean required sectional area = $40m^2$ (cf. table on P. 13)

Volume of cut = 2,000m x $40m^2$ = $80,000m^3$

. Total length of river to be improved = 5,000m

Volume of cut = 5,000m x $15m^2$ = $75,000m^3$

(mean sectional area requried for widening is $15m^2$)

In this case clearing is not considered, because it will be able to be carried out at the same time with the widening operations.

d) Estimate of cost for main rivers

. The Uquire River(8¢ $/yd^3$ by dragline = 10.5¢ $/m^3$)

For new short cut $53,000m^3$ x 10.5¢ $/m^3$ = \$5,565.-

. The Arena River

For new short cut	$53,000m^3$ x 10.5¢ $/m^3$	= \$5,565.-
For dredging	$4,230m^3$ x 10.5¢ $/m^3$	= \$ 444.-
For clearing	$16,450m^2$ x 14.4¢ $/m^2$	= \$2,369.-

. The Chandernagore River

For new short cut	$80,000m^3$ x 10.5¢ $/m^3$	= \$8,400.-
For widening	$75,000m^3$ x 10.5¢ $/m^3$	= \$7,875.-
	Total	\$30,221.-

(iii) Grand Total of estimated costs

For Drains	$65,783.-	
For Main Rivers	$30,221.-	
Grand Total	$96,004.-	≒ $96,000

9) Conclusion

Widening and straightening the Chandernagore River is most important for drainage scheme for Carlsen Field.

This must first be undertaken.

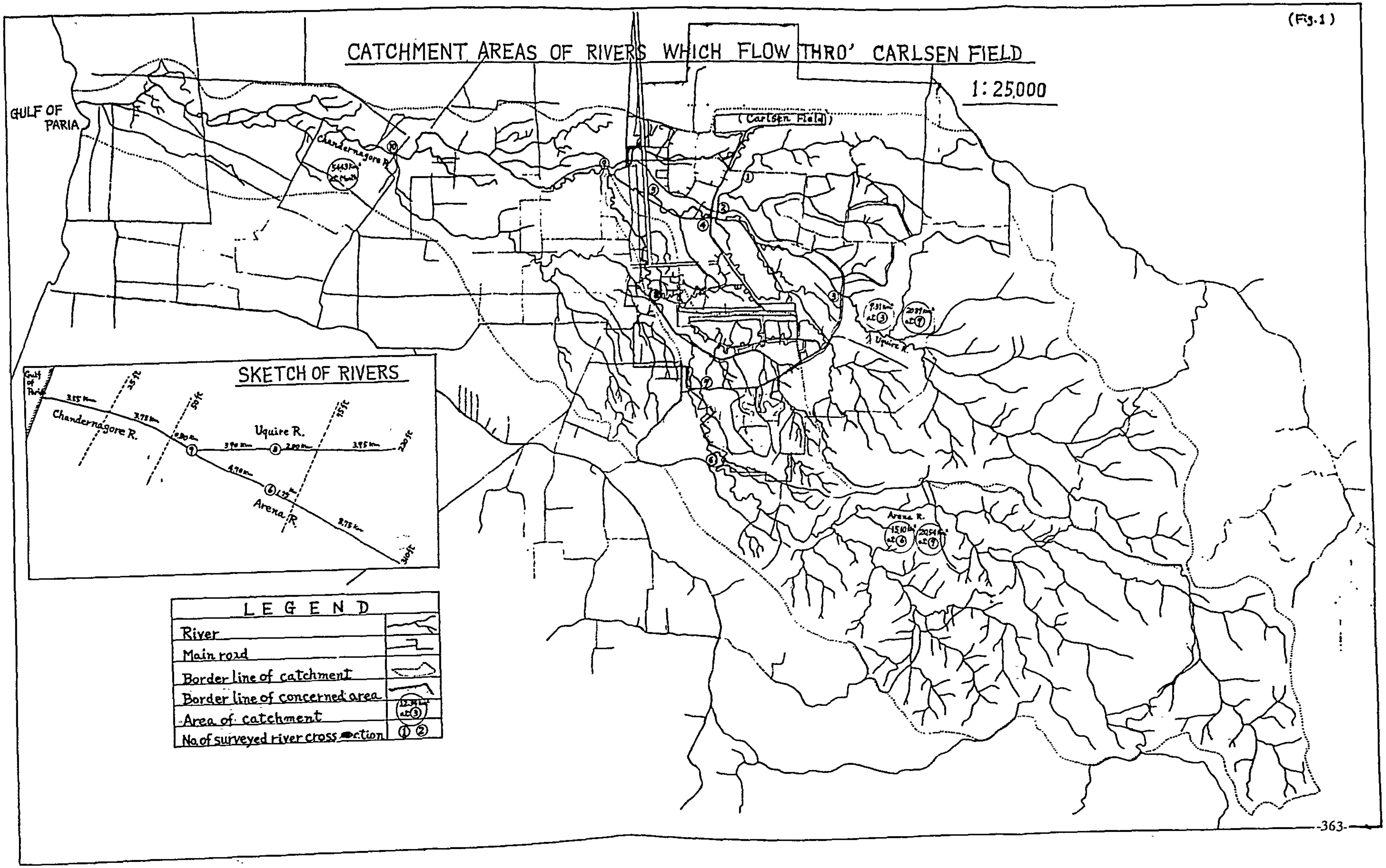
Although the Uquire River and the Arena River have sufficient cross sectional area to discharge the flood, within a day which should occur once in ten years, it is recommended to make them straight, deep and clear on the slopes of their banks.

As to their tributaries, (they were mentioned as drains above) building new short cuts, dredging and clearing on the slopes of their banks by manual labour.

In any case, the drainage works must be carried out from the lower reaches of the Chandernagore River, to the upper reaches of it and be continued to the Uquire River and the Arena River ..... to their tributaries.

Junichi Kitamura

(Fig.1)

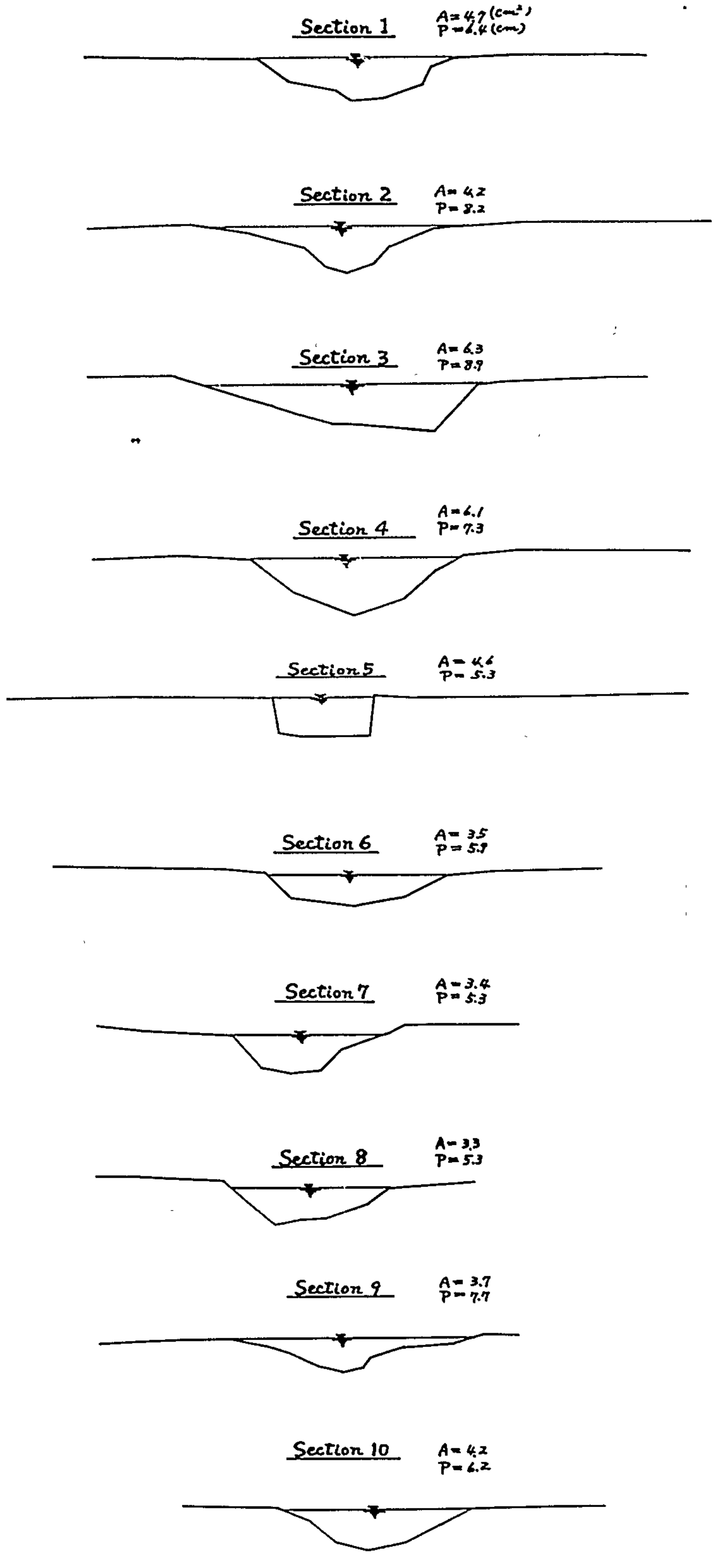


CATCHMENT AREAS OF RIVERS WHICH FLOW THRO' CARLSEN FIELD
1: 25,000
GULF OF PARIA
(Carlsen Field)
Chandernagore R.
Uquire R.
Arena R.
SKETCH OF RIVERS
Gulf of Paria
Chandernagore R.
Uquire R.
Arena R.
25 ft
50 ft
75 ft
220 ft
310 ft
LEGEND
River
Main road
Border line of catchment
Border line of concerned area
Area of catchment
No. of surveyed river cross section

(Fig. 2)

CROSS SECTIONS OF RIVERS THROUGH CARLSEN FIELD

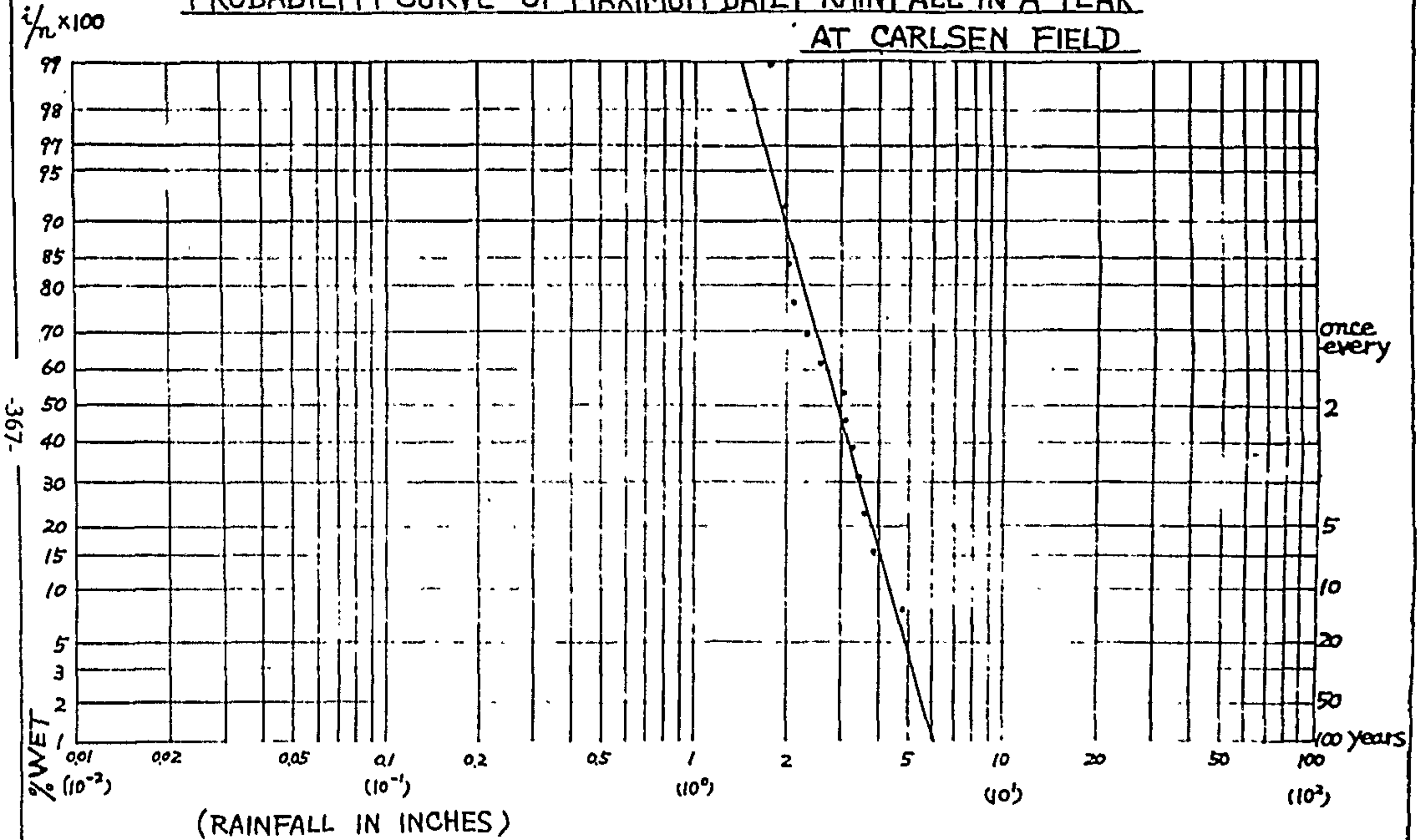
20′ = 1″

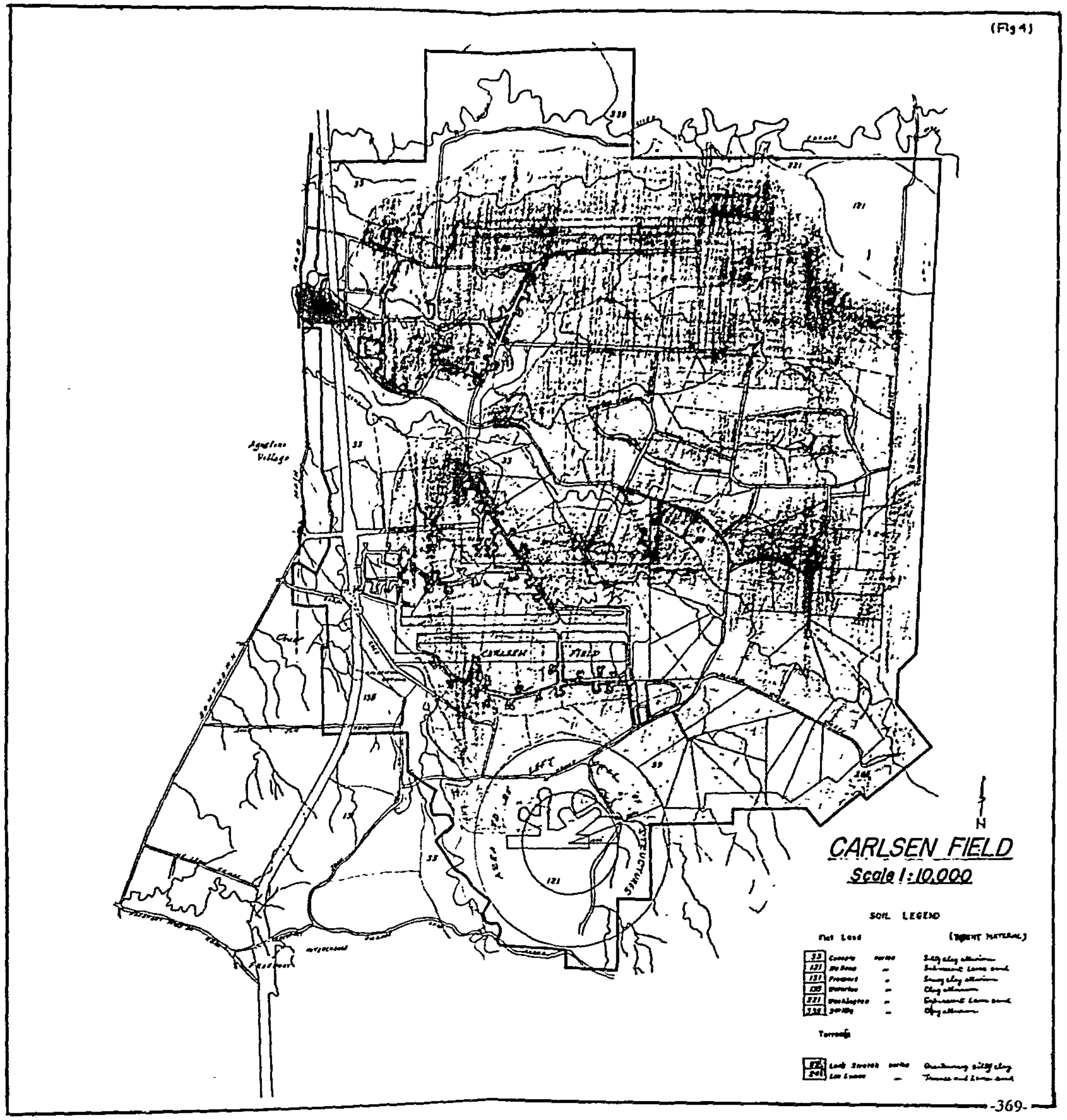


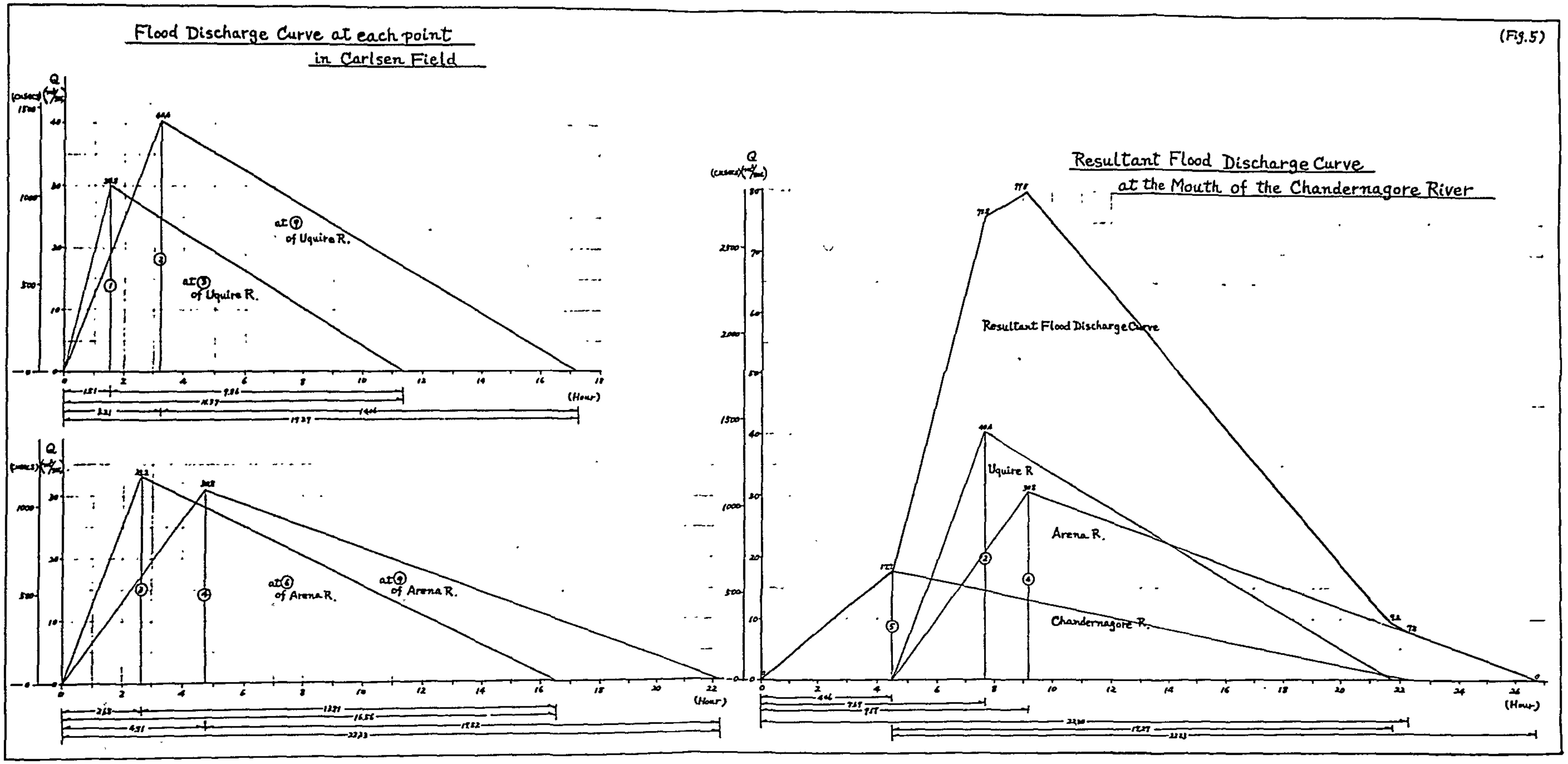
(Fig.3)

BY HAZEN'S METHOD

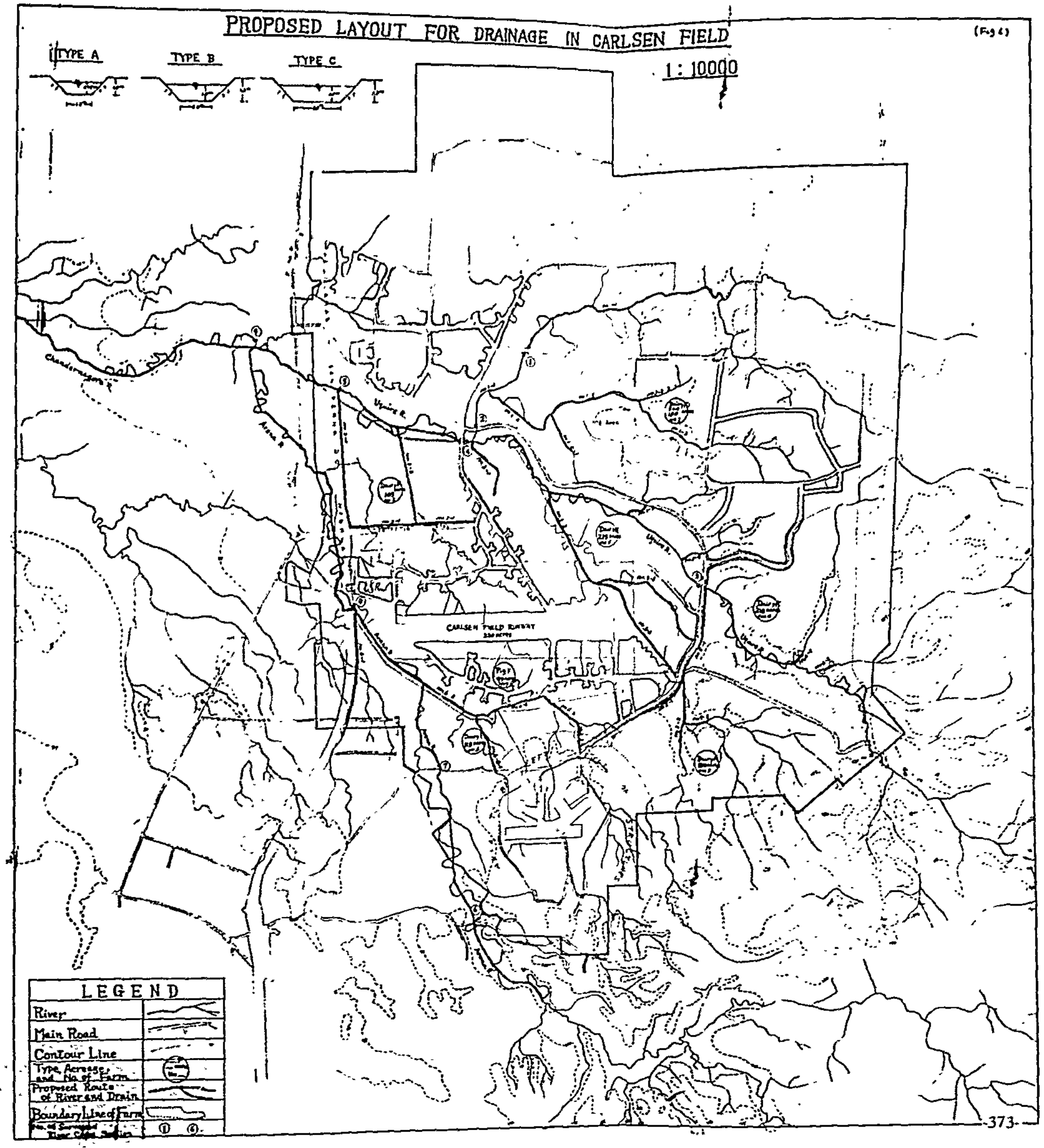
PROBABILITY CURVE OF MAXIMUM DAILY RAINFALL IN A YEAR AT CARLSEN FIELD




(Fig 4)
CARLSEN FIELD
Scale 1:10,000
SOIL LEGEND
CARLSEN
FIELD
N

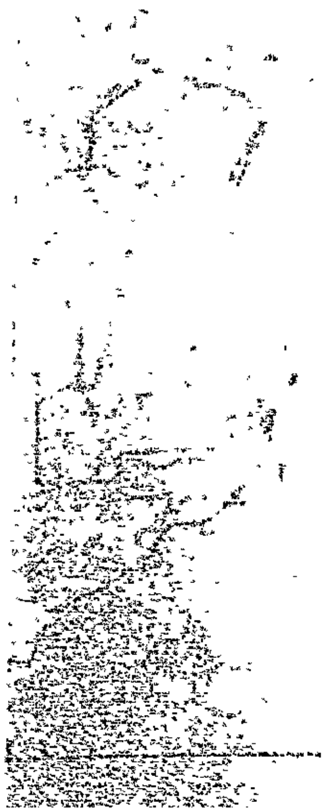


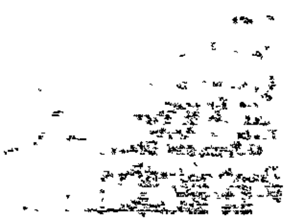
Flood Discharge Curve at each point
in Carlsen Field
(Fig.5)
Resultant Flood Discharge Curve
at the Mouth of the Chandernagore River
at ① of Uquire R.
at ③ of Uquire R.
at ⑥ of Arena R.
at ⑨ of Arena R.
Resultant Flood Discharge Curve
Uquire R
Arena R.
Chandernagore R.
(Hour)



PROPOSED LAYOUT FOR DRAINAGE IN CARLSEN FIELD
(Fig 4)
TYPE A
TYPE B
TYPE C
1 : 10000
CARLSEN FIELD RUNWAY
LEGEND
River
Main Road
Contour Line
Type, Acreage, and No. of Farm
Proposed Route of River and Drain
Boundary Line of Farm

[illegible]





(6) ESMERALDA November 1968

MOVEMENT

8, Oct.	Visited a branch office of the Ministry of Agriculture in Esmeralda to investigate the present condition with Mr. J. V. Smith, Drainage Superintendent (north) and was taken on tour by Mr. Cooper, Agricultural Assistant - To El Parizio Estate, Dundarg Estate and Guaracara Estate.
5, Nov.	Visited there again to give a surveyor the instruction about river sections to be measured.

INTRODUCTION

For purpose of this study, the following is quoted from the letter from the Ministry of Planning and Development, dated February 29th in 1968;

'At a recent meeting of the Crown Lands Development Programme Committee, the view was expressed that a drainage study of the project area was most desirable particularly in view of the drainage problem existing on the farm at present.
It was stressed that corrective measures should be implemented forthwith to solve the matter but that such action would be more effectively carried out if there was some master plan to follow.'

As a matter of urgency, a drainage study of the entire area that is, Waller Field, Carlsen Field and Esmeralda in that order of priority had been requested to be undertaken. According to the priority, the drainage studies for Waller Field and for Carlsen Field were completed in April and in September, respectively.

Therefore, the drainage study for Esmeralda will be carried out in this report.

DESCRIPTION OF AREA STUDIED

Esmeralda consists of three estates, that is El Parizio Estate, Guaracara Estate and Dundarg Estate, in which dairy farms and pig farms were being established by the Ministry of Agriculture since in 1965. The acreage of each Estate is as follows:

El Parizio Estate		127.4 acres
Guaracara Estate		98.0
Dundarg Estate		181.0
	Total	406.4 acres

According to the topographical map of $\frac{1}{62,500}$ with 50 feet contours, which is the only type available, the elevations above sea level are:- El Parizio Estate about 150ft to 300ft. Guaracara Estate about 150ft to 250ft and Dundarg Estate about 110ft to 250ft.

Esmeralda except a part of Dundarg estate has steeper slope than Waller Field and Carlsen Field. It can be called very hilly land.

The existing condition of each estate observed will be stated here.

1) El Parizio Estate

This estate is located in the upper reaches of the Piparo River which is one of the tributaries of the Guaracara River.

Two streams originated from springs in valleys in the north side of Guaracara-Tabaquite Road, flow down across the road to join the Piparo River.

There seems to be no problem about drainage in the area, because of highly sloped land as mentioned above.

The Piparo River has a 5ft dia, pipe at the crossing with one of the main road in the area. It is said that the capacity is enough to discharge flood. The Piparo River, however, goes meandering and is not well-maintained.

2) Guaracara Estate

This estate is located in the upper reaches of the main stream of the Guaracara River and consists of two blocks.

The estate similar to El Parizio Estate is highly inclined which is easily drained.

There is no problem about drainage, either.

3) Dundarg Estate

This estate consists of two blocks which stretch out on both side of Guaracara - Tabaquite Road along the Guaracara River. The higher lands are under the same condition as the other two estates. On the other hand, the lower lands along the road are water-logged and very swampy because there are no drains, although the lands have partly been reclaimed, 3-4 ft high with the earth which was obtained from the hill adjacent to the lands.

The main stream of the Guaracara River through Dundarg Estate was aligned and well-maintained, and was 15-20 ft wide and about 1 ft deep. There are 5 pig farms of 5 acres each on the opposite slope of the hill which is the water shed of the Piparo River and is situated downstream of El Parizio Estate.

The only problem in these pig farms is how to treat slops from the pig farms before their flowing into the Piparo River. Otherwise, the pig farms in the lower reaches will not be able to be supplied with clear water from the river, because the water has been contaminated with slopes from the pig farms in the upper reaches.

The dimensions of the rivers are shown on the following table and Fig. 1.

Location	Catchment Area		Length of River		Difference of Elevation	
	km^2	Sq. miles	Km	Miles	m	ft
At A of Guaracara R.	9.06	3.50	5.63	3.50	36.6 - 268.2	120 - 880
At B of Piparo R.	5.23	2.02	3.50	2.17	35.1 - 187.5	115 - 615
At mouth of Guaracara R.	98.87	38.17	21.25	13.20	0 - 268.2	0 - 880

(Note) A and B are the lowest points on the Guaracara River and the Piparo River respectively in Esmeralda.

(cf. Fig. 1)

Regarding rainfall, there are records of daily rainfall precipitations observed for at least the last ten years at five rain gauge stations near the area.

The maximum daily rainfalls at five stations each year are shown on the following table.

Year	Dam Site Navet	El Salvador	San Antonio	Mayo Quarry	Guaracara Reservoir
1958	2.09 inches	2.83 inches	2.83 inches	2.98 inches	3.36 inches
59	2.09	2.12	2.40	2.92	1.98
60	3.07	3.14	2.40	2.83	3.72
61	2.04	3.86	4.65	4.42	4.70
62	2.93	3.50	2.41	3.62	3.65
63	2.95	2.60	1.97	2.70	3.39
64	4.02	2.32	4.40	1.93	3.16
65	2.87	3.10	3.06	3.36	2.47
66	3.00	3.75	2.75	2.80	3.29
67	2.36	2.40	3.27	3.39	4.65

Calculation for probability of the maximum daily rainfall was carried out by these data. The result is shown on the following table on the next page. (cf. Fig. 2)

i	i/n x 100	Maximum daily rainfall				
		Dam site Navet	El Salvador	San Antonio	Mayo Quarry	Guaracara Reservoir
1	10.00	4.02 inches	3.86 inches	4.65 inches	4.42 inches	4.70 inches
2	20.00	3.07	3.75	4.40	3.62	4.65
3	30.00	3.00	3.50	3.27	3.39	3.72
4	40.00	2.95	3.14	3.06	3.36	3.65
5	50.00	2.93	3.10	2.83	2.98	3.39
6	60.00	2.87	2.83	2.75	2.92	3.36
7	70.00	2.36	2.60	2.41	2.83	3.29
8	80.00	2.09	2.40	2.40	2.80	3.16
9	90.00	2.09	2.32	2.40	2.70	2.47
10	100.00	2.04	2.12	1.97	1.93	1.98

n = 10

Probability	Maximum daily rainfall				
	Dam site Navet	El Salvador	San Antonio	Mayo Quarry	Guaracara Reservoir
Once in 2 years	2.80'	2.95'	3.00'	3.25'	3.60'
Once in 5 years	3.40	3.55	3.85	3.80	4.35
Once in 10 years	3.85	3.95	4.40	4.10	4.80
Once in 20 years	4.10	4.25	4.80	4.40	5.25
Once in 50 years	4.55	4.65	5.40	4.75	5.85
Once in 100 years	4.85	4.90	5.90	5.00	6.15

It can be regarded that the maximum daily rainfall in Esmeralda once in ten years exists between 3.85' calculated as the one at Dam site, Navet and 4.80' calculated as one at Guaracara Reservoir.

The existing geological condition cannot be stated here in details because of no geological map for Esmeralda in the Ministry of Agriculture.

According to the lithological map of Trinidad, calcareous clay is distributed on the north and the west of Guaracara-Tabaquite Road near Esmeralda, and yellow limestone is partially distributed on the north of that road. On the other hand, sandy, shale and sandy alluvium are distributed on the south and east sides.

MASTER PLAN OF DRAINAGE

1) Introduction

Designed flood discharge at A and B (cf. Fig 1) which are the lowest points on the Guaracara River and the Piparo River respectively in Esmeralda, will first be calculated.

Then, these designed flood discharges will be compared with the river discharging capacities at the points.

The drainage study of the respective blocks will successively be done.

2) Calculation of designed flood discharge

i) Determination of the designed basic rainfall precipitation (R). The maximum daily rainfall which occurs once in ten years at Guaracara Reservoir is adopted as the one which represents the intensity in whole catchment areas at A and B.

(i.e.) $R = 4.80'$

ii) Determination of the arrival time of flood from the upmost to each point (T) by Rziha's formula.

Velocity of flood: $V = 72(H/L)^{0.6}$ (Km/hr)

H: Difference of river-bed level between the upmost and the point
L: Length of river from the upmost to the point
T: Arrival time of flood from the upmost $T = L/V$ (hr)

	Location	H	L	H/L	$(H/L)^{0.6}$	V	T
1	at A of Guaracara R.	231.6 m	5,630 m	0.04114	0.14742	10.61 km/hr	0.53 hr
2	at B of Piparo R.	152.4	3,500	0.04354	0.15252	10.98	0.32

iii) Determination of the mean hourly rainfall intensity (r) during T

$$R = 4.80" = 121.9 \text{ mm}$$

$$r = \frac{R}{24}\left(\frac{24}{T}\right)^{2/3}$$

In case that T is less than 1.0 hr, 1 0 hr must be used for this calculation instead of T.

	Location	R/24(mm)	24/T	$(24/T)^{2/3}$	r (mm)
1	At A of Guaracara R.	5.08	24.0	8.32	42.3
2	At B of Piparo R.	5.08	24.0	8.32	42.3

iv) Determination of the maximum flood discharge (Qmax.)

$$Q_{max.} = \frac{1}{3.6} = f.r.A$$

f: run-off coefficient flood peak
A: catchment area (km^2)

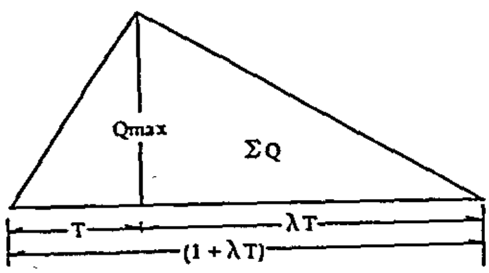
	Location	$\frac{1}{3.6}$	f	r (mm)	A (km^2)	Qmax.(m^3/s)
1	At A of Guaracara R.	0.2778	0.50	42.3	9.06	53.2
2	At B of Piparo R.	0.2778	0.50	42.3	5.23	30.7

v) Determination of the discharging time after flood peak (λT)

$$\Sigma Q = 1{,}000 R.f.A. = ½ Q_{max.} (1 + \lambda)\, T.3{,}300$$

$$\therefore \lambda = \frac{R.f'.A}{1.8 Q_{max}} - 1$$

f': total run-off coefficient in a flood



	Location	R (mm)	f	A(km^2)	RfA	T (hr)	Qmax (m^3/s)	1.8T Qmax	$\frac{RfA}{1.8TQmax}$	λ	λT(hr)
1	At A of Guaracara R.	121.9	0.70	9.06	773.1	0.53	53.2	50.8	15.22	14.22	7.54
2	At B of Piparo R.	121.9	0.70	5.23	446.3	0.32	30.7	17.7	25.21	24.21	7.75

vi) Determination of the total flood discharge (ΣQ)

$$Q = 1{,}000\ RfA$$

	Location	RfA	ΣQ(m^3)
1	At A of Guaracara R.	773.1	773,100
2	At B of Piparo R.	446.3	446,300

3) Calculation of river discharging capacity

Five cross sections, three on the Guarancara River and two on the Piparo River were measured as shown on Fig. 3.

River discharging capacities at A and B can be calculated on the basis of the result of the survey and by assuming the river-bed slope by the assistance of the map of 1/62,500 and by assuming coefficient of roughness with the existing river condition observed.

Section	Name of River	Measured area (cm^2)	Actual area (A) (m^2)	Measured wetted perimeter (cm)	Actual wetted perimeter (P) (m)
At A	Guaracara R	7.35	10.58	7.55	9.06
At B	Piparo R.	2.80	4.03	5.40	6.48

(By Kutter's N.D. formula) (cf. P. 7)

Hydraulic radius (R) (m)	(R½)	Estimated River slope (I)	Coefficient of roughness (n)	N	D	Velocity (V) (m/s)	Discharge (Q) (m^3/s)
1.168	1.081	$\frac{1}{225}$	0.050	2.890	1.167	1.502	15.89
0.622	0.789	$\frac{1}{210}$	0.050	2.990	1.166	0.951	3.83

4) Comparison between the river discharging capacities calculated in paragraph 3) and the maximum flood discharge calculated in paragraph 2) - iv)

Section	(A) River discharging capacity	(B) Maximum flood discharge	B/A
At A	15.89 (m^3/sec)	53.2 (m^3/sec)	3.35
At B	3.83	30.7	8.02

As a result of comparison between the river discharging capacities and the maximum flood discharge, it was found out that the Guaracara River at A can only discharge about a third of the maximum flood discharge and the Piparo River at B only about an eighth of it. It means that temporary flooding along the both rivers cannot be helped.

Under the present condition, the period of flooding will be calculated as follows:

$$\text{Total flood discharge at A} = 773{,}100\text{m}^3 \quad \text{(cf. 2) - iv)}$$
$$\text{at B} = 446{,}300\text{m}^3$$

Therefore, time required for discharge will be,

$$\text{at A} \qquad \frac{773{,}100}{15.89 \times 60 \times 60} = 13.5 \text{ hr.}$$

$$\text{at B} \qquad \frac{446{,}300}{3.83 \times 60 \times 60} = 32.4 \text{ hr}$$

The necessary river discharging capacity of the Piparo River for discharging the flood within a day (= 24 hrs) will be.

$$\frac{446{,}300}{24 \times 60 \times 60} = 5.17\text{m}^3/\text{s}$$

Therefore additional capacity to be provided for will be,

$$5.17 - 3.83 = 1.34 \text{ m}^3/\text{s}$$

Subsequently the additional cross sectional area of the Piparo River will be,

$$4.03\text{m}^2 \times \frac{1.34}{3.83} = 1.41\text{m}^2 = 1.5\text{m}^2$$

4) Drainage scheme for respective estates

It is difficult to establish a concrete drainage scheme at this stage, because detailed topographical maps are not available according to mapping section of Land and Survey Department. Only the outline of the scheme will be given in this report.

In any case a topographical map of $\frac{1}{10{,}000}$ of $\frac{1}{25{,}000}$ with 25 feet contours will be needed sooner or later.

(1) El Parizio Estate

As mentioned in 'Description of Area Studied', there is no drainage scheme required for this estate in particular.

Partly aligning, widening and deepening the Piparo River which runs through the estate as well as cutlassing the grass and the bush on its slope are very effective.

Two streams which flow down across Guaracara - TAbaquite Road into the Piparo River, overflow and erode the road during flood periods and as a result cause landslips.

New culverts with sufficient discharging capacities will therefore be needed.

(2) Guaracara Estate

There is no drainage scheme required for this area either.

Aligning the Guaracara River and shaping its river section will also be very useful for flood discharge.

(3) Dundarg Estate

This estate is divided into two blocks on both sides of Guaracara - Tabaquite Road - a west block and an east block.

In both blocks main drainage canals along the road and branch canals which meet at right angle with the main ones must be built. A part of the water-logged land has been reclaimed by filling. It is desirable that the other lower land be reclaimed by filling also.

(3-1) Calculation of drainage discharge from each block:

Discharging not only the water in the block but also the water flowing into the block from the upper land must be considered.

$$Q = \frac{A.\ Rn.f.10}{T.\ 3600}$$

Q:	drainage discharge (m^3/sec)	
A:	catchment area (ha)	catchment area of east block AE = 45ha
		catchment area of west block AW = 50ha
Rn:	maximum rainfall in n hours (mm) which is now adopted as 121.9mm in 24 hours (= a day)	
f:	runoff coefficient which is 0.4 to 0.6 in flat land.	
T:	time for drainage which is now adopted as 24 hours	

from the east block: $QE = \frac{45 \times 121.9 \times 0.6 \times 10}{24 \times 3,600} = 0.38m^3/s = 13.4$ cusecs

from the west block: $QW = \frac{50 \times 121.9 \times 0.6 \times 10}{24 \times 3,600} = 0.42m^3/s = 14.8$ cusecs

(3-2) Determination of sections of main and branch drainage canals:

It is difficult to determine the slope of the canals according to the topographical condition at this stage because of no detailed contour-map. Judging from the slope of an existing small canal along the road, the slope of 1/500 can be assumed as the ones of the main drainage canals. Side slopes of the canals can also be adopted as 1 : 1 in this case due to the observed soil type

I:	slope of canal which is $\frac{1}{500}$
n:	Coefficient of roughness which is assumed as 0.030 side slope of canal is adopted as 1 : 1

By Kutter's N.D - formula

$$A = (b_e + h)h$$

$$V = \frac{(N.R.)}{D+R^{1/2}}$$

b: width of canal-bed
h: depth
R: hydraulic radius
N = 2.554 and D = 0.71325
in case of $I = \frac{1}{500}$ and $n = 0.030$

Time of drain	h(m)	b(m)	$A(m^2)$	V(m/s)	$Q(m^3/s)$	Remarks
No. 1	0.5	1.0	0.75	0.627	0.470	for main drain in west block
No. 2	0.5	0.8	0.65	0.598	0.389	for main drain in east block
No. 3	0.5	0.5	0.50	0.546	0.273	for branch canals in both blocks

The location and the other dimensions are shown on Fig. 4 and in the following table.

Block	Name of drain	Length of drain		Type of drain	Remarks
East	E-M.D.	5,150ft	1,570m	No. 2	Total length
	E-1	1,035	315	No. 3	No. 1 = 1,160m = 3,805ft
	E-2	835	255	No. 3	No. 2 = 1,570m = 5,150ft
	E-3	1,085	330	No. 3	No. 3 = 1,660m = 5,445ft
	E-4	935	285	No. 3	
West	W-M.D.	3,805	1,160	No. 1	
	W-1	460	140	No. 3	
	W-2	785	240	No. 3	
	W-3	310	95	No. 3	

ESTIMATE OF THE COST FOR DRAINAGE SCHEME

Calculations for quantity of materials

1-1) Aligning the main rivers, deepening, widening and shaping their river section:

i) The Guaracara River from the crossing with an extension of Guaracara Local Road to the point A near Guaracara-Tabaquite Road L = 3,040m (=9,974ft)

Volume of cut: for aligning and shaping (A) $15m^2$ x (L) 400m = $6,000m^3$ ($7,848 yd^3$)

Area of cutlassing on the slope: (A) $4.3m^2$ x (L) (3,040 - 400)m = $11,350m^2$ = ($13,574yd^2$)

ii) The Piparo River from the crossing with the eastern boundary line of El Parizio Estate to the point B near Guaracara-Tabaquite Road.

$L = 3{,}100m\ (= 10{,}171ft)$

	(A) (L)
Volume of cut: for widening deepening and aligning	$1.50m^2 \times 3{,}100m \fallingdotseq 4{,}650m^3\ (=6{,}082yd^3)$
	(slope) (L)
Area of cutlassing on the slope:	$5.6m \times 3{,}100m = 17{,}360m^2\ (=20{,}762yd^2)$

1-2) Building new drains in Dundarg Estate

Volume of cut:

For Type No. 1
$A = (b + h')h' \qquad L = 1{,}160m$
$b = 1.00m$
$h = 0.50m \quad \therefore\ h' = 0.67m$
$A = (1.00 + 0.67) \times 0.67 = 1.12m^2$
$\therefore\ V = A.L. = 1.12 \times 1{,}160 = 1{,}299m^3$

For Type No. 2
$A = (b+h')h' \qquad L = 1{,}570m$
$b = 0.8m$
$h = 0.5m \quad \therefore\ h' = 0.67m$
$A = (0.80 + 0.67) \times 0.67 = 0.98m^2$
$\therefore\ V = A.L. = 0.98 \times 1{,}570 = 1{,}539m^3$

For Type No. 3
$A = (b+h')h' \qquad L = 1{,}660m$
$b = 0.5m$
$h = 0.5m \quad h' = 0.67m$
$A = (0.50 + 0.67) \times 0.67 = 0.78m^2$
$\therefore\ V = A.L = 0.78 \times 1{,}660 = 1{,}295m^3$

Total $V = 4{,}133m^3\ (= 5{,}406yd^3)$

2) Estimate of the cost

2-1) Main rivers

i) The Guaracara River

For cutting $7{,}848\ yd^3 \times 8¢/yd^3 = \$628$
For cutlassing $13{,}574\ yd^2 \times 12¢/yd^2 = \$1{,}629$ Total \$2,257

ii) The Piparo River

For cutting $6{,}082\ yd^3 \times 8¢/yd^3 = \$487$
For cutlassing $20{,}762\ yd^2 \times 12¢/yd^2 = \$2{,}491$ Total \$2,978

2-2) New drains in Dundarg Estate (\$2.20/yd^3 by manual labour)

For cutting $5{,}406\ yd^3 \times \$2.20/yd^3 = \$11{,}893.$

3) Grand total of estimated costs

For main rivers	
The Guaracara River	$ 2,257
The Piparo River	$ 2,978
For new drains in Dundarg Estate	$11,893
Total	$17,128
Contingency (about 10% of above total)	$ 1,672
Grand total	$18,800 T.T.

CONCLUSION

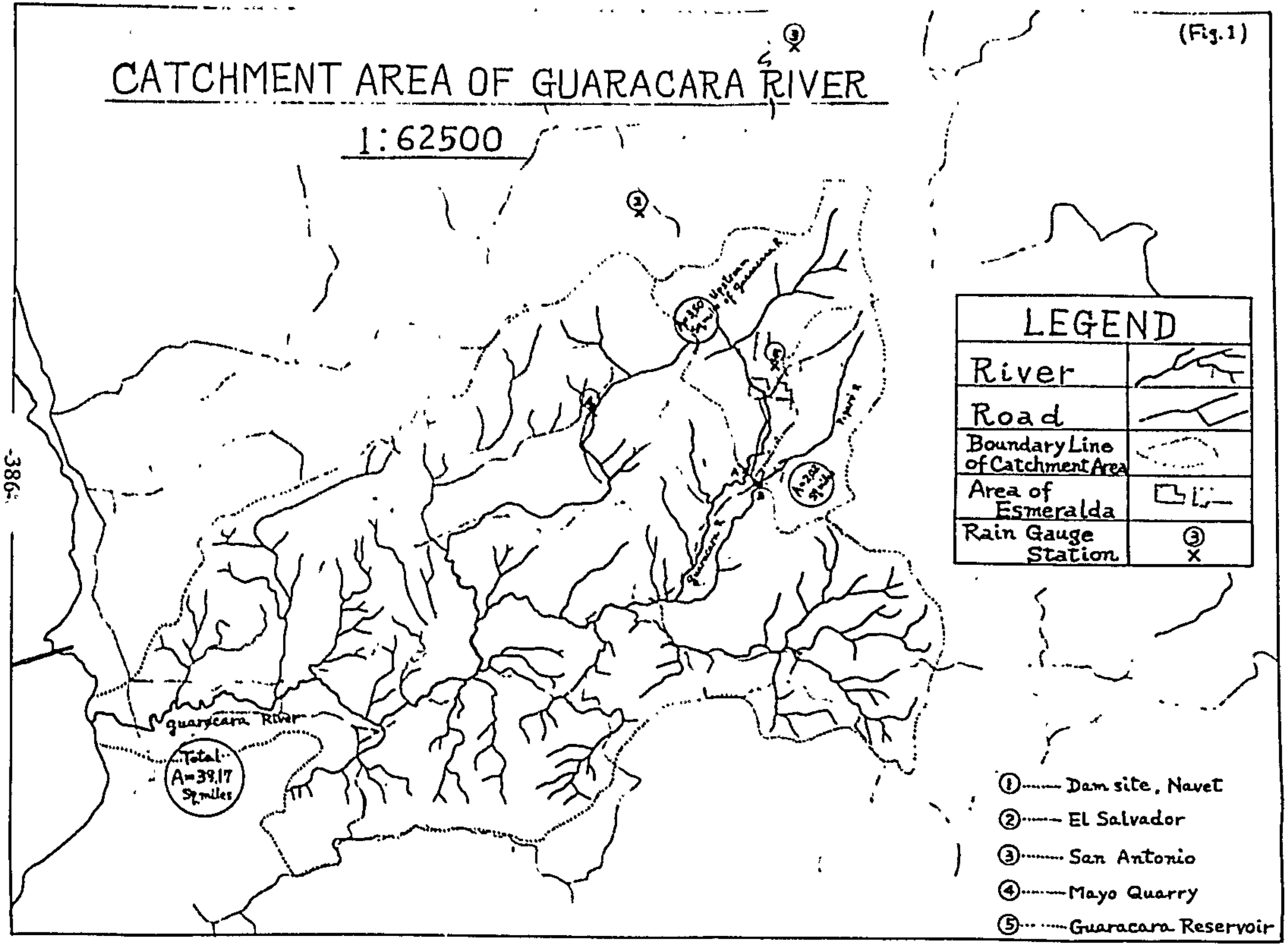
This area, Esmeralda has the least problems about drainage in the three crown lands concerned ... Waller Field, Carlsen Field and Esmeralda.

The condition of the rivers in this area, however, is nearly the same as the others, that is, the river-bed slope is steep, time of concentration is shorter that 1 hour, the river section cannot accomodate the maximum flood discharge.

As a result, flood occurs very often but lasts for a short time.

So as to reduce and prevent the damage due to flood, drains in Dundarg Estate must first be built and main rivers must be partly aligned, widened deepened, shaped and cutlassed on their slopes.

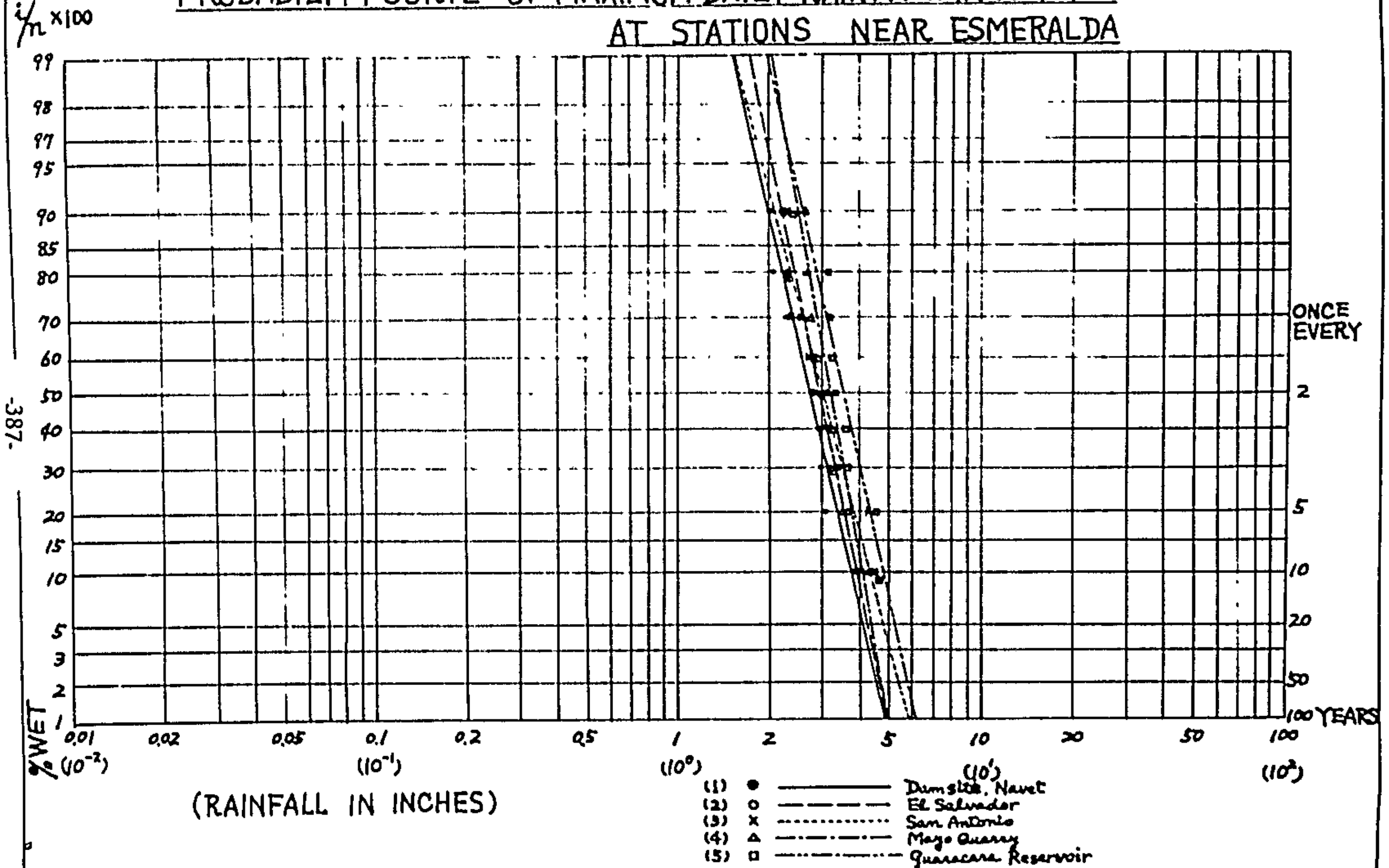
Junichi Kitamura


(Fig.1)
CATCHMENT AREA OF GUARACARA RIVER
1:62500
LEGEND
River
Road
Boundary Line of Catchment Area
Area of Esmeralda
Rain Gauge Station
Guaracara River
Total A=39.17 Sq.miles
① Dam site, Navet
② El Salvador
③ San Antonio
④ Mayo Quarry
⑤ Guaracara Reservoir

(Fig. 2)

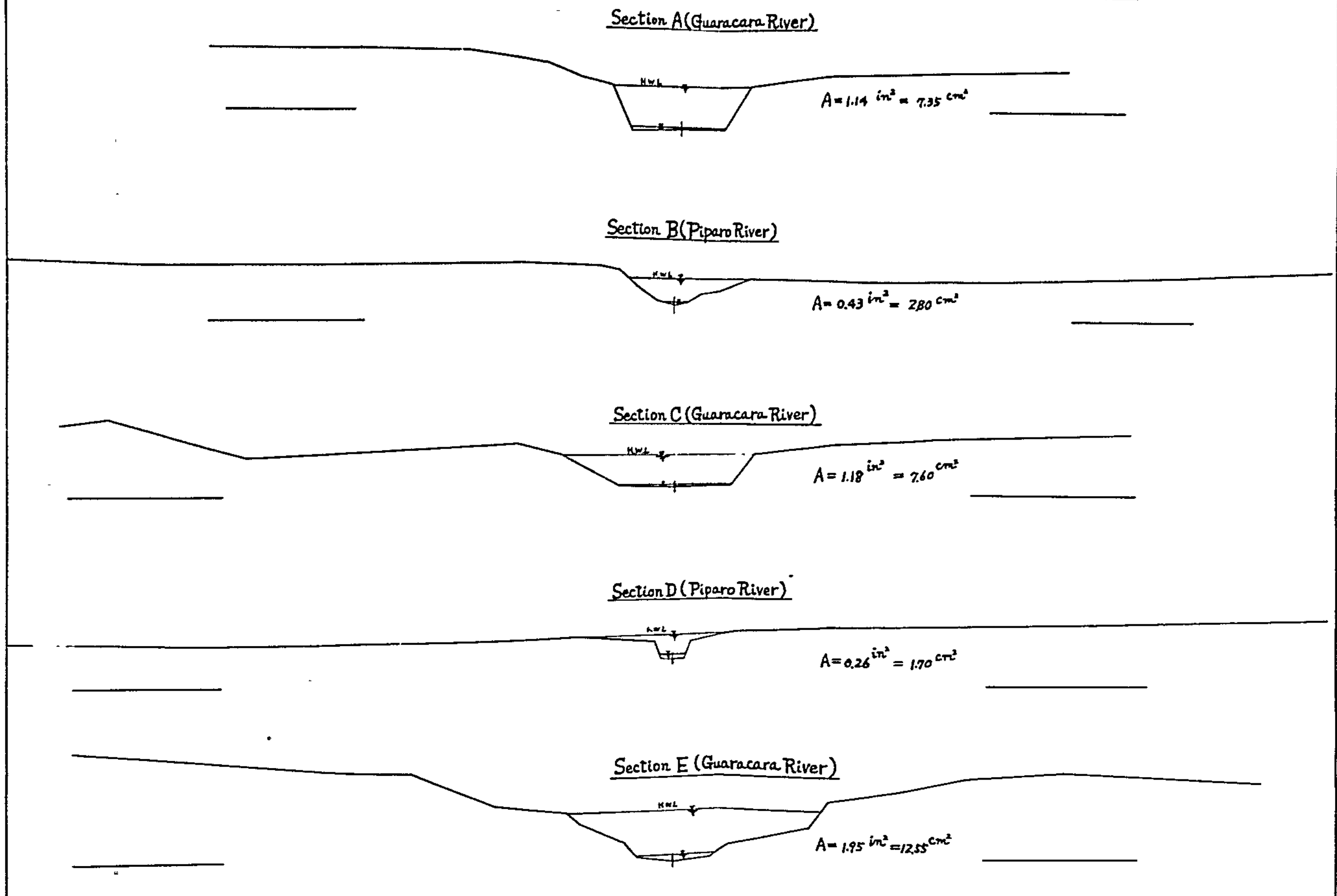
BY HAZEN'S METHOD

PROBABILITY CURVE OF MAXIMUM DAILY RAINFALL IN A YEAR AT STATIONS NEAR ESMERALDA



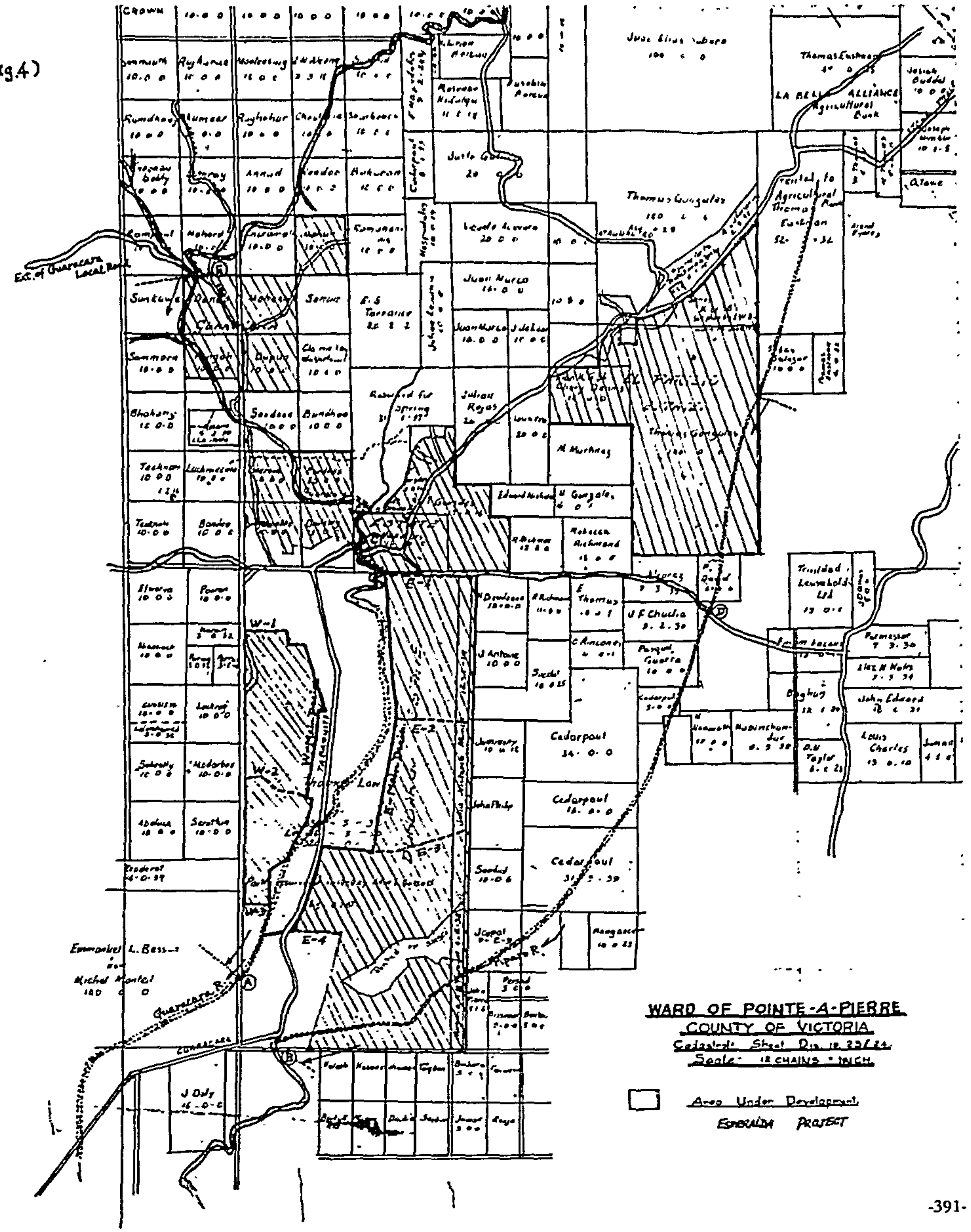
(Fig.3)

CROSS SECTIONS OF MAIN RIVERS IN ESMERALDA (1" = 10')



H.W.Ls were assumed as high as possible.
Cross sectional areas in case of H.W.L.s
were measured with planimeter.

(Fig.4)


WARD OF POINTE-A-PIERRE
COUNTY OF VICTORIA
Scale: 12 CHAINS = 1 INCH
Area Under Development
ESMERALDA PROJECT

[IV] HYDROLOGY

[H Y D R O L O G Y]

By Junichi Kitamura, Lucille Newallo, Sllewyn Snaggs

(1) HYDROMETEOROLOGICAL NETWORK - IN TOBAGO

INTRODUCTION

Official visits to Tobago were paid by Mrs. L. Newallo, Mr. Snaggs officers of the Hydrological Section, Drainage Division and Mr. J. Kitamura, Engineer from Japan, from12th August till 4th September in 1968.

The purpose of these officers' visit are as follows:-

(a) Examine all gauging equipment and accessories; State type of gauge and condition; Name of observer and postal address

(b) Determine geographical location and elevation of ground above M.S.L. (use maps provided).

(c) Collect all rainfall records held by observers which have not been posted to Drainage division.

(d) Investigate new site (as indicated on map) from pluviometric stations.

(e) Determine suitable locations for installing tide gauges on coastline.

(f) Inspect rivers for suitable gauging sites.

The names of stations required are shown below.

(A) Pluviometric Stations

1) Botanic Garden
2) Government Stock Farm
3) Bon Accord Estate
4) Friendship Estate
5) Aukenskeoch
6) Mount Irvine
7) Plymouth
8) Moriah Police Station
9) Grafton Estate
10) Hermitage
11) Hillsborough Dam
12) Parlatuvier
13) The Lure
14) Kendal Estate
15) Louis D'or
16) Charlotteville
17) King's Bay
18) Speyside
19) The Whim

(B) Tide Gauge Locations

1) Scarborough
2) King's Bay
3) Man of War Bay

(C) Stream Gauging Site

1) Gt. Courland River
2) Goldsborough River
3) Louis D'or River

MOVEMENT

12, Aug. (Mon)	Arrival in Tobago Visited Friendship Estate and Government Stock Farm
13, Aug. (Tue)	Visited Rest House and Breeding Unit in Charlotteville and inspected the coastline of Man of War Bay in Charlotteville.
14, Aug. (Wed)	Visited E.C. School in Speyside and Mr. Rosenwald in King's Bay Estate. Inspected the coast line of King's Bay. Visited the Branch Office of Ministry of Agriculture in Louis D'or and got back an automatic gauge for repairs.
15, Aug. (Thu)	Visited the Branch Office in Louis D'or again and investigated the Louis D'or River near the main bridge on Wind-Ward Road. Visited Roxborough Estate, too.
16, Aug. (Fri)	Visited Mr. George in Speyside to install new gauge and revisited Roxborough Estate. Visited Kendal Estate and got back a gauge.
17, Aug. (Sat)	Prepared reports.
18, Aug. (Sun)	(Holiday)
19, Aug. (Mon)	Visited Hillsborough Dam. Got a gauge back from Peggie River Nursery, and set it up at Prime Minister' Quarters in St. George. Inspected the Goldsborough River near the bridge on Windward Road. Visited Trafalgar Estate and took a photograph of the rain gauge. Mr. Watts in the Estate was not interested in the rain gauge. As a result, couldn't go to Hermitage where he owns.

20, Aug (Tue)	Visited Mr. Bishop in Aukenskeoch Estate. Inspected the wharf in Scarborough for tide gauge site. Visited Botanic Garden and Tobago Plantation Ltd.
21, Aug. (Wed)	Visited Aukenskeoch and Tobago Plantation Ltd. again. Visited Bon Accord Estate and Crown Point Air Port.
22, Aug. (Thu)	Visited Grafton Estate and set up a gauge. Visited Mr. De Verteuil in Plymouth and Mrs. Scott in Courland Estate. Inspected the Courland River near the bridge on Plymouth Road. Investigated site to set up a new gauge in the Whim. Visited Franklin Estate. The owner returned the gauge because he is not interested in rainfall observations.
23, Aug. (Fri)	Re-visited Bon Accord Estate. Friendship Estate and Government Stock Farm. Visited Mount Irvine Estate and changed the site because of a possibility of new houses being built near the old site. Visited Mr. R. Johnson in Patience Hill who has been reading his own rain gauge for three years, and West Indian Tobacco Co., Ltd. in Signal Hill. Retrieved a gauge from Chateau. Continued to investigate a new site in the Whim.
24, Aug. (Sat)	Visited Mr. Leacok P.S. and Assistant, Mr. Geyette. Prepared reports.
25, Aug. (Sun)	(Holiday)
26, Aug. (Mon)	Visited Speyside but could not get a boat to Little Tobago. Visited L'anse Fourmi to set up a new gauge in new Methodist School. Checked the Louis D'or River again. Visited Roxborough Estate again. Could not visit the Lure because it was found that the gauge there had been stolen.
27, Aug. (Tue)	Visited the Ministry of Works to collect some data of rainfall but could not collect any. Prepared reports.
28, Aug. (Wed)	Director of Drainage, Mr. C. Taylor arrived in Tobago in the morning and visited Mr. Leacok, the P.S. Visited Scarborough and in particular landslip affected area at Crusoe Hotel, Mount Irvine and surrounding areas, the Courland River to inspect sea erosion and river discharges with Mr. C. Taylor.

29, Aug. (Thu)	Visited Prime Minister's Quarters, Plymouth and Patience Hill with Mr. Taylor to inspect rain gauges and inspect sea erosion in Scarborough.
30, Aug. (Fri)	Visited Pigeon Point and Sea coasts near Scarborough to inspect sea erosion. Could not go to the Whim and Parlatuvier because of car trouble and heavy rain. Met Mr. Coneyette, Senior Engineer in Works Department in Tobago. Mr. C. Taylor returned to Trinidad.
31, Aug. (Sat)	(Independence day) (Holiday). Returned to Trinidad (Mr. Kitamura).
1, Sep. (Sun)	(Holiday) Returned to Trinidad (Mrs. Newallo).
2, Sep. (Mon)	Re-visited Kendal Estate and King's Bay to find new observers and got them.
3, Sep. (Tue)	Re-visited the Whim, but setting new rain-gauge was impossible. Visited Breeding Unit, Runnemede; Castara Methodist School, and Breeding Unit, Parlatuvier.
4, Sep. (Wed)	Returned to Trinidad (Mr. Snaggs).

DESCRIPTION OF PLUVIOMETRIC STATIONS STUDIED

The results of the study for not only required pluviometric stations but also some not required, are shown on attached data.

The names of stations added to study are as follows:

(1) Breeding Unit in Charlotteville.
(2) Roxborough Estate.
(3) Peggie River Nursery, the gauge was returned and re-installed at Prime Minister's Quarters, St. George.
(4) Trafalgar Estate – a gauge could not be returned.
(5) Crown Point Airport.
(6) Courland Estate.
(7) Franklin Estate, a gauge was withdrawn and shipped to Port of Spain.
(8) Patience Hill.
(9) Signal Hill.
(10) Chateau – A gauge was withdrawn and shipped to Port of Spain.
(11) L'anse Fourmi – a gauge was set up.
(12) Runnemede.

On the other hand the names of stations on programme which could not be investigated during the visit are as follows:

(1) Hermitage .... Could not be visited because the estate owner did not permit our visit.
(2) The Lure .... was not visited because of the gauge having been stolen.
(3) The Whim .... A gauge could not be established because of car trouble and the absence of an observer (the school master) whose school was regarded as an only suitable new site.

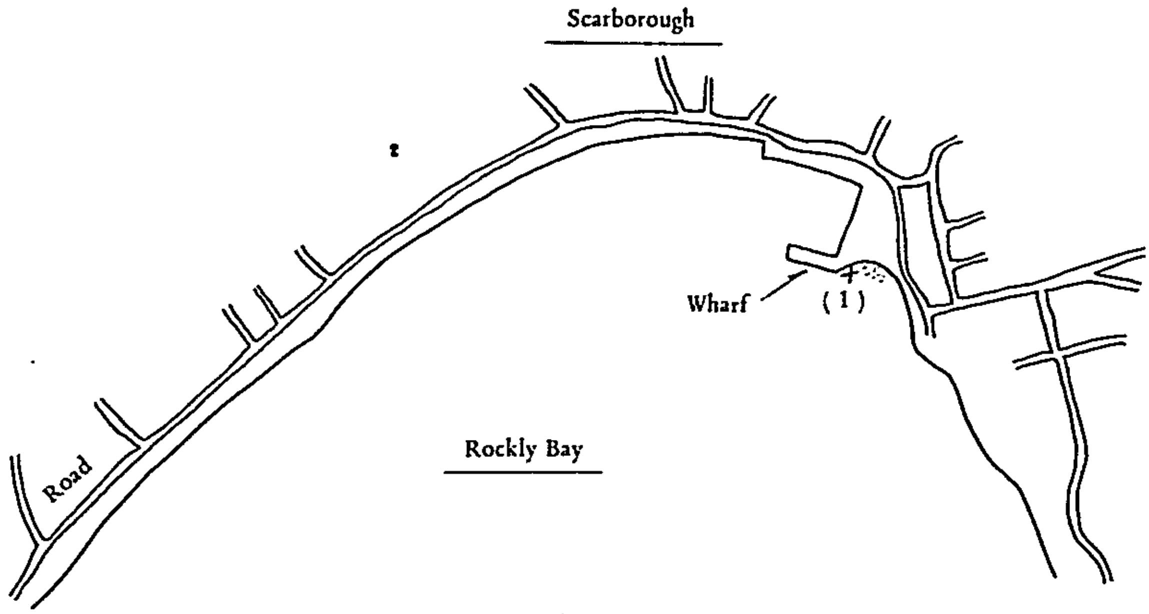
Some owners of estates requested that automatic rain gauges would be set in their estate instead of non-recording type. If an ample budget for the automatic rain-gauges is secured, then they could be set up as soon as possible and they would become very effective and time-saving for observers.

DESCRIPTION OF TIDE GAUGE LOCATIONS STUDIED

(1) Scarborough

As a tide gauge site, for either gauge-board or an automatic tide gauge, the outer side of the wharf is recommended. It is seldom used as a landing place. A jetty about 10 ft. long from the wharf to an automatic tide-gauge site is only needed. It is said that the maximum difference of tidal level is 5 - 6 ft. Although some boulders and rocks used as a base of the wharf, can be seen at the bottom close to the wharf, the bottom of the sea seems to be generally sandy.

It will therefore be not so difficult to set up the gauge, if the wharf is used for the purpose.



(1) : A site taken a photograph

(2) King's Bay

Kings Bay consists of two sandy beaches. Between them there is a rocky point.

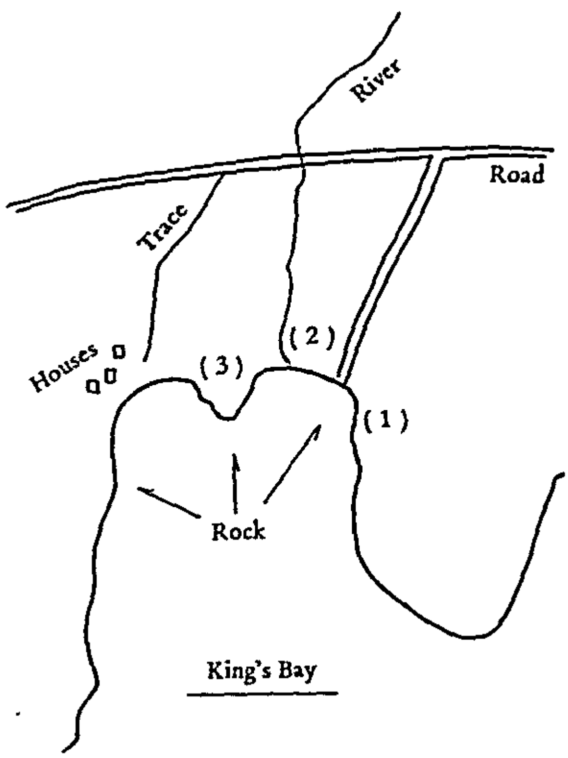
Near the western sandy beach, there are a few houses. As the site to set up the gauge-board, the vicinity of that rocky point is recommended, because gauge-reading on a large rock is available by making short trail to the rock. Piling for the gauge-board seems to be not so difficult and the people who will be asked to read the gauge live near there.

Also in the case of setting up an automatic tide-gauge, by using this rock, the length of the jetty required will be made short (about 20 ft in length will be necessary).

It is said that the maximum difference of tidal level is about 9 feet. Therefore, if a jetty is built from the beach to the automatic gauge-site, the length will be longer than 100 feet.

The nearer a wave comes to the shore, the more amplitude of wave-vibration occurs. So setting up the gauge as far as possible away from the shore is desirable.

On the other hand, the deeper the site chosen, the more expensive the setting up the gauge and building the jetty will be. These things are important in determining the site.



(1), (2) and (3) : Sites taken photographs

(3) Man of War Bay

Charlotteville is situated in the east side of the Bay where there is a mild sandy beach. Inside of the beach is protected by retaining walls, and a main road is running inside along the beach.

At both ends of the village the sandy beach and roads disappear and they turn to a rocky coast near the cliffs.

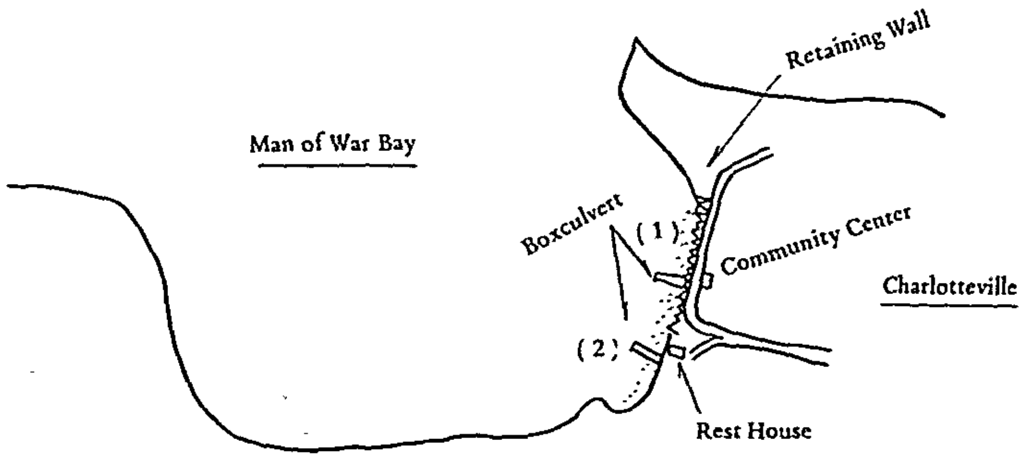
As a tide-gauge site, the sandy beach must therefore be selected in consideration of facilities of traffic and setting up the gauge.

Two box culverts about 5' by 5' which had been used as main drains to discharge water from inland before, run across the sandy beach into the sea. One comes from the front of the Community Center and the other from near the Rest House as a jetty. (Actually the latter one is shown as a jetty on the aerial map.)

It is said that H.H.T.L., is nearly equal to the level of the road which runs along the beach and L.L.T.L. is nearly the same level as the sill of the end of the box culvert. As a result, the maximum difference between then can be found to be 9 feet.

It is therefore suggested that a gauge board for low tide be set 10 feet far away from the end of the box culvert and a gauge-board for high tide be set near the beginning of the box culvert. (In front of the Community Center, the retaining wall itself can be used as a gauge board for high tide by marking graduation.)

So as to set up an automatic tide-gauge a real jetty from the road longer than the box culvert 70 - 80 ft and 7 ft higher than that must be built.



(1) and (2) : Sites taken photographs

DESCRIPTION OF STREAM GAUGING SITES STUDIED

(1) Gt. Courland River

This river is the longest one in Tobago and has also the largest catchment area. Accordingly the flood discharge is also quite much and it is said that the maximum flood stage reached just under the girder of the bridge on Plymouth Road.

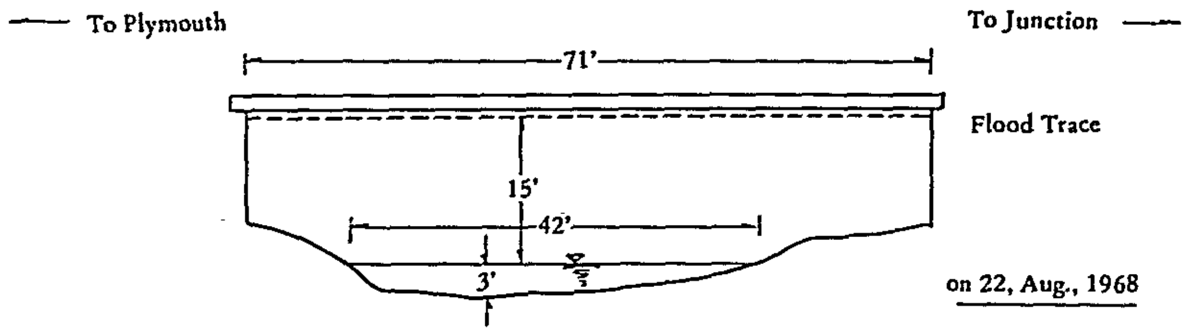
The maximum water depth at that time will therefore be about 17 - 18 ft. Consequently the gauge-board of 20 ft in length is needed for this river.

Although the river flows straight from the point about 300 feet upstream from the bridge to the bridge itself, it has a bend just down the bridge. Setting up both an automatic stream-gauge and gauge-board around the bridge is recommended because of facilities of traffic and gauge-reading, and because it is situated near the sea. (The catchment area at this point is nearly equal to the whole catchment area of this river.)

A connecting bridge 30 ft long from the embankment will be needed for setting up an automatic stream gauge, unless the main bridge is used for the purpose.

The abutments of the bridge are situated too far away from the river stream to use for setting up the gauge. (cf. cross section attached.)

Gt. Courland River Bridge



(2) Goldsborough River

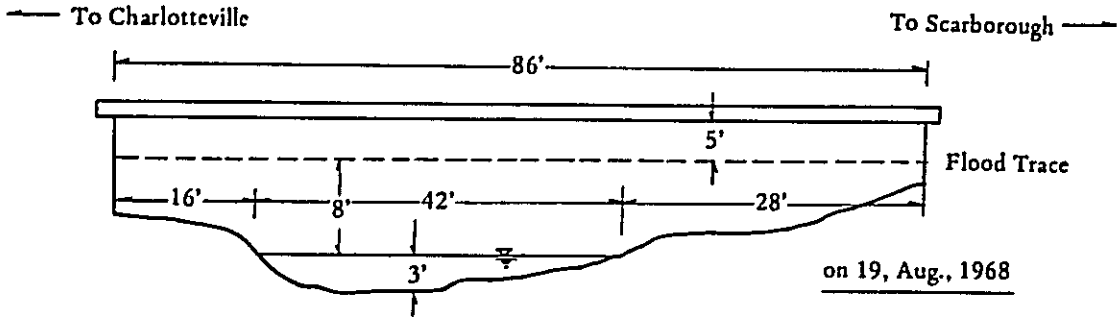
There is no appropriate site for setting up a gauge downstream from the bridge on Windward Road, because of stretching sand bars. The section between the point about 40 ft upstream from the bridge and the bridge itself is only suitable for setting up a gauge.

A gauge-board of 10 ft in length has already been set up at the upstream side of the left abutment by a department of the Ministry, Tobago. However, the gauge-board is left as its bolts cemented into abutment, were taken out by floodwater.

How it has been observed by Tobago Works Dept., must be investigated and checked. As a result of inspecting flood-trace, the maximum water depth in flood is regarded as 8 - 9 ft.

The short connecting bridge from the abutment will be needed for setting up an automatic stream-gauge. (cf. cross section attached.)

Goldsborough River Bridge



(3) Louis D'or River

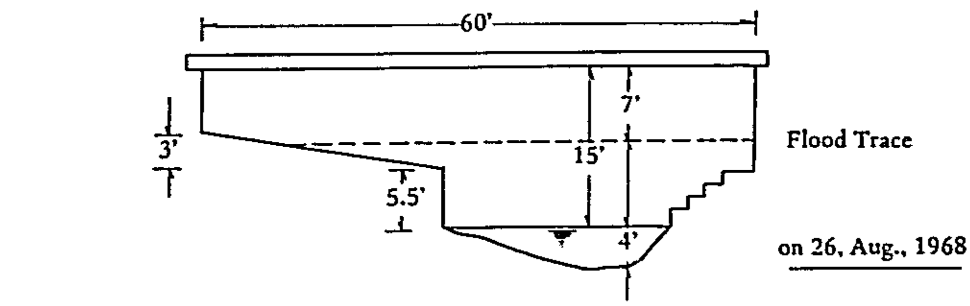
The section between 100 ft upstream from the bridge on Windward Road and 200 ft downstream from the bridge is suitable for setting up a stream gauge.

A gauge-board of 10 ft in length has already been set up at the side of the right abutment by one of the departments of the Ministry, Tobago. How this gauge-board is being observed by the respective department must be investigated and checked.

As a result of inspecting flood-trace, the maximum water depth in flood is about 12 ft.

The short connecting bridge from the abutment will be needed for setting up an automatic stream-gauge. (cf. cross section attached.)

Louis D'or River Bridge



In conclusion, for these three weeks, the most important works mentioned above have been carried out very smoothly with no trouble.

Arrangements and reservations through the Tobago Affairs Ministry, and assistance from its Works Department, are gratefully appreciated.

J. Kitamura, Engineer

L. Newallo, Engineering Assistant I

HYDROMETEOROLOGICAL NETWORK – TOBAGO

Subject: Location of Rain Gauge at Runnemede Breeding Unit – Ministry of Agriculture: Map 2 d

1.	Geographical location:	Latitude: 11° 15' 07.55" Longitude: 60° 42' 35.07"
2.	Elevation:	Approx. 1,050' above M.S.L.
3.	Type of gauge:	5" Dia. Non Recording Rain Gauge manufactured by H.H. & D New Cltham London SE9
4.	Owner of gauge:	Ministry of Works, Drainage Division, Hydrology Section
5.	Observer:	Mr. Raffie Patterson (Ag. Stock Man, Agriculture)
6.	Height of gauge above ground:	1' 8"
7.	Height of nearest obstacle:	6'
8.	Distance from nearest obstacle:	20'
9.	Condition around gauge:	Level position on steeply sloping ground
10.	Duration of record:	January 1948 to date
11.	Time of observation:	

J. Kitamura, Engineer & L. Newallo, Engineering Assistant I

Subject: Location of Rain Gauge at Blenheim House: Map 2 H

1.	Geographical location:	Latitude Longitude) To be determined
2.	Elevation:	
3.	Type of gauge:	5" Dia. Non Recording Rain Gauge manufactured by Casella London No. W5050
4.	Owner of gauge:	Ministry of Works, Drainage Division, Hydrology Section
5.	Observer:	Miss Nellie Warner
6.	Height of gauge above ground:	1' 7½"
7.	Height of nearest obstacle:	Approx. 20' high
8.	Distance from nearest obstacle:	72'
9.	Condition around gauge:	Gently sloping with grass about 2" high
10.	Duration of record:	Aug. 20, 1968
11.	Time of observation:	8:00 a.m. daily

J. Kitamura, Engineer & L. Newallo, Engineering Assistant I

Subject: Location of Rain Gauge at Castara Methodist School – Ministry of Education: Map 2 B

1.	Geographical location:	Latitude: 11° 16' 46.47" N Longitude: 60° 41' 36.57" W
2.	Elevation:	225' above M.S.L.
3.	Type of gauge:	5" Dia. Non Recording Rain Gauge manufactured by Casella, (splayed base)
4.	Owner of gauge:	Ministry of Works, Drainage Division, Hydrology Section
5.	Observer:	Mr. Carlos Sebro
6.	Height of rim above gauge:	9"
7.	Distance from nearest obstacle:	31' 0"
8.	Height of nearest obstacle:	Approx. 30'
9.	Duration of record:	May 1961 to present date
10.	Condition of ground surface around gauge:	Slightly sloping with grass about 4" high
11.	Time of observation:	8:00 a.m. daily

J. Kitamura, Engineer & L. Newallo, Engineering Assistant I

Subject: Location of Rain Gauge at Moriah Police Station – Ministry of Home Affairs: Map 2 O

1.	Geographical location:	Latitude: 11° 14' 25.66" Longitude: 60° 43' 03.83"
2.	Elevation:	Approx. 738' above M.S.L.
3.	Owner of gauge:	Ministry of Works, Drainage Division, Hydrology Section
4.	Type of gauge:	5" Dia. Non Recording manufactured by Casella London No. 5050
5.	Observer:	Corporal Errol Redman
6.	Height of gauge above ground:	4' 0"
7.	Height of nearest obstacle:	Approx. 25'
8.	Distance from nearest obstacle:	20'
9.	Condition around gauge:	Gauge is placed on a wooden stand 2' 4" high on isolated hill.
10.	Duration of record:	June 1949 to present date
11.	Time of observation:	8:00 a.m. daily

J. Kitamura, Engineer & L. Newallo, Engineering Assistant I

Subject: Location of Rain Gauge at Charlotteville Rest House – Charlotteville Tobago: Map 3 B

1.	Geographical location:	Latitude: 11° 19' 01.4" N Longitude: 60° 33' 05.2" W
2.	Elevation:	Approx. 3' above M.S.L.
3.	Type of gauge:	5" Dia. Non Recording manufactured by H.H. & P (T) Ltd. London (splayed base)
4.	Owner of gauge:	Ministry of Works, Drainage Division, Hydrology Section
5.	Observer:	Miss Victoria Collette, Caretaker of Rest House
6.	Height of gauge above ground:	1' 7"
7.	Distance from nearest obstacle:	13'
8.	Height of nearest obstacle:	20' approximately
9.	Condition of ground:	Flat
10.	Duration of record:	1st November 1950 to present date
11.	Time of observation:	8:00 a.m. daily

J. Kitamura, Engineer & L. Newallo, Engineering Assistant I

Subject: Location of Rain Gauge of Parlatuvier Breeding Unit – Ministry Agriculture: Map 2 C

1.	Geographical location:	Latitude: 11° 17' 57.42" N Longitude: 60° 38' 27.76" W
2.	Elevation:	Approx. 319' above M.S.L.
3.	Type of gauge:	5" Dia. Non Recording Rain Gauge manufactured by Casella No. 2600 (Snowdown Pattern)
4.	Owner of gauge:	Ministry of Works, Drainage Division, Hydrology Section
5.	Observer:	Mr. Ebenezer Burris, Field Assistant I Agriculture
6.	Height of rim above ground:	9"
7.	Distance from nearest obstacle:	25'
8.	Height of nearest obstacle:	Approx. 15'
9.	Duration of record:	June 1956 to present date
10.	Condition of ground around gauge:	Steep slope with grass about 3" high
11.	Time of observation:	8:00 a.m. daily

J. Kitamura, Engineer & L. Newallo, Engineering Assistant I

Subject: Location of Rain Gauge at White Rock Estate – Plymouth: Map 1 C

1.	Geographical location:	Latitude: 11° 12' 59.1" N Longitude: 60° 46' 00.6" W
2.	Elevation:	Approx. 113' above M.S.L.
3.	Type of gauge:	5" Non Recording manufactured by Casella London No. W5050 (splayed base)
4.	Owner of gauge:	Ministry of Works, Drainage Division, Hydrology Section
5.	Observer:	Mr. J. de Verteuil
6.	Height of gauge above ground:	1' 8"
7.	Height of nearest obstacle:	35' approximately
8.	Distance from nearest obstacle:	28'
9.	Condition around gauge:	Gauge is placed in discarded oil tins on hilly land.
10.	Duration of record:	Nov. 1953 to present date
11.	Time of observation:	8:00 a.m. daily

J. Kitamura, Engineer & L. Newallo, Engineering Assistant I

Subject: Location of Rain Gauge at Mt. Irvine Estate – Mt. Irvine: Map 1 F

1.	Geographical location:	Latitude: 11° 11' 12.2" N Longitude: 60° 47' 40.5" W
2.	Elevation:	Approx. 75' above M.S.L.
3.	Type of gauge:	5" Dia. Non Recording manufactured by Casella London No. W5050 (splayed base)
4.	Owner of gauge:	Ministry of Works, Drainage Division, Hydrology Section
5.	Observer:	Mr. Ancil Dillon, Manager, David Martineau
6.	Height of gauge above ground:	3' 7"
7.	Height of nearest obstacle:	30'
8.	Distance from nearest obstacle:	200'
9.	Condition around gauge:	Flat with grass about 1" high
10.	Duration of record:	Aug. 1963 to present date
11.	Time of observation:	8:00 a.m. daily

J. Kitamura, Engineer & L. Newallo, Engineering Assistant I

Subject: Location of Rain Gauge at Govt. Stock Farm – Ministry of Agriculture: Map 1 F

1.	Geographical location:	Latitude: 11° 10' 17.5" N Longitude: 60° 45' 13.1" W
2.	Elevation:	Approx. 38' above M.S.L.
3.	Type of gauge:	5" Dia. Non Recording manufactured by Stanley London (splayed base)
4.	Owner of gauge:	Ministry of Works, Drainage Division, Hydrology Section
5.	Height of gauge:	1' 8"
6.	Observer:	Mr. Adolphus Edward, Ag. A. A. III
7.	Height of nearest obstacle:	15' approximately
8.	Distance from nearest obstacle:	25'
9.	Condition around gauge:	Hilly ground
10.	Duration of record:	January 1910 to present date
11.	Time of observation:	8:00 a.m. daily

J. Kitamura, Engineer & L. Newallo, Engineering Assistant I

Subject: Location of Rain Gauge at Met. Station – Crown Point: Map 1 E

1.	Geographical location:	Latitude: 11° 08' 53.5" N Longitude: 60° 50' 25.6" W
2.	Elevation:	Approx. 25' above M.S.L.
3.	Type of gauge:	5" Dia. Non Recording manufactured by Casella London No. W5050 (splayed base)
4.	Observers:	Mr. Edward Jenes and Mr. Theophilus Jackson
5.	Owner of gauge:	Met. Department
6.	Height of rim above ground:	1' 0" gauge is buried in ground.
7.	Distance from nearest obstacle:	127' 0"
8.	Height of nearest obstacle:	Approx. 30' high
9.	Duration of record:	2nd May 1967 to present date
10.	Condition of ground surface around gauge:	Flat, with grass about 15" high

J. Kitamura, Engineer & L. Newallo, Engineering Assistant I

Subject: Location of Rain Gauge at Patience Hill Near Amity Hope Trig Station: Map 1 F

1.	Geographical location:	Latitude: 11° 11' 18.9" N Longitude: 60° 45' 58.7" W
2.	Elevation:	Approx. 600' above M.S.L.
3.	Type of gauge:	5" Dia. Non Recording manufactured by Negretti Zambra
4.	Owner of gauge:	Private
5.	Height of gauge above ground:	9"
6.	Observer:	Mr. Robert C. Johnson Jr.
7.	Height of nearest obstacle:	20' approximately
8.	Distance from nearest obstacle:	51"
9.	Condition around gauge:	Gauge buried on well kept lawn
10.	Duration of record:	1st August 1966 to date
11.	Time of observation:	8:00 a.m. daily

J. Kitamura, Engineer & L. Newallo, Engineering Assistant I

Subject: Location of Gauge at Gafton Estate: Map 1 F

1.	Geographical location:	Latitude: 11° 11' 41.5" Longitude: 60° 47' 14.1"
2.	Elevation:	Approx. 187' above M.S.L.
3.	Type of gauge:	5" Dia. Non Recording Rain Gauge manufactured by Stanley London (splayed base)
4.	Owner of gauge:	Ministry of Works, Drainage Division, Hydrology Section
5.	Observer:	Mrs. Eleanor Alefounder
6.	Height of gauge above around:	1' 8"
7.	Height of nearest obstacle:	30' approx.
8.	Distance from nearest obstacle:	60'
9.	Condition around gauge	Flat with grass about 8" high
10.	Duration of record:	16th July 1950 to 12th October 1957 records broken re-established.

J. Kitamura, Engineer & L. Newallo, Engineering Assistant I

Subject: Location of Rain Gauge at Kings Bay Estate: Map 3 H

1.	Geographical location:	Latitude: 11° 16' 02.68" Longitude: 60° 32' 49.80"
2.	Elevation:	Approx. 20' above M.S.L.
3.	Type of gauge:	5" Dia. Non Recording Rain Gauge manufactured by Stanley London (splayed base)
4.	Owner of gauge:	Ministry of Works, Drainage Division, Hydrology Section
5.	Observer: Manager: Owner:	Mr. Folton Waldron (Overseas) Mr. Eurias Colestine James B. Rosenwald
6.	Height of rim above ground:	1' 7"
7.	Distance from nearest obstacle:	76'
8.	Height of nearest obstacle:	Approx. 25' high
9.	Duration of record:	January 1910 to date
10.	Condition of ground surface around gauge:	Flat concrete base of demolished house
11.	Time of observation:	8:00 a.m. daily

J. Kitamura, Engineer & L. Newallo, Engineering Assistant I

Subject: Location of Rain Gauge at Bon Accord Estate: Map 1 E

1.	Geographical location:	Latitude: 10° 09' 21" N Longitude: 60° 49' 24.9" W
2.	Elevation:	Approx. 20' above M.S.L.
3.	Type of gauge:	5" Dia. Non Recording manufactured by Casella London No.W5050 (splayed base)
4.	Owner of gauge:	Ministry of Works, Drainage Division, Hydrology Section
5.	Observer:	Codrington James
6.	Height of rim above	2' 6"
7.	Distance from nearest obstacle:	10'
8.	Height of nearest obstacle:	Approx. 25'
9.	Duration of record:	Broken record from 1963 - 1967 re-established.
10.	Condition of ground surface around gauge:	Flat with grass about 6" high

J. Kitamura, Engineer & L. Newallo, Engineering Assistant I

Subject: Location of Rain Gauge at West Indian Tobacco Ltd., White Lodge Estate, Signal Hill: Map 1 F

1.	Geographical location:	Latitude: 11° 10' 37.9" N Longitude: 60° 45' 34.9" W
2.	Elevation:	Approx. 186' above M.S.L.
3.	Type of gauge:	5" Dia. Non Recording manufactured by Casella London No. 2069
4.	Owner of gauge:	West Indian Tobacco Ltd.
5.	Observer:	Mr. Woodsdale Nero, Station Supt.
6.	Height of gauge above ground:	3' 2"
7.	Height of nearest obstacle:	40'
8.	Distance from nearest obstacle:	56'
9.	Condition around gauge:	Gauge is placed on concrete stand on well kept lawn.
10.	Duration of record:	September 1963 to present date
11.	Time of observation:	8:00 a.m. daily

J. Kitamura, Engineer & L. Newallo, Engineering Assistant I

Subject: Location of Rain Gauge Roxborough Estate, Roxborough, Tobago: Map 3 D

1.	Geographical location:	Latitude: 11° 15' 01.1" N Longitude: 60° 35' 00.2" W
2.	Elevation:	Approx. 25' above M.S.L.
3.	Type of gauge:	5" Dia. Non Recording manufactured by Casella London (splayed base)
4.	Owner of gauge:	Ministry of Works, Drainage Division, Hydrology Section
5.	Observer:	Manager: Mr. Easton Humphrey
6.	Height of gauge above ground:	3' 3"
7.	Distance from nearest obstacle:	40' approximately
8.	Height of nearest obstacle:	30'
9.	Condition around gauge:	Gauge placed on concrete wall 1.7¾ high and 2' 7" thick by 49' long
10.	Duration of record:	October 1958 to present date but broken in 1962 and 1963.
11.	Time of observation:	8:00 a.m. daily

J. Kitamura, Engineer & L. Newallo, Engineering Assistant I

Subject: Location of Rain GAuge at Louis D'or Demonstration Station

1.	Geographical location:	Latitude: 11° 15' 03.37" Longitude: 60° 33' 59.10"
2.	Elevation:	Approx. 70' above M.S.L.
3.	Type of gauge:	5" Dia. Non Recording Rain Gauge manufactured by Casella London No. W5050
4.	Owner of gauge:	Ministry of Works, Drainage Division, Hydrology Section
5.	Observer: Senior Officer:	Mr. Benedict Job Kelvin Polish, A. A. III
6.	Height of rim above ground:	7"
7.	Height of nearest obstacle:	25'
8.	Distance from nearest obstacle:	40'
9.	Duration of record:	May 1951 to date
10.	Condition of ground around gauge:	Gently sloping
11.	Time of observation:	8:00 a.m. daily

J. Kitamura, Engineer & L. Newallo, Engineering Assistant I

Subject: Location of Rain Gauge at Charlotteville Breeding Unit – Ministry of Agriculture: Map 3 E

1.	Geographical location:	Latitude: 11° 18' 52.5" N Longitude: 60° 33' 08.5" W
2.	Elevation:	Approx. 20' above M.S.L.
3.	Type of gauge:	5" Dia. Non Recording manufactured by H.H. & D New Elton Ltd. London (splayed base)
4.	Owner of gauge:	Ministry of Works, Drainage Division, Hydrology Section
5.	Observer:	Mr. Daniel Clarks, A. A. I
6.	Height of gauge above ground:	2' 4"
7.	Height of nearest obstacle:	24' approximately
8.	Distance from nearest obstacle:	19'
9.	Condition around gauge:	Gauge is set on a concrete slab 1' long x 1' wide and 10" high
10.	Duration of record:	from January 1961
11.	Time of observation:	8:00 a.m. daily

J. Kitamura, Engineer & L. Newallo, Engineering Assistant I

Subject: Location of Rain Guage at Ministry of Agriculture, Botanic Station, Tobago: Map 2 G

1.	Geographical location:	Latitude: 11° 10' 49.07" N Longitude: 60° 44' 07.07" W
2.	Elevation:	Approx. 20' above M.S.L.
3.	Type of gauge:	5" Dia. Non Recording manufactured by Casella London No. 5050 (splayed base)
4.	Observer:	Mr. Alexander Charles (Ag. Stores Clerk)
5.	Owner of gauge:	Ministry of Works, Drainage Division, Hydrology Section
6.	Height of rim above ground:	1' 7½"
7.	Distance from nearest obstacle:	120'
8.	Height of nearest obstacle:	20' approximately
9.	Condition of ground surface around gauge:	Flat with surrounding area gently undulating
10.	Duration of record:	January 1910 to present date
11.	Time of observation:	8:00 a.m. daily

J. Kitamura, Engineer & L. Newallo, Engineering Assistant I

Subject: Location of Rain Gauge at Courland Estate, Plymouth: Map 1 C

1.	Geographical location:	Latitude: 11° 12' 36.5" N Longitude: 60° 46' 29.7" W
2.	Elevation:	Approx. 50' above M.S.L.
3.	Type of gauge:	5" Dia. Non Recording Rain Gauge manufactured by Negretti Aambra
4.	Owner of gauge:	Private
5.	Observer:	Oswald H. Scott
6.	Height of gauge above ground:	1' 9"
7.	Height of nearest obstacle:	35' approximately
8.	Distance from nearest obstacle:	65'
9.	Condition around gauge:	Gauge is placed in concrete slab, on land gently sloping.
10.	Duration of record:	January 1954 to present date
11.	Time of observation:	8:00 a.m. daily

J. Kitamura, Engineer & L. Newallo, Engineering Assistant I

Subject: Location of Rain Gauge at Diamond Estates, Auckenskeoch: Map 1 F

1.	*Geographical location:*	Latitude: 11° 09' 49.0" N Longitude: 60° 46' 05.1" W
2.	Elevation:	Approx. 25' above M.S.L.
3.	Type of gauge:	5" Dia. Non Recording manufactured by Casella London No. W5050 (splayed base)
4.	Owner of gauge:	Ministry of Works, Drainage Dept., Hydrology Section
5.	Observer:	Mr. N. Anthony Bishop
6.	Height of gauge:	2' 0"
7.	Height of nearest obstacle:	12"
8.	Distance from nearest obstacle:	74'
9.	Condition around gauge:	Gently undulating but with grass about 1"
10.	Duration of record:	January 1951 to date
11.	Time of observation:	8:00 a.m. daily

J. Kitamura, Engineer & L. Newallo, *Engineering Assistant I*

Subject: Location of Rain Gauge at Friendship Estate: Map 1 J

1.	Geographical location:	Latitude: 11° 08' 44.9" N Longitude: 60° 46' 19.5" W
2.	Elevation:	Approx. 20' above M.S.L.
3.	Type of gauge:	5" Dia. Non Recording manufactured by W. Rollason & Co., Ltd. No. 1078/28
4.	Observer: Manager:	Mrs. Lucille Alfred Mr. Thomas Wallace
5.	Owner of gauge:	Ministry of Works, Drainage Division, Hydrology Section
6.	Height of rim above ground:	1' 8"
7.	Distance from nearest obstacle:	28'
8.	Height of nearest obstacle:	10' approximately
9.	Duration of record:	March 1912 to date
10.	Condition of ground surface around gauge:	Flat with grass about 3' high
11.	Time of observation:	8:00 a.m. daily

J. Kitamura, Engineer & L. Newallo, Engineering Assistant I

Subject: Location of Rain Gauge at L'Anse Formi Methodist School, Ministry of Education

1.	Geographical location:	Latitude: 11° 18' 29.06" Longitude: 60° 36' 57.09"
2.	Elevation:	Approx. 642' above M.S.L.
3.	Type of gauge:	5" Dia. Non Recording manufactured by Casella London No. 3184
4.	Observer:	Mr. George James, Pupil Teacher
5.	Owner of gauge:	Ministry of Works, Drainage Division, Hydrology Section
6.	Height of gauge above ground:	1' 0" above ground gauge is buried.
7.	Distance from nearest obstacle:	29'
8.	Height of nearest obstacle:	20' approximately
9.	Condition around ground:	Steep slope with grass about 6" high
10.	Duration of record:	New station but records from L'Anse Formi Estate from February 1951 to June 1952
11.	Time of observation:	8:00 a.m. daily

J. Kitamura, Engineer & L. Newallo, Engineering Assistant I

Subject: Location of Rain Gauge at Kendal Estate, Ministry of Agriculture: Map 3 G

1.	Geographical location:	Latitude: 11° 14' 29.80" Longitude: 60° 35' 48.70"
2.	Elevation:	Approx. 354' above M.S.L.
3.	Type of gauge:	5" Dia. Non Recording manufactured by Stanley London
4.	Owner of gauge:	Ministry of Works, Drainage Division, Hydrology Section
5.	Observer:	Mr. Clovis Duke
6.	Height of rim above ground:	8"
7.	Distance from nearest obstacle:	157'
8.	Height of nearest obstacle:	Approx. 25'
9.	Duration of record:	January 1924 to date
10.	Condition of ground:	Steeply sloping
11.	Time of observation:	8:00 a.m. daily

J. Kitamura, Engineer & L. Newallo, Engineering Assistant I

Subject: Location of Rain Gauge at Speyside: Map 3 E

1.	Geographical location:	Latitude: 11° 17' 47.10" Longitude: 60° 32' 08.25"
2.	Elevation:	Approx. 20' above M.S.L.
3.	Type of gauge:	5" Dia. Non Recording manufactured by Stanley London (splayed base)
4.	Owner of gauge:	Ministry of Works, Drainage Division, Hydrology Section
5.	Observer:	Mr. Jeremish George
6.	Height of rim above ground:	1' 7"
7.	Distance from nearest obstacle:	13' 0"
8.	Height of nearest obstacle:	12' 0"
9.	Duration of record:	May 1961 to date
10.	Condition of ground surface around gauge:	Flat with grass about 1" high
11.	Time of observation:	8:00 a.m. daily

J. Kitamura, Engineer & L. Newallo, Engineering Assistant I

Subject: Location of Rain Gauge at Lowland Estate, Tobago Plantation Limited: Map 1 J

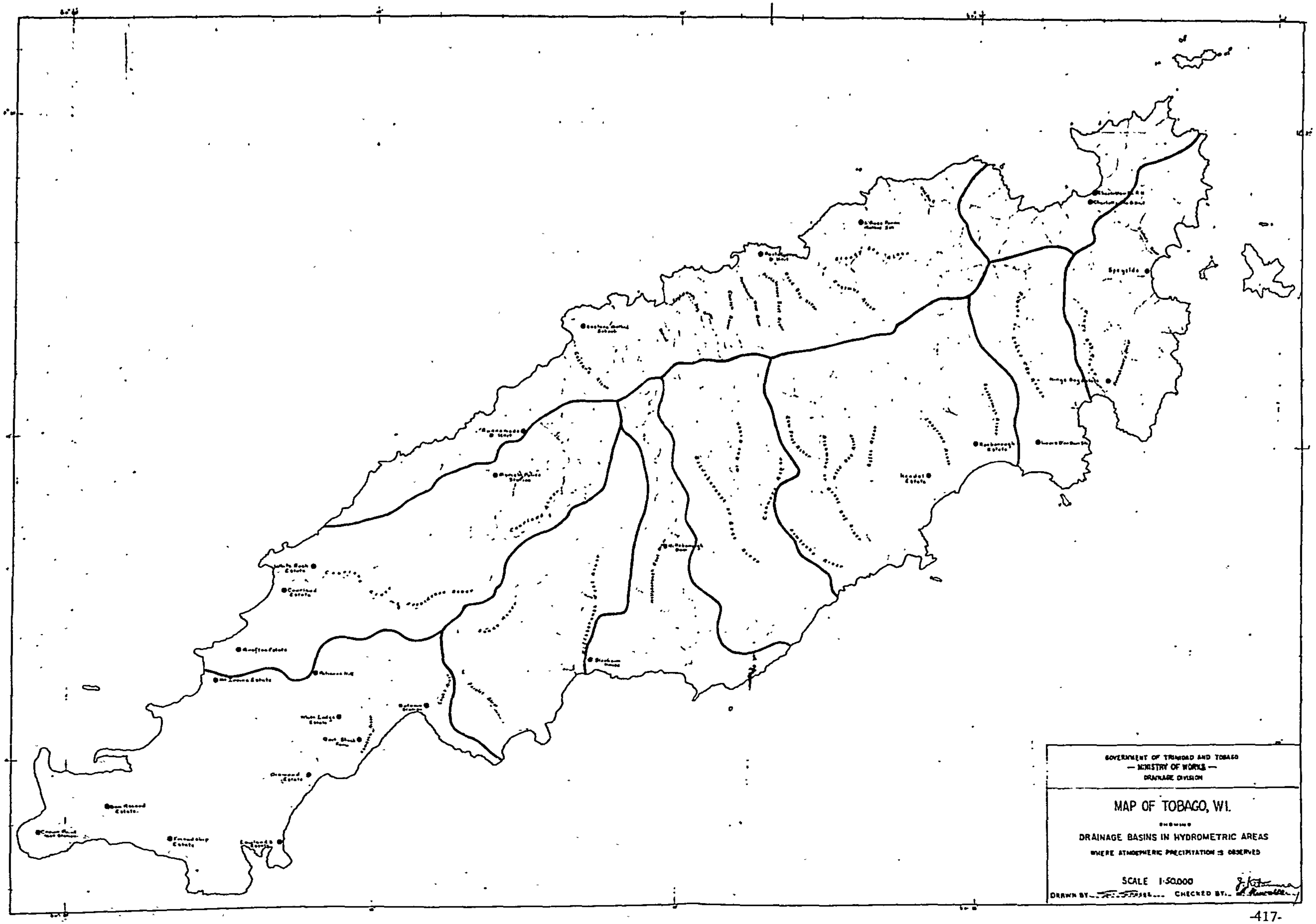
1.	Geographical location:	Latitude: 11° 08' 40.2" N Longitude: 60° 46' 32.7" W
2.	Elevation:	Approx. 20' above M.S.L.
3.	Type of guage:	5" Dia. Non Recording manufactured by Casella London No. W5050
4.	Manager: Observer:	Mr. Edward K. Maynard Miss Jean Frank
5.	Owner of gauge:	Ministry of Works, Drainage Division, Hydrology Section
6.	Height of rim of gauge:	2' 6"
7.	Distance from nearest obstacle:	45'
8.	Height of nearest obstacle:	25'
9.	Duration of record:	January 1956 to present date
10.	Condition of ground around gauge:	Flat with gauge on concrete slab surrounded by fence
11.	Time of observation:	8:00 a.m. daily

J. Kitamura, Engineer & L. Newallo, Engineering Assistant I

Subject: *Location of Rain Gauge at Hillsborough Dam,* Water and Sewerage Authority: Map 2 E

1.	Geographical location:	Latitude: 11° 13' 20.3" N Longitude: 60° 40' 13.1" W
2.	Elevation:	Approx. 788' above M.S.L.
3.	Type of gauge:	5" Dia. Non Recording manufactured by Casella London No. W5050 (splayed base)
4.	Observer:	Mr. Davidson Edmund (Water Works Superintendent)
5.	Owner of gauge:	Ministry of Works, Drainage Division, Hydrology Section
6.	Height of gauge above ground:	2' 8¼"
7.	Distance from nearest obstacle:	32'
8.	Height of nearest obstacle:	12' approximately
9.	Condition around gauge:	Gauge is placed on wooden stand 2' sq. and 1' 1" high on wall of dam.
10.	Duration of record:	April 1948 to date
11.	Time of observation:	8:00 a.m. daily

J. Kitamura, Engineer & L. Newallo, Engineering Assistant I


GOVERNMENT OF TRINIDAD AND TOBAGO
— MINISTRY OF WORKS —
DRAINAGE DIVISION
MAP OF TOBAGO, W.I.
SHOWING
DRAINAGE BASINS IN HYDROMETRIC AREAS
WHERE ATMOSPHERIC PRECIPITATION IS OBSERVED
SCALE 1:50,000
DRAWN BY
CHECKED BY

DESIGN FOR SETTING AUTOMATIC RIVER STAGE RECORDER (ISHIDA TYPE)

The designs for setting the automatic river stage recorders from the Japanese Government, on the San Juan River and on *Caroni Irrigation* Canal were carried out as shown on attached figures. According to them, lists of the quantity of materials are made as follows: (Exclusive of earth volume for cutting and back filling.)

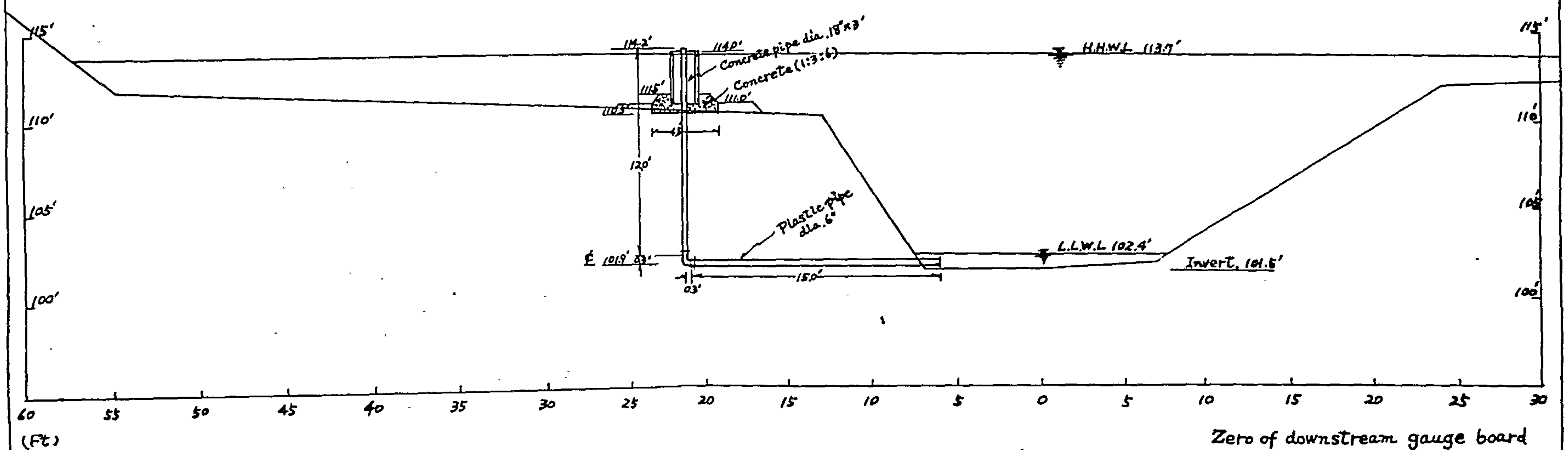
	FOR THE SAN JUAN RIVER						FOR CARONI IRRIGATION CANAL					
Item	Number	Dia.	Width	Thickness	Length	Remarks	Number	Dia.	Width	Thickness	Length	Remarks
Plastic pipe	2	6"			20'	with socket	1	6"			20'	with socket
Plastic elbow	1	6"					1	6"				
Concrete pipe	1	18"			3'		1	18"			3'	
Wooden pile	4	4"			6'		4	4"			55'	
Board	4		4"	1"	5' - 2"		4		4"	1"	5' - 2"	
Board	13		4"	1"	2'		13		4"	1"	2'	
Board	3		4"	1.5"	1'		3		4"	1.5"	1'	
Bolt, nut and washer	4	¼"			3"		4	¼"			3"	
Nail	Some				2" - 3"		Some				2" - 3"	
Lock for box	1					fit for the hole of the box	1					fit for the hole of the box
Concrete (1 : 3 : 6)	4.0' x 4.75' x 1.0' = 19.0 cu.ft. (Cement (Sand (Gravel				= ¾ cu.yd. 3.5 bags 0.45 cu.yd. 0.70 cu.yd.		4.0' x 4.5' x 1.0' = 18 cu.ft.				= 2/3 cu.yd. 3.0 bags 0.4 cu.yd. 0.6 cu.yd.	

30th Nov., 1968, by Junichi Kitamura, Irrigation Engineer

(Fig.1)

Design for Setting the Automatic River Stage Recorder

Plastic Pipe 2 × dia. 6" × 20'
" Elbow 1 × dia. 6"
Concrete Pipe 1 × dia. 18" × 3'

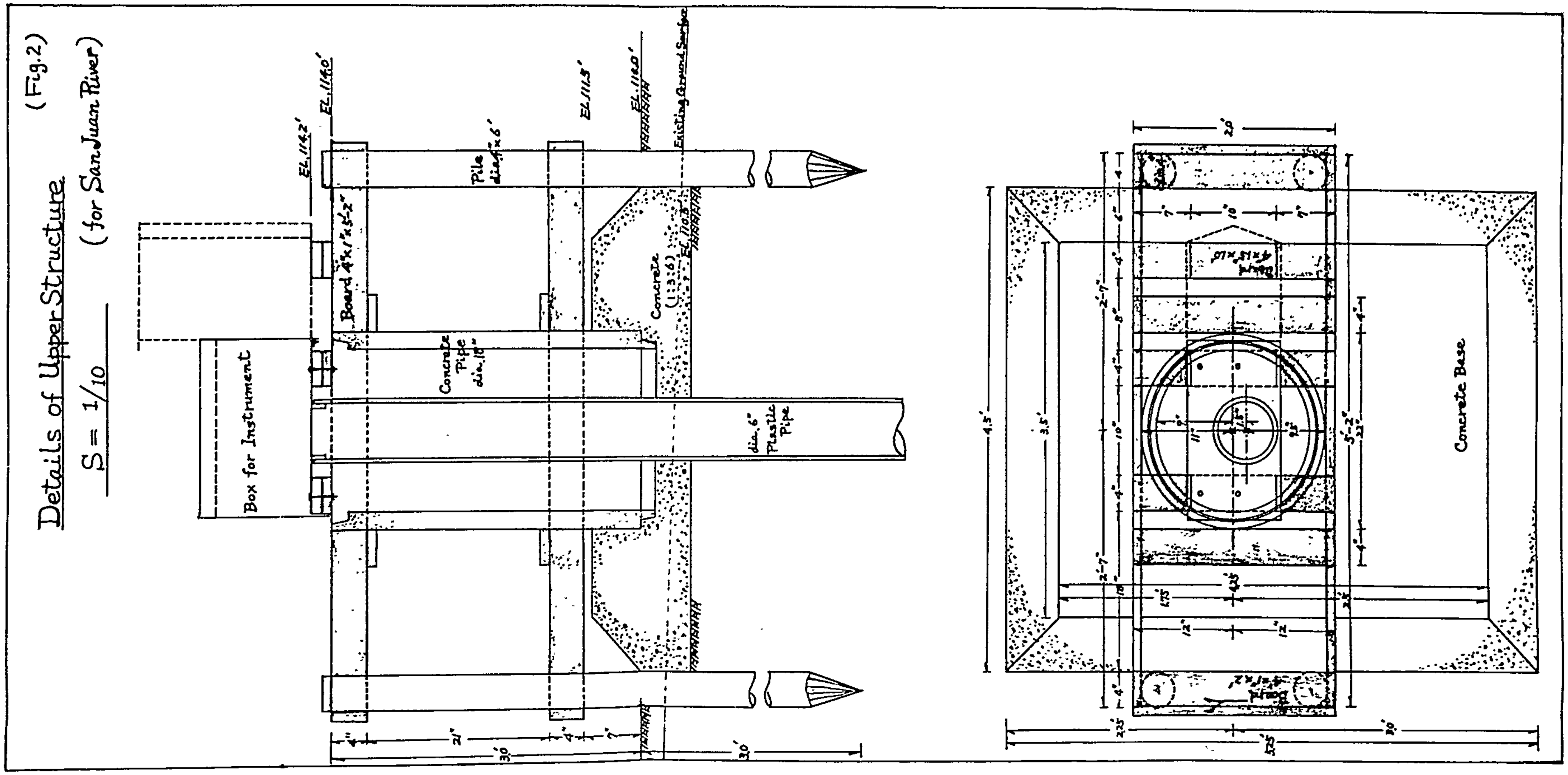


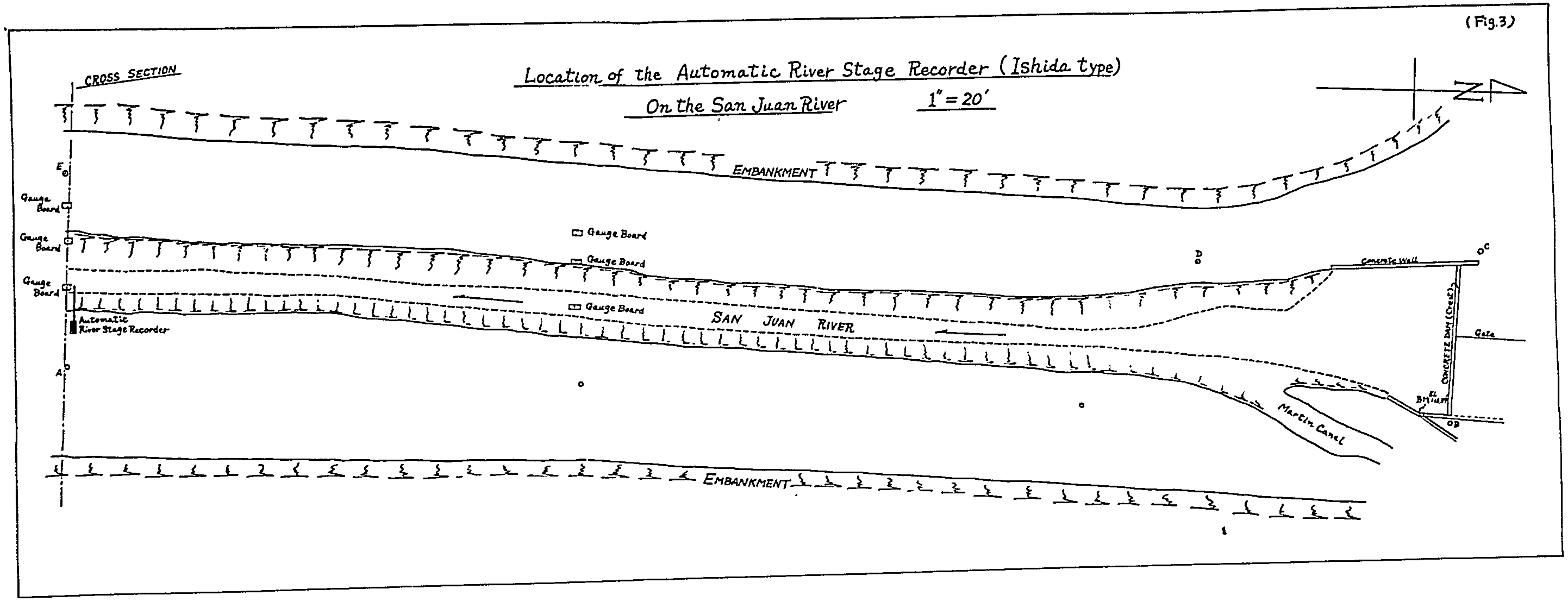
Zero of downstream gauge board is 101.90' T.G.R.

SECT. 5 AT STA. 460' D/S OF DAM

SAN JUAN RIVER

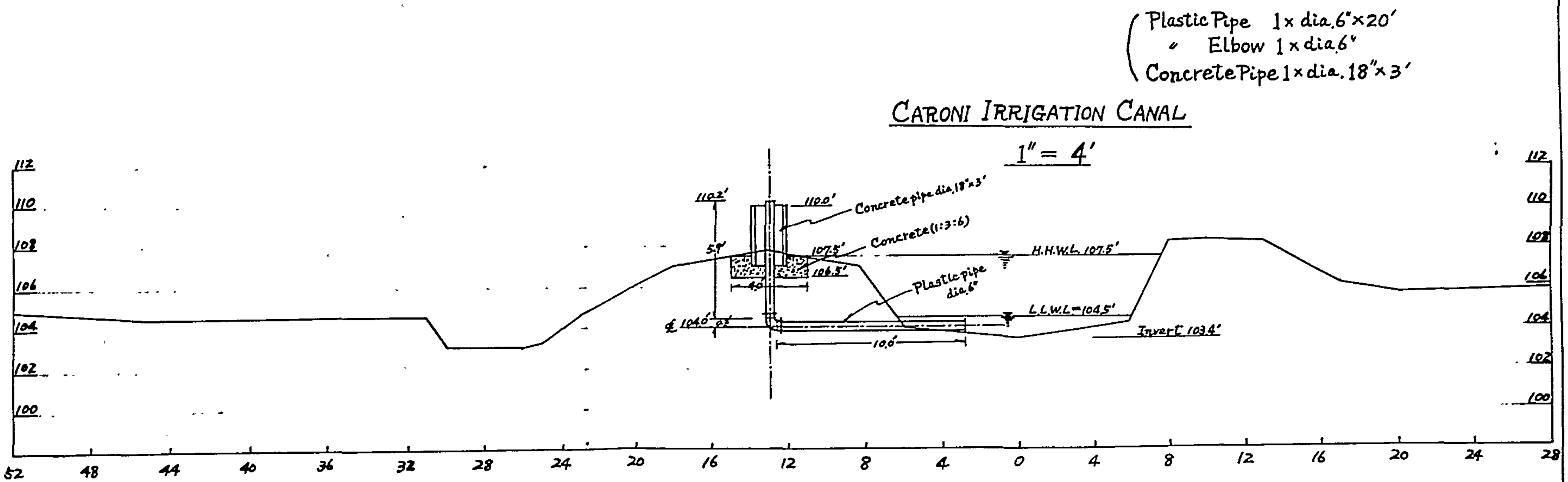
1" = 5'


Details of Upper Structure
(Fig.2)
S = 1/10
(for San Juan River)
Box for Instrument
Board 4"x1"x5'-2"
Pile dia.4"x6'
Concrete Pipe dia.18"
dia.6" Plastic Pipe
Concrete (1:3:6)
EL.114.2'
EL.114.0'
EL.111.5'
EL.111.0'
EL.110.5'
Existing Ground Surface
Concrete Base


(Fig.3)
Location of the Automatic River Stage Recorder (Ishida type)
On the San Juan River
1" = 20'
CROSS SECTION
E
Gauge Board
Gauge Board
Gauge Board
Automatic River Stage Recorder
A
EMBANKMENT
Gauge Board
Gauge Board
Gauge Board
SAN JUAN RIVER
D
C
Concrete Wall
Gate
CONCRETE DAM (Creek)
Martin Canal
B
EMBANKMENT

(Fig.1)

Design for Setting the Automatic River Stage Recorder

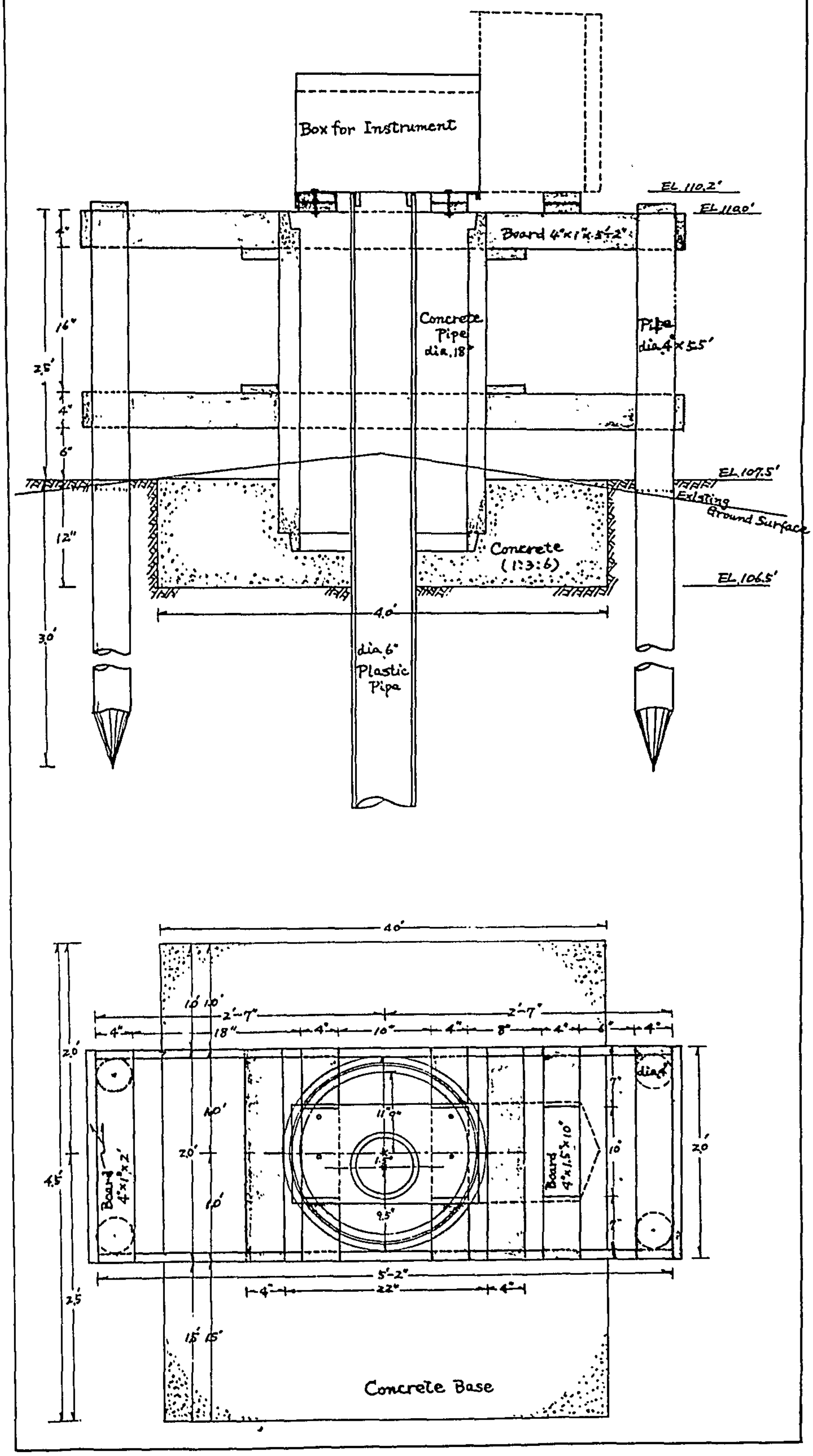


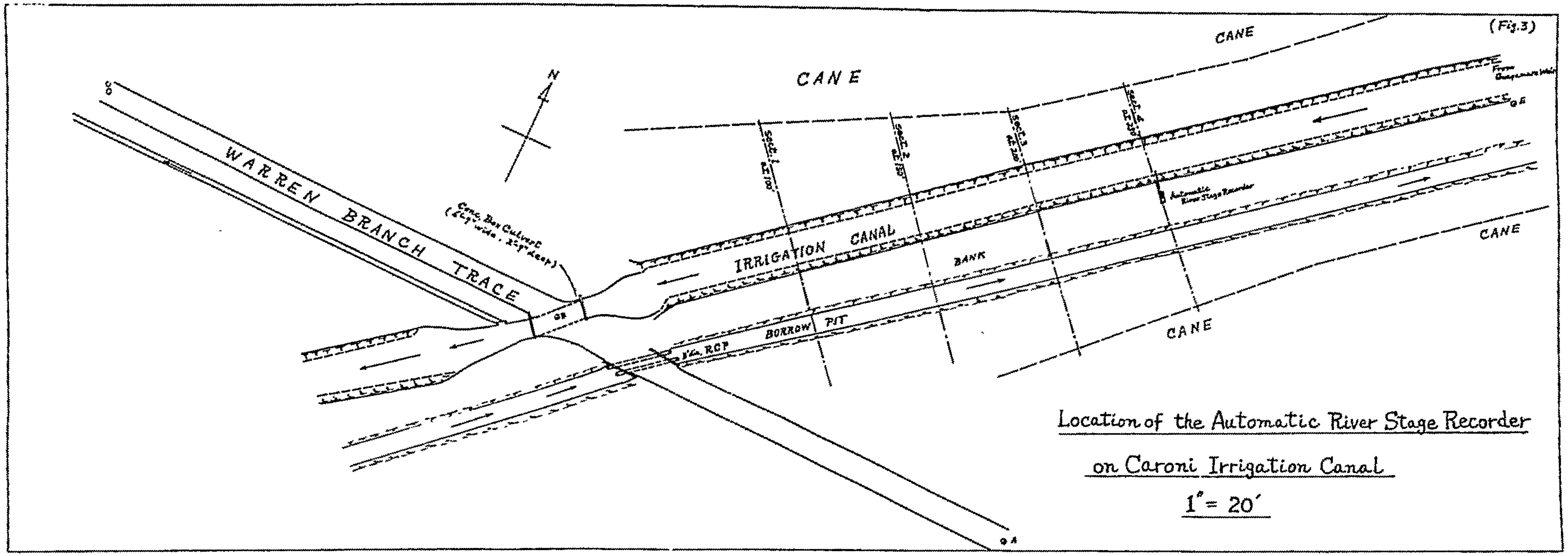
SECTION 4. 250 FT U/S OF BRIDGE
ON WARREN BRANCH TRACE

(Fig.2)

Details of Upper Structure

S= 1/10 (for Caroni Irrigation Canal)





Location of the Automatic River Stage Recorder on Caroni Irrigation Canal

1" = 20'