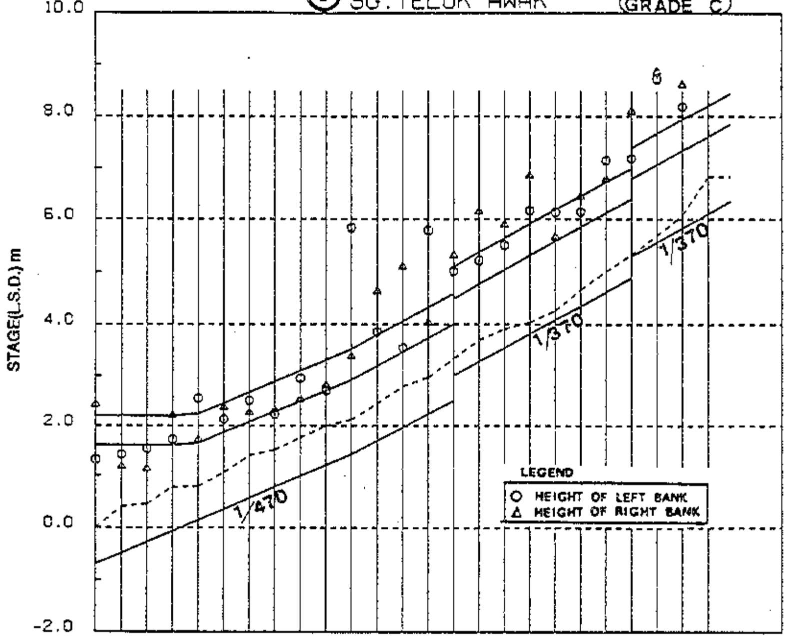
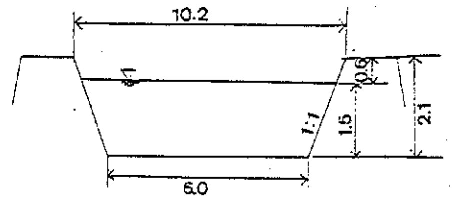
② SG. TELUK AWAK (GRADE C)



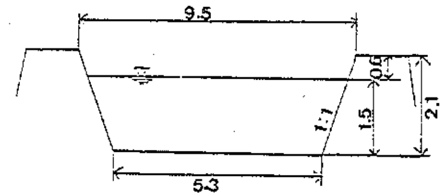
PROPOSED BED LEVEL	-0.700	-0.487	-0.274	-0.062	0.151	0.364	0.577	0.789	1.002	1.215	1.428	1.698	1.968	2.238	2.508 3.008	3.279	3.549	3.820	4.090	4.360	4.630	4.801 5.301	5.571	5.841	6.111	6.350
DESIGN H.W.L.	1.600	1.600	1.600	1.600	1.651	1.864	2.077	2.289	2.502	2.715	2.928	3.198	3.468	3.738	4.000 4.500	4.779	5.049	5.320	5.590	5.860	6.130	6.401 6.801	7.071	7.341	7.611	7.850
PROPOSED BANK LEVEL	2.200	2.200	2.200	2.200	2.251	2.464	2.677	2.889	3.102	3.315	3.528	3.798	4.068	4.338	4.608 5.109	5.379	5.649	5.920	6.190	6.460	6.730	7.001 7.401	7.671	7.941	8.211	8.450
DISTANCE	0.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	88.000
CHAINAGE	0.000	100.000	200.000	300.000	400.000	500.000	600.000	700.000	800.000	900.000	1000.000	1100.000	1200.000	1300.000	1400.000	1500.000	1600.000	1700.000	1800.000	1900.000	2000.000	2100.000	2200.000	2300.000	2400.000	2488.000

Q = 21 m³/s

SG. TELUK AWAK



N	L/I	H	A	B	R	RR	V	Q
.025	470.000	1.500	11.250	9.300	1.098	1.065	1.964	22.097

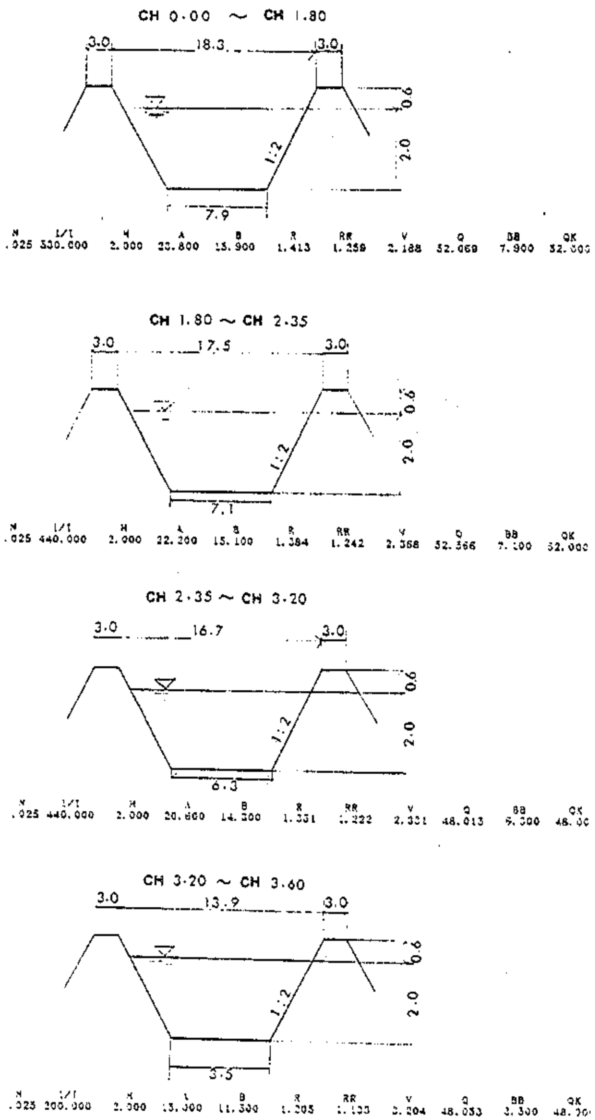


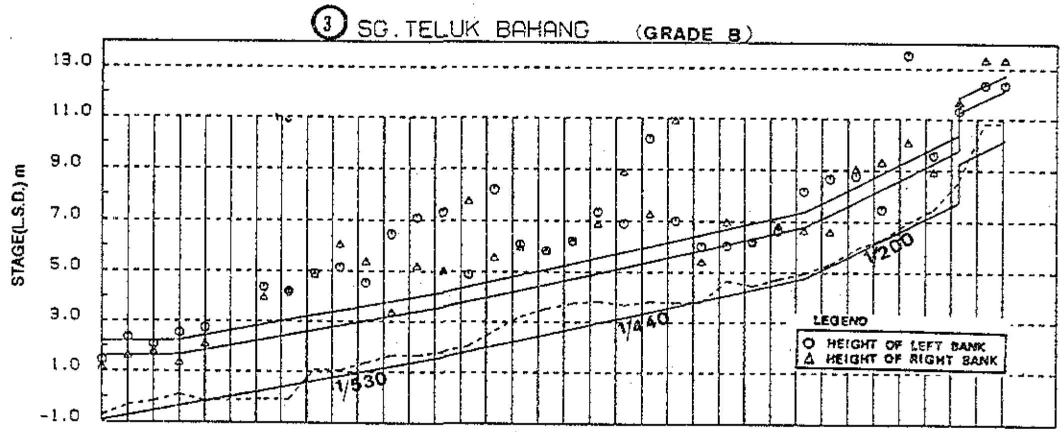
N	L/I	H	A	B	R	RR	V	Q
.025	370.000	1.500	10.200	8.300	1.069	1.045	2.174	22.174

FIG. H-20-1 **PROPOSED LONGITUDINAL PROFILE AND CROSS SECTION OF SG. TELUK AWAK**

THE STUDY ON FLOOD MITIGATION AND DRAINAGE IN PENANG ISLAND

JICA



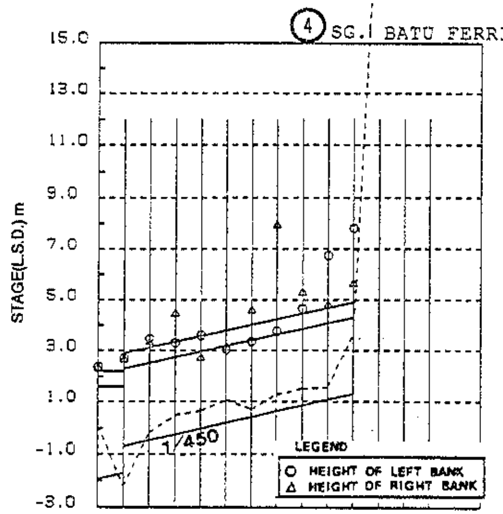


CHAINAGE	DISTANCE	PROPOSED BANK LEVEL	DESIGN H.W.L.	PROPOSED BED LEVEL
0.000	0.000	2.200	1.600	-0.520
100.000	100.000	2.200	1.600	-0.731
200.000	100.000	2.200	1.600	-0.543
300.000	100.000	2.246	1.646	-0.354
400.000	100.000	2.435	1.835	-0.165
1100.000	230.000	2.869	2.269	0.269
1200.000	100.000	3.057	2.457	0.457
1300.000	100.000	3.246	2.646	0.646
1400.000	100.000	3.435	2.835	0.835
1500.000	100.000	3.623	3.023	1.023
1600.000	100.000	3.812	3.212	1.212
1700.000	100.000	4.001	3.401	1.401
1800.000	100.000	4.189	3.589	1.589
1900.000	100.000	4.417	3.817	1.817
2000.000	100.000	4.644	4.044	2.044
2100.000	100.000	4.871	4.271	2.271
2200.000	100.000	5.099	4.499	2.499
2300.000	100.000	5.326	4.726	2.726
2400.000	100.000	5.553	4.953	2.953
2500.000	100.000	5.780	5.180	3.180
2600.000	100.000	6.008	5.408	3.408
2700.000	100.000	6.235	5.635	3.635
2800.000	100.000	6.462	5.862	3.862
2900.000	100.000	6.689	6.089	4.089
3000.000	100.000	6.917	6.317	4.317
3100.000	100.000	7.144	6.544	4.544
3200.000	100.000	7.371	6.771	4.771
3300.000	100.000	7.871	7.271	5.271
3400.000	100.000	8.371	7.771	5.771
3500.000	100.000	8.871	8.271	6.271
3600.000	100.000	9.371	8.771	6.771
3700.000	100.000	9.871	9.271	7.271
3800.000	100.000	10.371 / 11.871	9.771 / 11.271	7.771 / 9.271
3900.000	100.000	12.371	11.771	9.771
3977.000	77.000	12.756	12.156	10.156

Q = 52 m³/s Q = 48 m³/s Q = 32 m³/s

FIG. H-20-2 **PROPOSED LONGITUDINAL PROFILE AND CROSS SECTION OF SG. TELUK BAHANG**

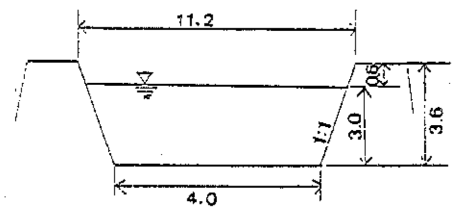
THE STUDY ON FLOOD MITIGATION AND DRAINAGE IN PENANG ISLAND



PROPOSED BED LEVEL	-1.920	-1.698 -1.698	-0.476	-0.253	-0.031	0.191	0.413	0.636	0.858	1.080	1.302	0.000	0.000	0.000
DESIGN H.W.L.	1.600	1.600 2.302	2.524	2.747	2.969	3.191	3.413	3.636	3.858	4.080	4.302	0.000	0.000	0.000
PROPOSED BANK LEVEL	2.200	2.200 2.802	3.124	3.347	3.569	3.791	4.013	4.236	4.458	4.680	4.902	0.000	0.000	0.000
DISTANCE	0.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000
CHAINAGE	0.000	100.000	200.000	300.000	400.000	500.000	600.000	700.000	800.000	900.000	1000.000	1100.000	1200.000	1300.000

Q = 53 m³/s

SG. BATU FERRINGGHI



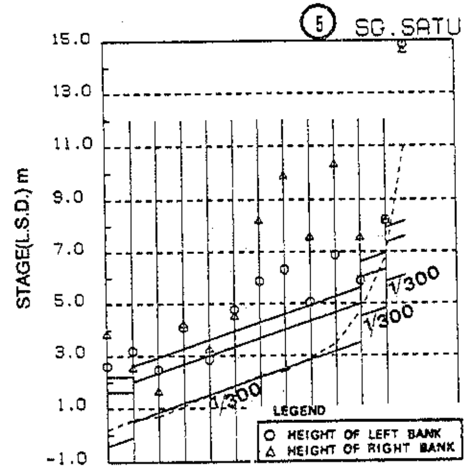
N	I/T	H	A	B	R	RR	V	Q	SB	QX
.025	450.000	3.000	21.000	10.000	1.682	1.414	2.667	56.004	4.000	55.000

FIG. H-20-3 **PROPOSED LONGITUDINAL PROFILE AND CROSS SECTION OF SG. BATU FERRINGGHI**

THE STUDY ON FLOOD MITIGATION AND DRAINAGE IN PENANG ISLAND

JICA

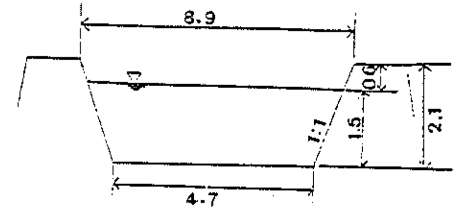
(5) SG. SATU (GRADE C)



PROPOSED BED LEVEL	-0.420	-.987	.513	0.847	1.180	1.513	1.847	2.180	2.513	2.847	3.180	3.513	4.513	4.847	5.847	6.101
DESIGN H.W.L.	1.600	1.600	2.013	2.347	2.680	3.013	3.347	3.680	4.013	4.347	4.680	5.013	6.013	6.347	7.347	7.601
PROPOSED BANK LEVEL	2.200	2.200	2.613	2.947	3.280	3.613	3.947	4.280	4.613	4.947	5.280	5.613	6.613	6.847	7.847	8.201
DISTANCE	0.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	76.200			
CHAINAGE	0.000	100.000	200.000	300.000	400.000	500.000	600.000	700.000	800.000	900.000	1000.000	1100.000	1176.000			

Q = 20 m³/s

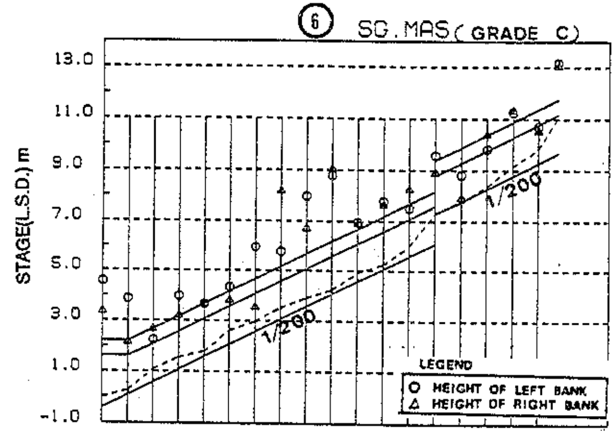
SG. SATU



N	L/I	H	A	B	R	RR	V	Q	SB	OK
.025	300.000	1.500	9.300	7.700	1.040	1.028	2.371	22.046	4.700	22.000

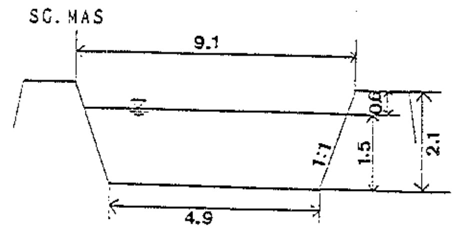
FIG. H-20-4	**PROPOSED LONGITUDINAL PROFILE AND CROSS SECTION OF SG. SATU**

THE STUDY ON FLOOD MITIGATION AND DRAINAGE IN PENANG ISLAND



PROPOSED BED LEVEL	-0.420	0.080	0.580	1.080	1.580	2.080	2.580	3.080	3.580	4.080	4.580	5.080	5.580	6.080 7.280	7.780	8.280	8.780	9.280	9.668
DESIGN H.W.L.	1.600	1.600	2.080	2.580	3.080	3.580	4.080	4.580	5.080	5.580	6.080	6.580	7.080	7.680 8.780	9.280	9.780	10.280	10.780	11.168
PROPOSED BANK LEVEL	2.200	2.200	2.680	3.180	3.680	4.180	4.680	5.180	5.680	6.180	6.680	7.180	7.680	8.180 9.380	9.880	10.380	10.880	11.380	11.769
DISTANCE	0.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	77.600
CHAINAGE	0.000	100.000	200.000	300.000	400.000	500.000	600.000	700.000	800.000	900.000	1000.000	1100.000	1200.000	1300.000	1400.000	1500.000	1600.000	1700.000	1778.000

Q = 27 m³/s

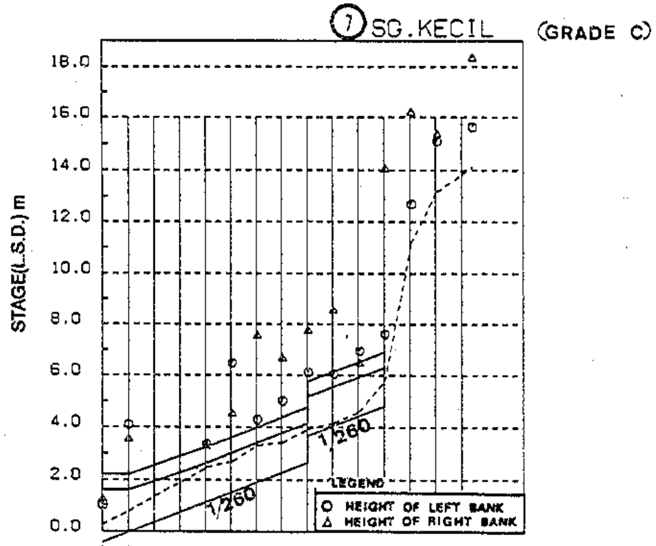


N	I/I	H	A	B	R	RR	V	Q
.025	200.000	1.500	9.600	7.900	1.050	1.033	2.922	23.051

FIG. H-20-5	**PROPOSED LONGITUDINAL PROFILE AND CROSS SECTION OF SG. MAS**

THE STUDY ON FLOOD MITIGATION AND DRAINAGE IN PENANG ISLAND

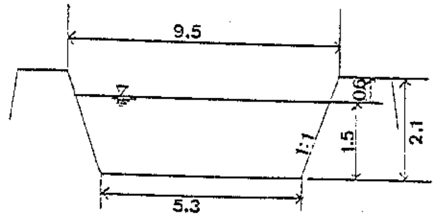
JICA



PROPOSED BED LEVEL	-0.420	-0.035	1.138	1.522	1.907	2.292	2.676 3.676	4.061	4.445	4.830	0.000	0.000	0.000
DESIGN H.W.L.	1.600	1.600	2.638	3.022	3.407	3.792	4.176 5.176	5.561	5.945	6.330	0.000	0.000	0.000
PROPOSED BANK LEVEL	2.200	2.200	3.238	3.622	4.007	4.392	4.776 5.776	6.161	6.545	6.930	0.000	0.000	0.000
DISTANCE	0.000	100.000	305.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	137.000
CHAINAGE	0.000	100.000	600.000	700.000	800.000	900.000	1000.000	1100.000	1200.000	1300.000	1400.000	1500.000	1637.000

Q = 26 m³/s

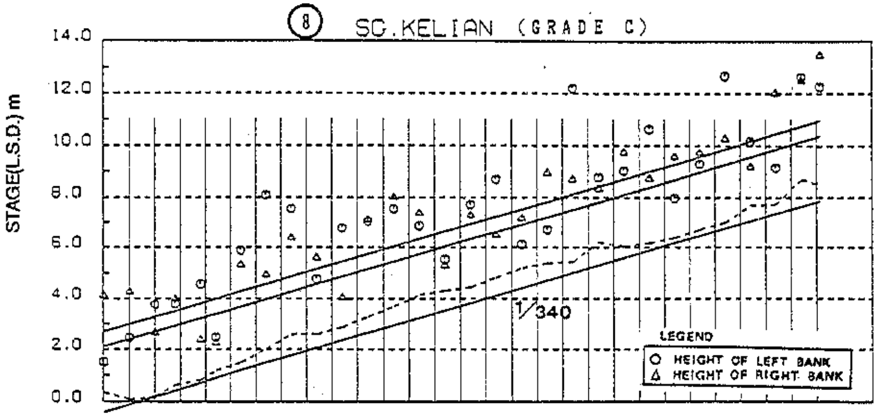
SG. KECIL



N	1/I	H	A	S	R	RR	V	Q
.025	260.000	1.500	10.200	8.300	1.069	1.045	2.593	26.452

FIG. H-20-6 **PROPOSED LONGITUDINAL PROFILE AND CROSS SECTION OF SG. KECIL**

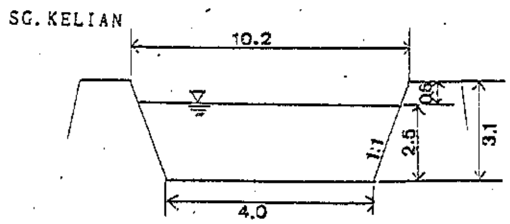
THE STUDY ON FLOOD MITIGATION AND DRAINAGE IN PENANG ISLAND



PROPOSED BED LEVEL	-0.400	-0.106	0.188	0.424	0.718	0.894	1.188	1.482	1.776	2.071	2.365	2.659	2.953	3.247	3.541
DESIGN H.W.L.	2.100	2.394	2.688	2.924	3.218	3.394	3.688	3.982	4.276	4.571	4.865	5.159	5.453	5.747	6.041
PROPOSED BANK LEVEL	2.700	2.994	3.288	3.524	3.818	3.994	4.288	4.582	4.876	5.171	5.465	5.759	6.053	6.347	6.641
DISTANCE	0.000	100.000	100.000	80.000	100.000	60.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000
CHAINAGE	0.000	0.100	0.200	0.300	0.400	0.500	0.600	0.700	0.800	0.900	1.000	1.100	1.200	1.300	1.400

PROPOSED BED LEVEL	3.835	4.129	4.424	4.718	5.012	5.306	5.600	5.894	6.188	6.482	6.776	7.071	7.365	7.659	7.869
DESIGN H.W.L.	6.335	6.629	6.924	7.218	7.512	7.806	8.100	8.394	8.688	8.982	9.276	9.571	9.865	10.159	10.369
PROPOSED BANK LEVEL	6.935	7.229	7.524	7.818	8.112	8.406	8.700	8.994	9.288	9.582	9.876	10.171	10.465	10.759	10.969
DISTANCE	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	71.500
CHAINAGE	1.500	1.600	1.700	1.800	1.900	2.000	2.100	2.200	2.250	2.400	2.500	2.600	2.700	2.800	2.871

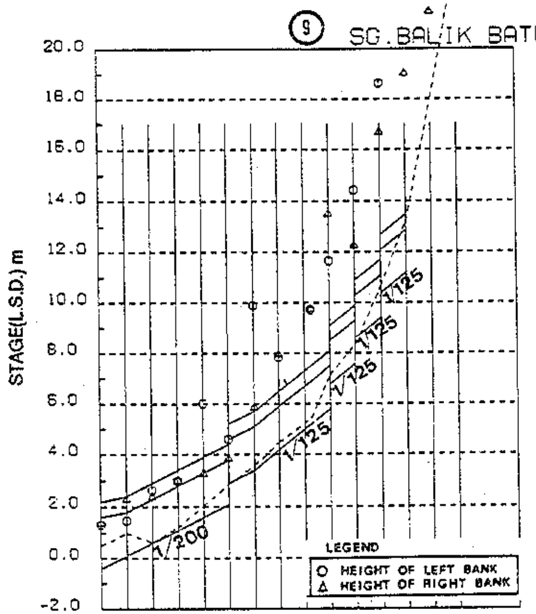
Q = 44



N	1/I	H	A	B	R	RR	V	Q
.025	340.000	2.500	15.250	9.000	1.468	1.292	2.802	45.529

FIG. H-20-7 **PROPOSED LONGITUDINAL PROFILE AND CROSS SECTION OF SG. KELIAN**

THE STUDY ON FLOOD MITIGATION AND DRAINAGE IN PENANG ISLAND



PROPOSED BED LEVEL	-0.400, 0.100, 0.600, 1.100, 1.600, 2.100, 2.800, 3.400, 4.200, 5.203, 5.800, 6.800, 7.600, 8.600, 9.400, 10.400, 11.200, 0.000, 8.888, 8.888
DESIGN H.W.L.	1.600, 1.800, 2.300, 2.800, 3.300, 3.800, 4.600, 5.100, 5.900, 6.909, 7.500, 8.500, 9.300, 10.300, 11.100, 12.100, 12.900, 0.000, 8.888, 8.888
PROPOSED BANK LEVEL	2.200, 2.400, 2.900, 3.400, 3.900, 4.400, 5.200, 5.700, 6.500, 7.508, 8.100, 8.100, 8.900, 10.000, 11.700, 12.700, 13.500, 0.000, 8.888, 8.888
DISTANCE	0.000, 100.000, 100.000, 100.000, 100.000, 100.000, 100.000, 100.000, 126.000, 78.000, 100.000, 100.000, 100.000, 100.000, 199.888, 33.088
CHAINAGE	0.000, 100.000, 200.000, 300.000, 400.000, 500.000, 600.000, 700.000, 826.000, 900.000, 1000.000, 1100.000, 1200.000, 1300.000, 1499.888, 1433.088

Q = 18 m³/s

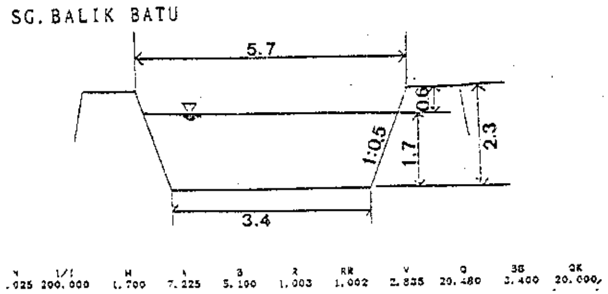
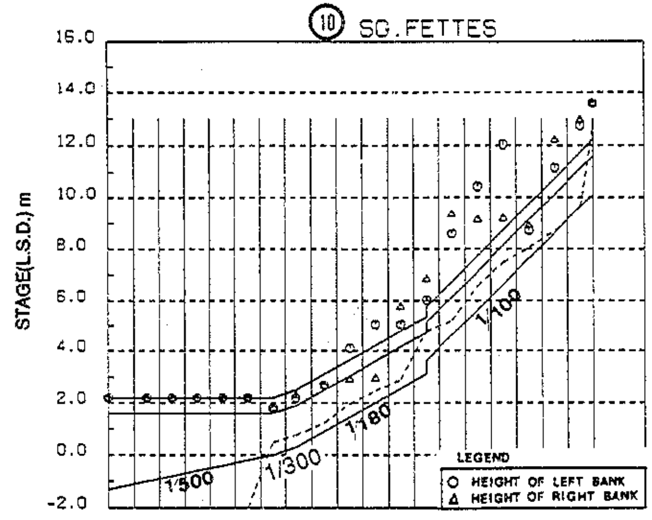


FIG. H-20-8	**PROPOSED LONGITUDINAL PROFILE AND CROSS SECTION OF SG. BALIK BATU**

THE STUDY ON FLOOD MITIGATION AND DRAINAGE IN PENANG ISLAND



PROPOSED BED LEVEL	-1.300	-1.000	-0.800	-0.600	-0.400	-0.200	0.000	0.300	0.911	1.467	2.022	2.578	3.133 3.633	4.633	5.633	6.633	7.633	8.633	9.633	10.133
DESIGN H.W.L.	1.600	1.600	1.600	1.600	1.600	1.600	1.600	1.900	2.511	3.067	3.622	4.178	4.733 5.133	6.133	7.133	8.133	9.133	10.133	11.133	11.633
PROPOSED BANK LEVEL	2.200	2.200	2.200	2.200	2.200	2.200	2.200	2.500	3.111	3.667	4.222	4.778	5.333 5.733	6.733	7.733	8.733	9.733	10.733	11.733	12.233
DISTANCE	-650.000	150.000	100.000	100.000	100.000	100.000	100.000	90.000	110.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	50.000
CHAINAGE	-650.000	-500.000	-400.000	-300.000	-200.000	-100.000	0.000	90.000	200.000	300.000	400.000	500.000	600.000	700.000	800.000	900.000	1000.000	1100.000	1200.000	1250.000

Q = 28 m³/s Q = 16 m³/s

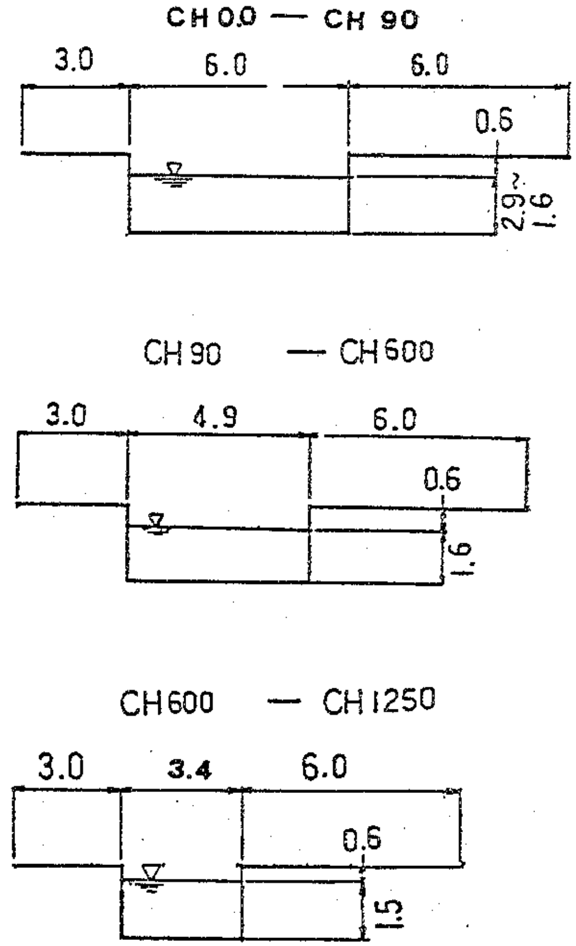
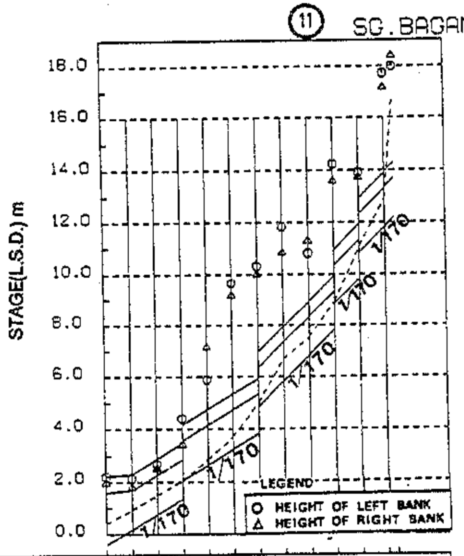


FIG. H-20-9 **PROPOSED LONGITUDINAL PROFILE AND CROSS SECTION OF SG. FETTES**

THE STUDY ON FLOOD MITIGATION AND DRAINAGE IN PENANG ISLAND

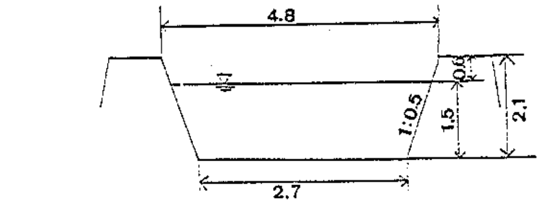
⑪ SG. BAGAN JERMAL (GRADE C)



PROPOSED BED LEVEL	-0.400	0.188	0.776	1.365 / 2.065	2.653	3.241	3.828 / 4.829	5.829	6.829	7.828 / 8.828	9.828 / 10.828	11.829 / 12.169
DESIGN H.W.L.	1.600	1.688	2.276	2.865 / 3.565	4.153	4.741	5.329 / 6.329	7.329	8.329	10.329 / 9.328	10.328 / 11.328	12.329
PROPOSED BANK LEVEL	2.200	2.288	2.876	3.465 / 4.165	4.753	5.341	5.928 / 6.928	7.929	8.929	9.828 / 10.828	11.828 / 12.928	13.929 / 14.269
DISTANCE	0.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000 / 36.000
CHAINAGE	0.000	100.000	200.000	300.000	400.000	500.000	600.000	700.000	800.000	900.000	1000.000	1100.000 / 1136.000

Q = 13 m³/s

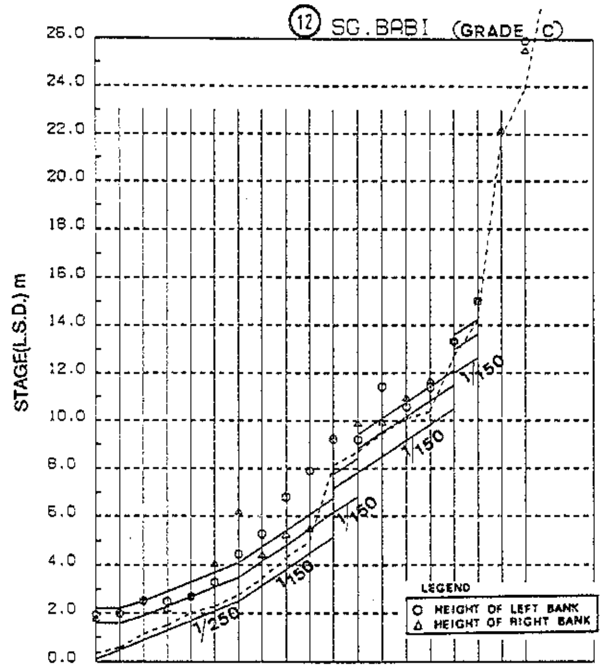
SG. BAGAN JERMAL



N	I/I	H	A	B	R	RR	V	Q
.025	170.000	1.500	5.175	4.200	.855	.901	2.763	14.299

FIG. H-20-10	PROPOSED LONGITUDINAL PROFILE AND CROSS SECTION OF SG. BAGAN JERMAL

THE STUDY ON FLOOD MITIGATION AND DRAINAGE IN PENANG ISLAND

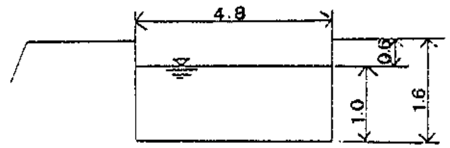


PROPOSED BED LEVEL	0.100	0.500	0.900	1.300	1.700	2.100	2.500	3.167	3.833	4.500	5.167 6.167	6.833 7.833	8.500	9.167	9.833	10.500 12.000	12.667	0.000	0.000	0.000
DESIGN H.W.L.	1.600	1.600	1.900	2.300	2.700	3.100	3.500	4.167	4.833	5.500	6.167 7.167	7.833 8.833	9.500	10.167	10.833	11.500 13.000	13.667	0.000	0.000	0.000
PROPOSED BANK LEVEL	2.200	2.200	2.500	2.900	3.300	3.700	4.100	4.767	5.433	6.100	6.767 7.767	8.433 9.433	10.100	10.767	11.433	12.100 13.600	14.267	0.000	0.000	0.000
DISTANCE	0.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	83.000
CHAINAGE	0.000	100.000	200.000	300.000	400.000	500.000	600.000	700.000	800.000	900.000	1000.000	1100.000	1200.000	1300.000	1400.000	1500.000	1600.000	1700.000	1800.000	1882.000

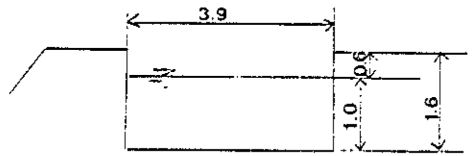
Q=62 m³/s

Q = 12 m³/s

SG. BABI



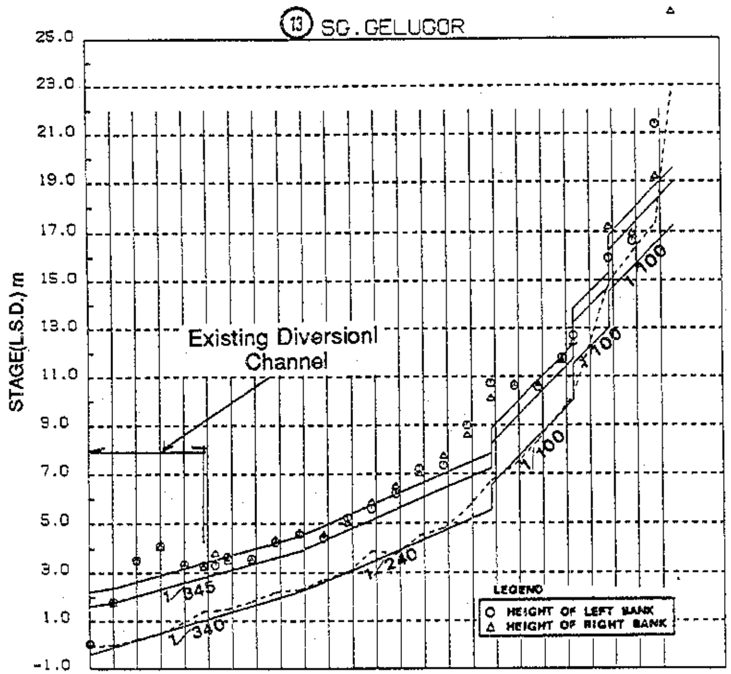
N	1/I	H	A	B	R	RR	V	Q
.020	250.000	1.000	4.800	4.800	.706	.793	2.507	12.034



N	1/I	H	A	B	R	RR	V	Q
.020	150.000	1.000	3.900	3.900	.661	.759	3.098	12.082

FIG. H-20-11	**PROPOSED LONGITUDINAL PROFILE AND CROSS SECTION OF SG. BABI**

THE STUDY ON FLOOD MITIGATION AND DRAINAGE IN PENANG ISLAND



CHAINAGE	DISTANCE	PROPOSED BANK LEVEL	DESIGN H.W.L.	PROPOSED BED LEVEL
0.000	0.000	2.200	1.600	-0.397
100.000	100.000	2.353	1.753	-0.103
200.000	100.000	2.627	2.027	0.191
300.000	100.000	2.901	2.301	0.485
400.000	100.000	3.175	2.575	0.779
480.010	80.010	3.395	2.795	1.015
1050.000	50.000	3.532	2.932	1.162
1100.000	50.000	3.669	3.069	1.309
1200.000	100.000	3.943	3.343	1.603
1300.000	100.000	4.216	3.616	1.897
1400.000	100.000	4.491	3.891	2.191
1500.000	100.000	4.908	4.309	2.608
1600.000	100.000	5.325	4.725	3.025
1700.000	100.000	5.741	5.141	3.441
1800.000	100.000	6.158	5.558	3.858
1900.000	100.000	6.575	5.975	4.275
2000.000	100.000	6.991	6.391	4.691
2100.000	100.000	7.408	6.808	5.108
2200.000	100.000	7.825 / 8.825	7.225 / 8.225	5.525 / 6.525
2300.000	100.000	9.825	9.225	7.525
2400.000	100.000	10.825	10.225	8.525
2500.000 / 2550.000 / 2700.000	100.000 / 50.000 / 150.000	11.825 / 12.325 / 13.825 / 15.325 / 16.825	11.225 / 11.725 / 13.225 / 14.725 / 16.225	9.525 / 10.025 / 11.525 / 13.025 / 14.525
2800.000	100.000	17.825	17.225	15.525
2900.000	100.000	19.825	18.225	16.525
2973.000	73.200	19.557	18.957	17.257

Q = 29 m³/s | Q = 20 m³/s

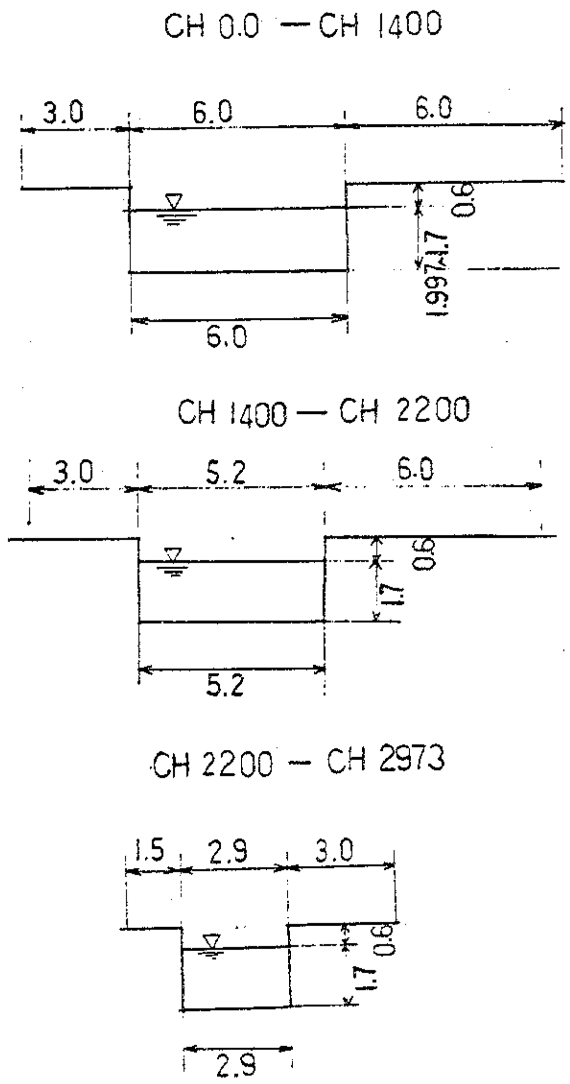
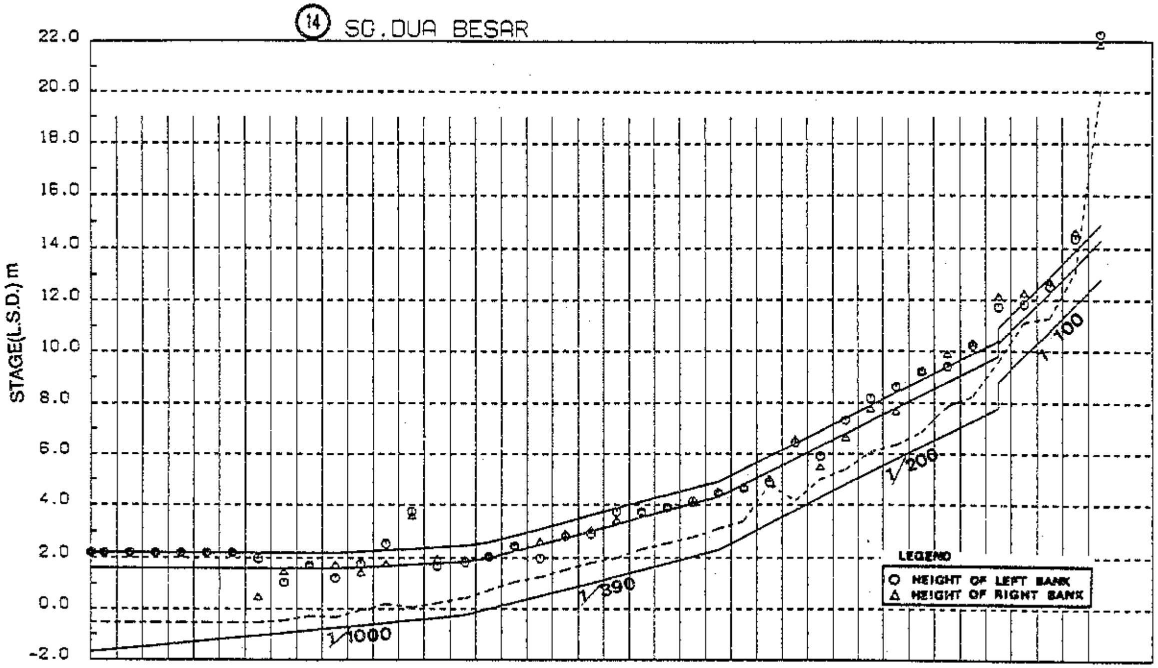


FIG. H-20-12 **PROPOSED LONGITUDINAL PROFILE AND CROSS SECTION OF SG. GELUGOR**

THE STUDY ON FLOOD MITIGATION AND DRAINAGE IN PENANG ISLAND

JICA



CHAINAGE	DISTANCE	PROPOSED BANK LEVEL	DESIGN H.W.L.	PROPOSED BED LEVEL
-650.000	0.000	2.200	1.600	-1.700
-600.000	50.000	2.200	1.600	-1.650
-500.000	100.000	2.200	1.600	-1.550
-400.000	100.000	2.200	1.600	-1.450
-300.000	100.000	2.200	1.600	-1.350
-200.000	100.000	2.200	1.600	-1.250
-100.000	100.000	2.200	1.600	-1.150
0.000	100.000	2.200	1.600	-1.050
100.000	100.000	2.200	1.600	-0.950
200.000	100.000	2.200	1.600	-0.850
300.000	100.000	2.200	1.600	-0.750
400.000	100.000	2.241	1.641	-0.650
500.000	100.000	2.295	1.695	-0.550
600.000	100.000	2.348	1.748	-0.450
700.000	100.000	2.402	1.802	-0.350
808.000	107.500	2.459	1.859	-0.242
900.000	92.500	2.595	1.995	-0.005
1000.000	100.000	2.851	2.251	0.251
1100.000	100.000	3.108	2.508	0.508
1200.000	100.000	3.364	2.764	0.764
1300.000	100.000	3.620	3.020	1.020
1400.000	100.000	3.877	3.277	1.277
1500.000	100.000	4.133	3.533	1.533
1600.000	100.000	4.390	3.790	1.790
1700.000	100.000	4.646	4.046	2.046
1800.000	100.000	4.902	4.302	2.302
1900.000	100.000	5.402	4.802	2.802
2000.000	100.000	5.902	5.302	3.302
2100.000	100.000	6.402	5.802	3.802
2200.000	100.000	6.902	6.302	4.302
2300.000	100.000	7.402	6.802	4.802
2400.000	100.000	7.902	7.302	5.302
2500.000	100.000	8.402	7.802	5.802
2600.000	100.000	8.902	8.302	6.302
2700.000	100.000	9.402	8.802	6.802
2800.000	100.000	9.902	9.302	7.302
2900.000	100.000	10.402 / 10.902	9.802 / 10.302	7.802 / 8.802
3000.000	100.000	11.902	11.302	9.802
3100.000	100.000	12.902	12.302	10.802
3200.000	100.000	13.902	13.302	11.802
3300.000	100.000	14.902	14.302	12.802

Q= 55m³/s (CH -650 – CH 808) ; Q=30m³/s (CH 808 – CH 1800) ; Q=27m³/s (CH 1800 – CH 2900) ; Q=7m³/s (CH 2900 – CH 3300)

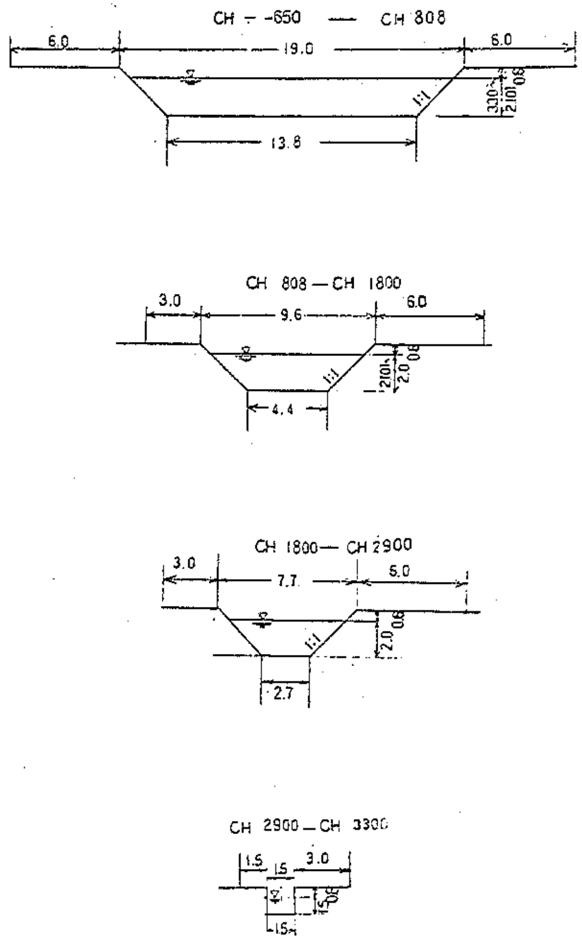
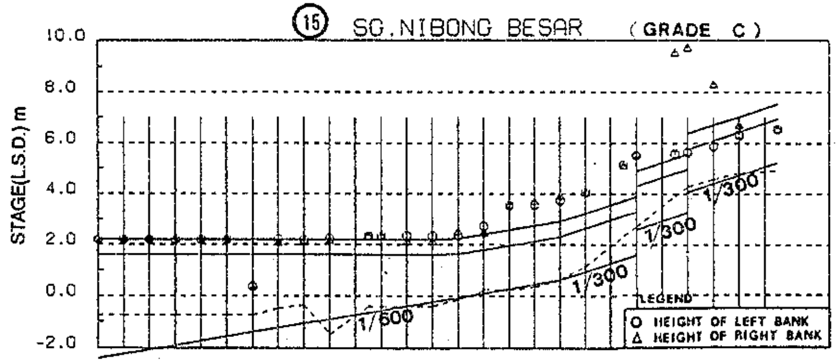


FIG. H-20-13 **PROPOSED LONGITUDINAL PROFILE AND CROSS SECTION OF SG. DUA BESAR**

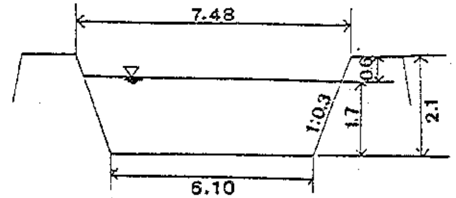
THE STUDY ON FLOOD MITIGATION AND DRAINAGE IN PENANG ISLAND



CHAINAGE	DISTANCE	PROPOSED BANK LEVEL	DESIGN H.W.L.	PROPOSED BED LEVEL
-600.000	0.000	2.200	1.600	-2.400
-500.000	100.000	2.200	1.600	-2.233
-400.000	100.000	2.200	1.600	-2.067
-300.000	100.000	2.200	1.600	-1.900
-200.000	100.000	2.200	1.600	-1.733
-100.000	100.000	2.200	1.600	-1.557
0.000	100.000	2.200	1.600	-1.400
100.000	100.000	2.200	1.600	-1.233
200.000	100.000	2.200	1.600	-1.067
300.000	100.000	2.200	1.600	-0.900
450.000 / 500.000	150.000 / 50.000	2.200 / 2.200	1.600 / 1.600	-0.650 / -0.567
600.000	100.000	2.200	1.600	-0.400
700.000	100.000	2.200	1.600	-0.233
800.000	100.000	2.233	1.633	-0.067
900.000	100.000	2.400	1.800	0.100
1000.000	100.000	2.567	1.967	0.267
1100.000	100.000	2.733	2.133	0.433
1200.000	100.000	2.900	2.300	0.600
1300.000	100.000	3.233	2.633	0.933
1450.000 / 1500.000	150.000 / 50.000	3.733 / 3.900 / 4.900	3.133 / 3.300 / 4.300	1.433 / 1.600 / 2.600
1650.000 / 1700.000	150.000 / 50.000	5.400 / 5.567 / 6.367	4.800 / 4.867 / 5.767	3.100 / 3.267 / 4.067
1800.000	100.000	6.700	6.100	4.400
1300.000	100.000	7.033	6.433	4.733
1950.000	150.000	7.533	6.933	5.233

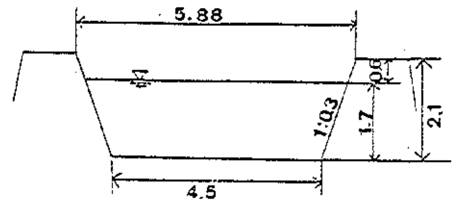
Q = 25m³/s

CH-0.60 ~ CH 0.0
CH 0.60 ~ CH 1.20
SG. NIBONG BESAR



N	L/T	H	A	B	R	RR	V	Q	BB	QK
.020	600.000	1.700	11.237	7.120	1.164	1.107	2.259	25.388	6.100	25.000

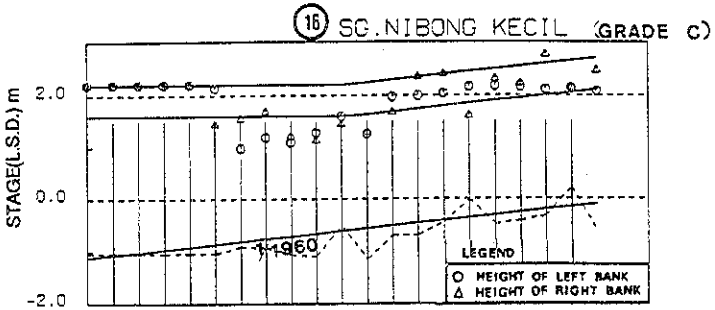
CH 1.20 ~ CH1.65



N	L/T	H	A	B	R	RR	V	Q	BB	QK
.020	300.000	1.700	8.517	5.520	1.058	1.038	2.997	25.529	4.500	25.000

FIG. H-20-14 **PROPOSED LONGITUDINAL PROFILE AND CROSS SECTION OF SG. NIBONG BESAR**

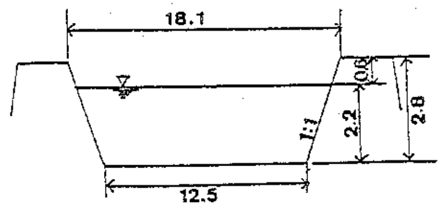
THE STUDY ON FLOOD MITIGATION AND DRAINAGE IN PENANG ISLAND



PROPOSED BED LEVEL	-1.120	-1.069	-1.018	-0.967	-0.916	-0.865	-0.814	-0.763	-0.712	-0.661	-0.610	-0.559	-0.508	-0.457	-0.406	-0.355	-0.304	-0.253	-0.202	-0.151	-0.100
DESIGN H.W.L.	1.600	1.600	1.600	1.600	1.600	1.600	1.600	1.600	1.600	1.600	1.600	1.641	1.692	1.743	1.794	1.845	1.896	1.947	1.998	2.049	2.100
PROPOSED BANK LEVEL	2.200	2.200	2.200	2.200	2.200	2.200	2.200	2.200	2.200	2.200	2.200	2.241	2.292	2.343	2.394	2.445	2.496	2.547	2.598	2.649	2.700
DISTANCE	0.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	98.260
CHAINAGE	-500.000	-400.000	-300.000	-200.000	-100.000	0.000	100.000	200.000	300.000	400.000	500.000	600.000	700.000	800.000	900.000	1000.000	1100.000	1200.000	1300.000	1400.000	1498.000

Q = 42 m³/s

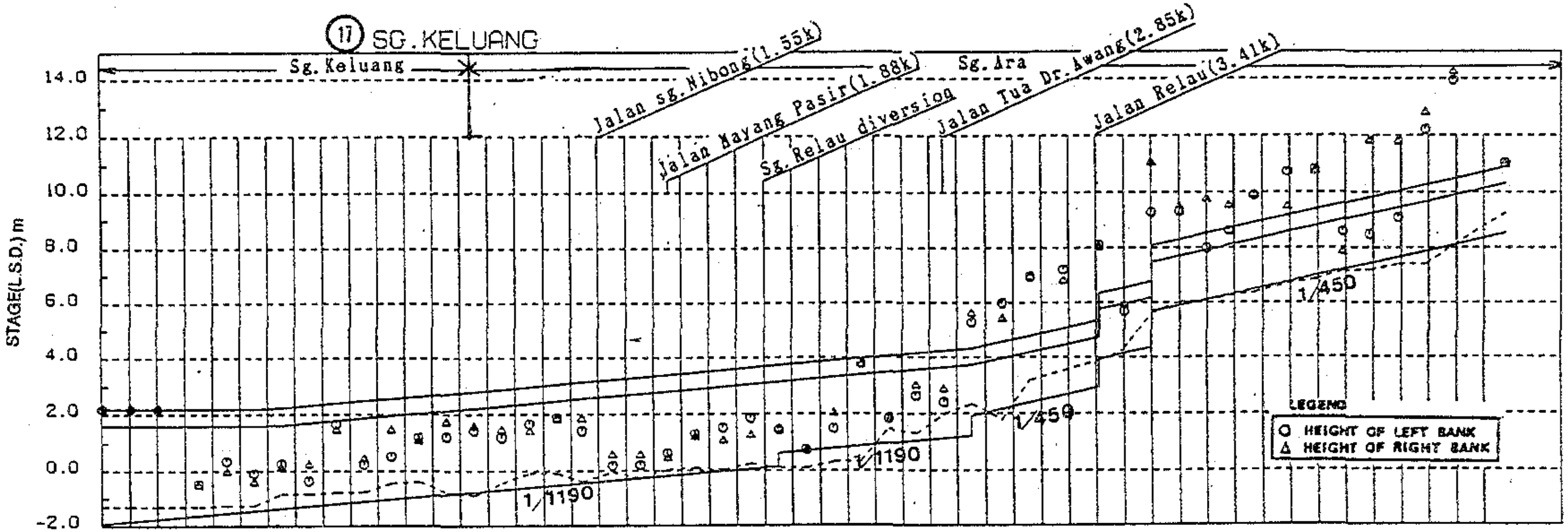
SG. NIBONG KECIL



N	I/I	H	A	B	R	RR	V	Q
.025	1960.000	2.200	32.340	16.900	1.727	1.440	1.301	42.065

FIG. H-20-15 **PROPOSED LONGITUDINAL PROFILE AND CROSS SECTION OF SG. NIBONG KECIL**

THE STUDY ON FLOOD MITIGATION AND DRAINAGE IN PENANG ISLAND



CHAINAGE	DISTANCE	PROPOSED BANK LEVEL	DESIGN H.W.L.	PROPOSED BED LEVEL
-200.000	0.000	2.200	1.600	-1.920
-100.000	100.000	2.200	1.600	-1.836
0.000	100.000	2.200	1.600	-1.762
150.000	150.000	2.200	1.600	-1.626
250.000	100.000	2.200	1.600	-1.542
350.000	100.000	2.200	1.600	-1.458
450.000	100.000	2.226	1.626	-1.374
550.000	100.000	2.310	1.710	-1.290
650.000	100.000	2.394	1.794	-1.206
750.000	100.000	2.478	1.878	-1.122
850.000	100.000	2.562	1.962	-1.038
950.000	100.000	2.646	2.046	-0.954
1050.000	100.000	2.730	2.130	-0.870
1150.000	100.000	2.814	2.214	-0.786
1250.000	100.000	2.898	2.298	-0.702
1350.000	100.000	2.983	2.383	-0.617
1450.000	100.000	3.067	2.467	-0.533
1540.000	90.000	3.142	2.542	-0.468
1650.000	110.000	3.235	2.635	-0.365
1750.000	100.000	3.319	2.719	-0.281
1850.000	100.000	3.403	2.803	-0.197
1950.000	100.000	3.487	2.887	-0.113
2050.000	100.000	3.571	2.971	-0.029
2150.000	100.000	3.655	3.055	0.055
2250.000	100.000	3.738 / 3.738	3.138 / 3.138	0.138 / 0.638
2350.000	100.000	3.823	3.223	0.723
2450.000	100.000	3.907	3.307	0.807
2550.000	100.000	3.991	3.391	0.891
2650.000	100.000	4.075	3.475	0.975
2750.000	100.000	4.159	3.559	1.059
2850.000	100.000	4.243	3.643	1.143
2950.000	100.000	4.327 / 4.327	3.727 / 3.727	1.227 / 1.827
3060.000	110.000	4.572	3.872	2.172
3160.000	100.000	4.794	4.194	2.394
3280.000	120.000	5.060	4.460	2.660
3410.000	130.000	6.348 / 6.348	4.748 / 5.748	2.848 / 3.848
3500.000	90.000	6.549	5.949	4.149
3600.000	100.000	6.772 / 8.072	6.172 / 7.472	4.372 / 5.672
3700.000	100.000	8.294	7.694	5.894
3800.000	100.000	8.516	7.916	6.116
3880.000	80.000	8.694	8.094	6.294
3970.000	90.000	8.894	8.294	6.494
4090.000	120.000	9.160	8.560	6.760
4190.000	100.000	9.383	8.783	6.983
4290.000	100.000	9.605	9.005	7.205
4385.000	95.000	9.818	9.218	7.418
4490.000	104.000	10.049	9.449	7.649
4590.000	100.000	10.272	9.672	7.872
4690.000	100.000	10.494	9.894	8.094
4879.000	189.000	10.914	10.914	8.514

Q = 125 m³/s (chainage -200.000 to 1540.000)
Q = 110 m³/s (chainage 1540.000 to 2250.000)
Q = 40 m³/s (chainage 2250.000 to 3410.000)
Q = 35 m³/s (chainage 3410.000 to 4879.000)

FIG. H-20-16 LONGITUDINAL PROFILE OF SG. KELUANG

THE STUDY ON FLOOD MITIGATION AND DRAINAGE IN PENANG ISLAND

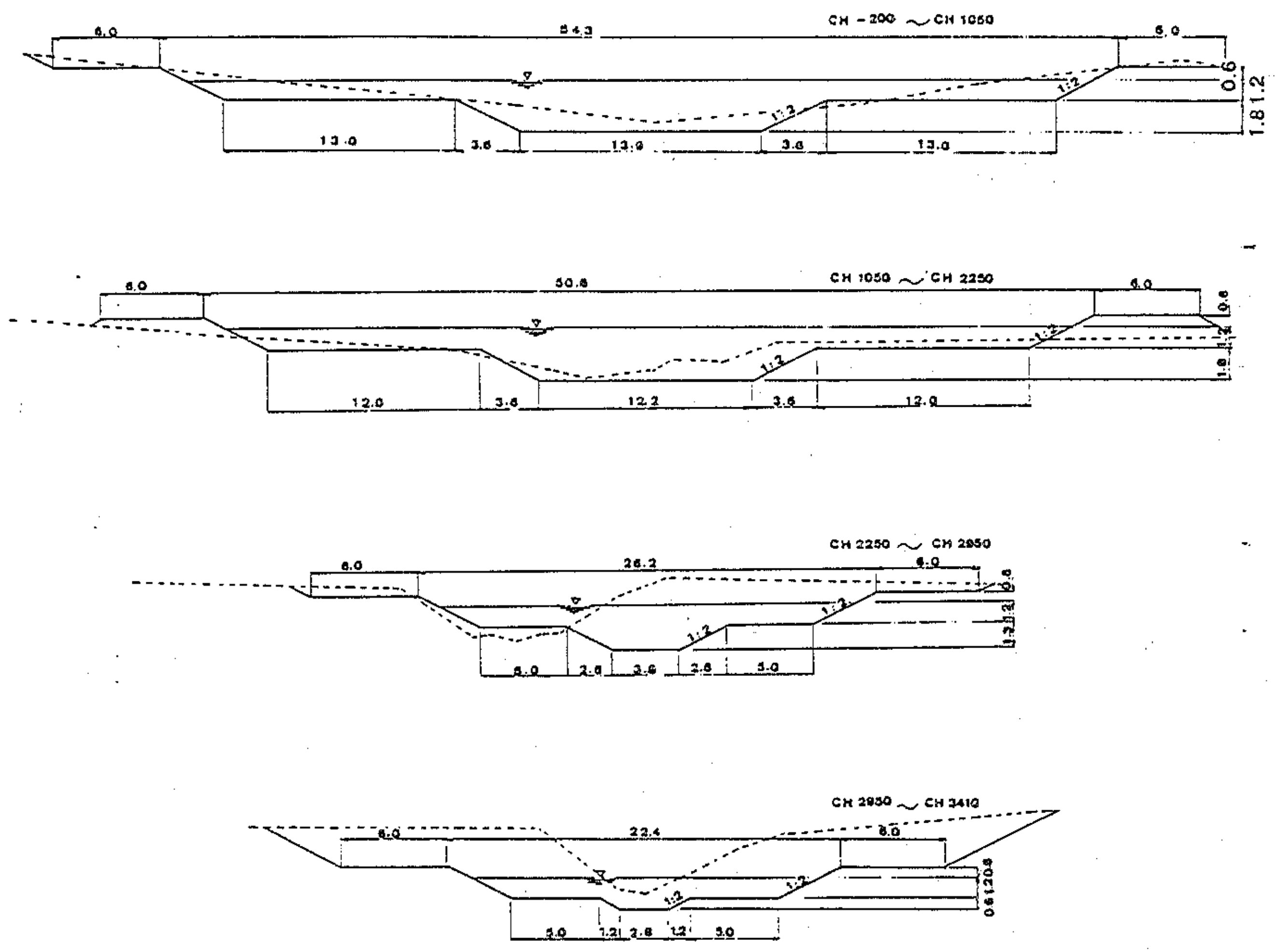
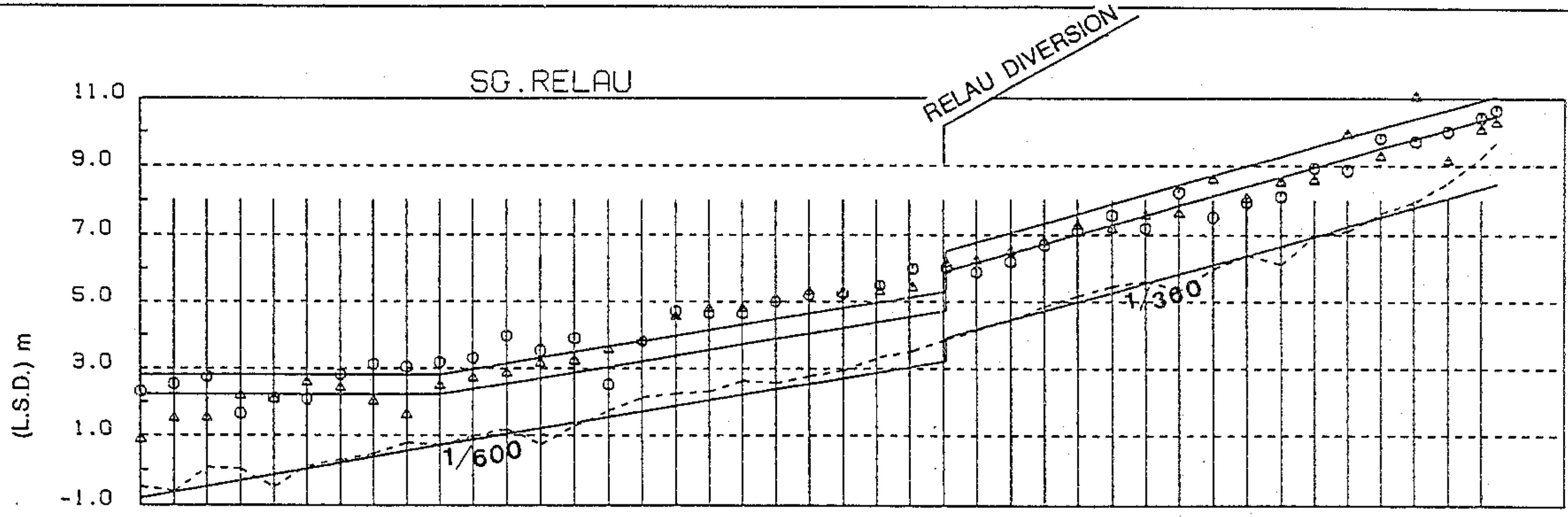


FIG. H-20-17	PROPOSED CROSS SECTIONS OF SG. KELUANG

THE STUDY ON FLOOD MITIGATION AND DRAINAGE IN PENANG ISLAND

JICA



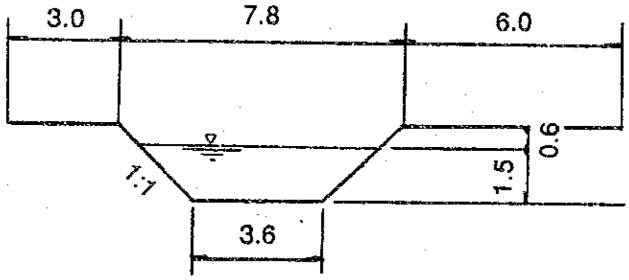
CHAINAGE	DISTANCE	PROPOSED BANK LEVEL	DESIGN H.W.L.	PROPOSED BED LEVEL
0.000	0.000	2.814	2.214	-0.786
100.000	100.000	2.814	2.214	-0.619
200.000	100.000	2.814	2.214	-0.453
300.000	100.000	2.814	2.214	-0.286
400.000	100.000	2.814	2.214	-0.119
500.000	100.000	2.814	2.214	0.047
600.000	100.000	2.814	2.214	0.214
700.000	100.000	2.814	2.214	0.381
800.000	100.000	2.814	2.214	0.547
900.000	100.000	2.814	2.214	0.714
1000.000	100.000	2.981	2.381	0.881
1100.000	100.000	3.147	2.547	1.047
1200.000	100.000	3.314	2.714	1.214
1300.000	100.000	3.481	2.881	1.381
1400.000	100.000	3.647	3.047	1.547
1500.000	100.000	3.814	3.214	1.714
1600.000	100.000	3.981	3.381	1.881
1700.000	100.000	4.147	3.547	2.047
1800.000	100.000	4.314	3.714	2.214
1900.000	100.000	4.481	3.881	2.381
2000.000	100.000	4.647	4.047	2.547
2100.000	100.000	4.814	4.214	2.714
2210.000	110.000	4.997	4.397	2.897
2310.000	100.000	5.164	4.564	3.064
2410.000	100.000	5.331 6.530	4.731 5.930	3.231 3.830
2500.000	90.000	6.780	6.180	4.180
2600.000	100.000	7.057	6.457	4.457
2700.000	100.000	7.335	6.735	4.735
2800.000	100.000	7.613	7.013	5.013
2900.000	100.000	7.891	7.291	5.291
3000.000	100.000	8.169	7.569	5.569
3100.000	100.000	8.446	7.846	5.846
3200.000	100.000	8.724	8.124	6.124
3300.000	100.000	9.002	8.402	6.402
3400.000	100.000	9.280	8.680	6.680
3500.000	100.000	9.557	8.957	6.957
3600.000	100.000	9.835	9.235	7.235
3700.000	100.000	10.113	9.513	7.513
3805.000	105.000	10.405	9.805	7.805
3900.000	95.000	10.669	10.069	8.069
4000.000	100.000	10.946	10.346	8.346
4045.120	45.120	11.072	10.472	8.472

12 m³/s (0.000 – 2410.000) — 65 m³/s (2410.000 – 4045.120)

FIG. H-20-18	LONGITUDINAL PROFILE OF SG. RELAU

THE STUDY ON FLOOD MITIGATION AND DRAINAGE IN PENANG ISLAND

CH 0.0 — CH 2410



CH 2410 — CH 4045

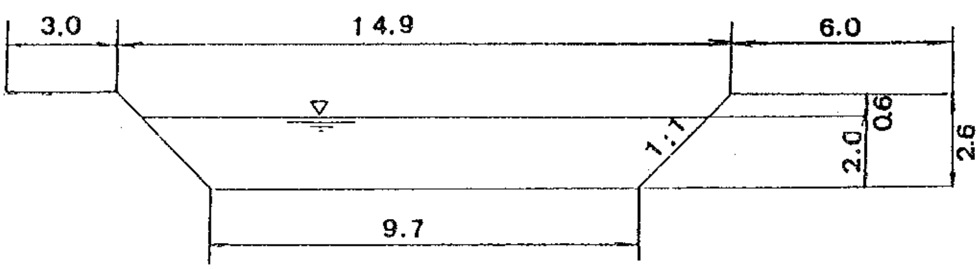
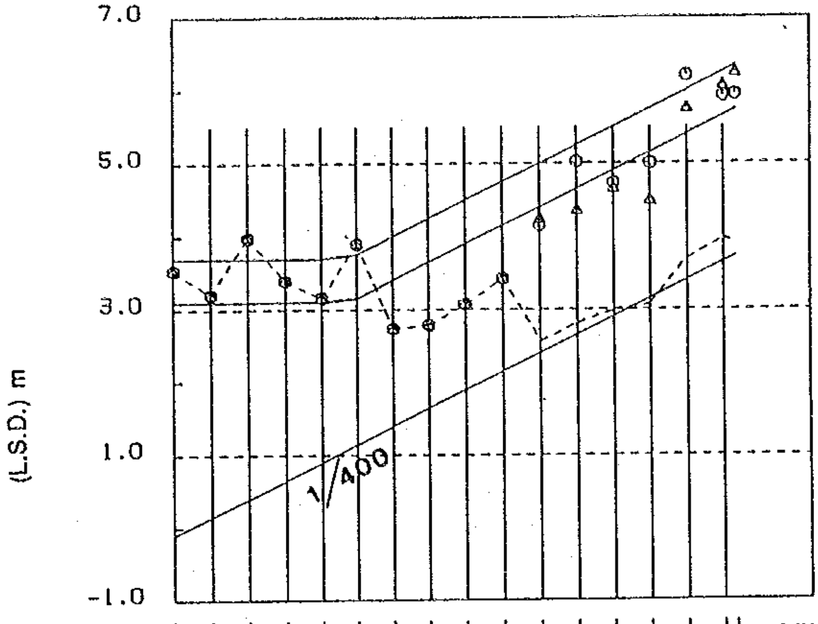


FIG. H-20-19	PROPOSED CROSS SECTIONS OF SG. RELAU

THE STUDY ON FLOOD MITIGATION AND DRAINAGE IN PENANG ISLAND



PROPOSED BED LEVEL	-0.097	0.153	0.403	0.653	0.903	1.153	1.403	1.653	1.903	2.153	2.403	2.653	2.903	3.153	3.403	3.653	3.735
DESIGN H.W.L.	3.097	3.097	3.097	3.097	3.097	3.153	3.403	3.653	3.903	4.153	4.403	4.653	4.903	5.153	5.403	5.653	5.735
PROPOSED BANK LEVEL	3.697	3.697	3.697	3.697	3.697	3.753	4.003	4.253	4.503	4.753	5.003	5.253	5.503	5.753	6.003	6.253	6.335
DISTANCE	0.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	33.000
CHAINAGE	0.000	100.000	200.000	300.000	400.000	500.000	600.000	700.000	800.000	900.000	1000.000	1100.000	1200.000	1300.000	1400.000	1500.000	1533.000

Q=70

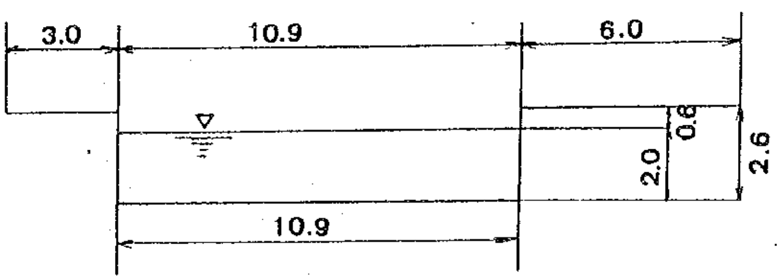
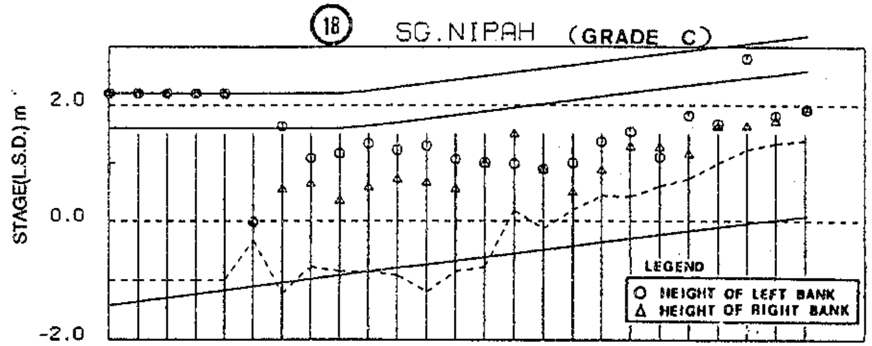


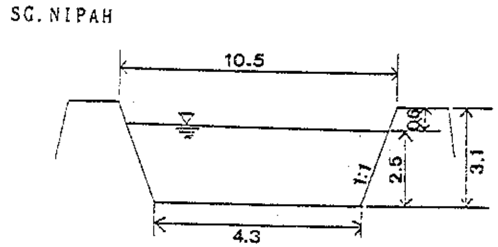
FIG. H-20-20	LONGITUDINAL PROFILE AND PROPOSED CROSS SECTION OF SG. RELAU DIVERSION CHANNEL

THE STUDY ON FLOOD MITIGATION AND DRAINAGE IN PENANG ISLAND



CHAINAGE	DISTANCE	PROPOSED BANK LEVEL	DESIGN H.W.L.	PROPOSED BED LEVEL
-500.000	0.000	2.200	1.600	-1.420
-400.000	100.000	2.200	1.600	-1.357
-300.000	100.000	2.200	1.600	-1.295
-200.000	100.000	2.200	1.600	-1.232
-100.000	100.000	2.200	1.600	-1.170
0.000	100.000	2.200	1.600	-1.107
100.000	100.000	2.200	1.600	-1.045
200.000	100.000	2.200	1.600	-0.982
300.000	100.000	2.200	1.600	-0.920
400.000	100.000	2.243	1.643	-0.857
500.000	100.000	2.305	1.705	-0.795
600.000	100.000	2.368	1.768	-0.732
700.000	100.000	2.430	1.830	-0.670
800.000	100.000	2.493	1.893	-0.607
900.000	100.000	2.555	1.955	-0.545
1000.000	100.000	2.618	2.017	-0.482
1100.000	100.000	2.680	2.080	-0.420
1200.000	100.000	2.743	2.142	-0.357
1300.000	100.000	2.805	2.205	-0.295
1400.000	100.000	2.868	2.267	-0.232
1500.000	100.000	2.930	2.330	-0.170
1600.000	100.000	2.993	2.392	-0.107
1700.000	100.000	3.055	2.455	-0.045
1800.000	100.000	3.118	2.517	0.018
1906.000	106.000	3.184	2.584	0.084

Q = 22 m³/s

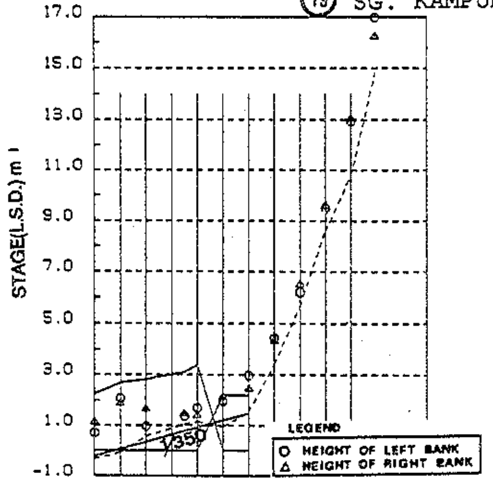


N	1/I	H	A	B	R	RR	V	Q
.025	1600.000	2.500	17.000	8.300	1.495	1.307	1.307	22.227

FIG. H-20-21	**PROPOSED LONGITUDINAL PROFILE AND CROSS SECTION OF SG. NIPAH**

THE STUDY ON FLOOD MITIGATION AND DRAINAGE IN PENANG ISLAND

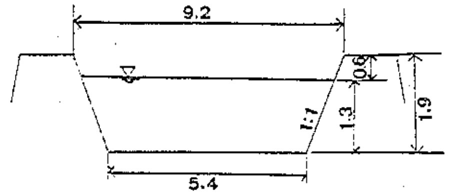
(19) SG. KAMPUNG MASJID (GRADE C)



PROPOSED BED LEVEL	-0.220	0.066	0.351	0.780	0.923	1.209	1.494	0.000	0.000	0.000	0.000	0.000
DESIGN H.W.L.	2.251	2.680	2.823	3.109	3.394	0.000	0.000	0.000	0.000	0.000	0.000	0.000
PROPOSED BANK LEVEL	0.000	0.000	0.000	0.000	0.000	2.200	2.200	0.000	0.000	0.000	0.000	0.000
DISTANCE	0.000	100.000	100.000	150.000	50.000	100.000	100.000	100.000	100.000	100.000	100.000	92.400
CHAINAGE	0.000	100.000	200.000	350.000	400.000	500.000	600.000	700.000	800.000	900.000	1000.000	1092.000

Q = 1.8 m^3/s

SG. KAMPUNG MASJID

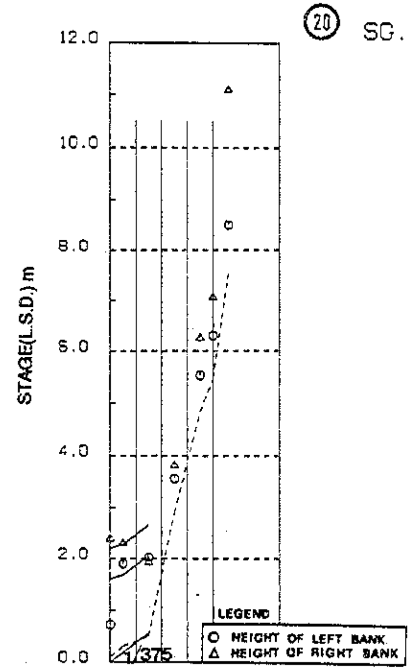


N	I/I	H	A	B	R	RR	V	Q
.025	350.000	1.300	8.710	8.000	.960	.973	2.080	18.117

FIG. H-20-22 **PROPOSED LONGITUDINAL PROFILE AND CROSS SECTION OF SG. KAMPUNG MASJID**

THE STUDY ON FLOOD MITIGATION AND DRAINAGE IN PENANG ISLAND

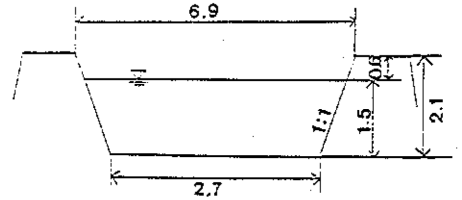
⑳ SG. IKAN MATI (GRADE C)



PROPOSED BED LEVEL	0.000	0.189	0.566	0.000	0.000	0.000	0.000
DESIGN H.W.L.	1.600	1.689	2.066	0.000	0.000	0.000	0.000
PROPOSED BANK LEVEL	2.200	2.269	2.665	0.000	0.000	0.000	0.000
DISTANCE	0.000	50.000	100.000	100.000	100.000	50.000	61.110
CHAINAGE	0.000	50.000	150.000	250.000	350.000	400.000	461.000

Q=12m³/s

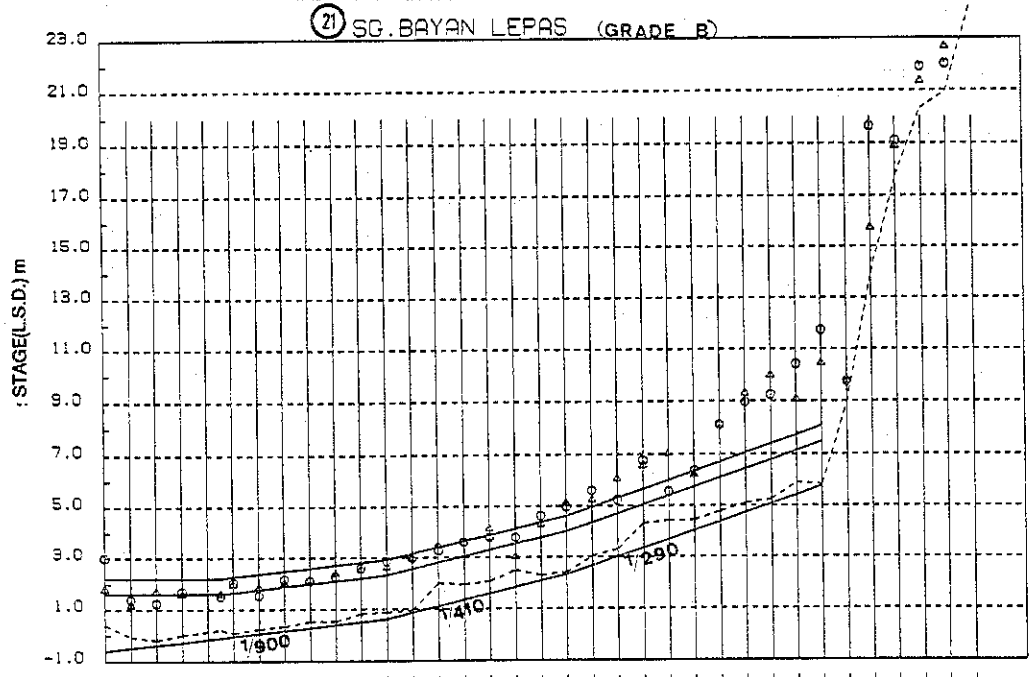
SG. IKAN MATI



N	1/I	H	A	B	R	RR	V	Q
.025	375.000	1.500	6.300	5.700	.907	.937	1.936	12.197

FIG. H-20-23 **PROPOSED LONGITUDINAL PROFILE AND CROSS SECTION OF SG. IKAN MATI**

THE STUDY ON FLOOD MITIGATION AND DRAINAGE IN PENANG ISLAND



PROPOSED BED LEVEL	-0.600	-0.489	-0.378	-0.267	-0.100 -0.044	0.067	0.178	0.289	0.400	0.511	0.622	0.866	1.110	1.354	1.598	1.842	2.086	2.330	2.674	3.019	3.364	3.709	4.054	4.399	4.744	5.089	5.433	5.778	0.000	0.000	0.000	0.000	0.000	0.000
DESIGN H.W.L.	1.600	1.600	1.600	1.600	1.600 1.656	1.767	1.878	1.989	2.100	2.211	2.322	2.566	2.810	3.054	3.298	3.542	3.786	4.030	4.374	4.719	5.064	5.409	5.754	6.099	6.444	6.789	7.133	7.478	0.000	0.000	0.000	0.000	0.000	0.000
PROPOSED BANK LEVEL	2.200	2.200	2.200	2.200	2.200 2.256	2.367	2.478	2.589	2.700	2.811	2.922	3.166	3.410	3.654	3.898	4.142	4.386	4.630	4.974	5.319	5.664	6.009	6.354	6.699	7.044	7.389	7.733	8.078	0.000	0.000	0.000	0.000	0.000	0.000
DISTANCE	0.000	100.000	100.000	100.000	150.000 50.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000
CHAINAGE	0.000	100.000	200.000	300.000	450.000 500.000	600.000	700.000	800.000	900.000	1000.000	1100.000	1200.000	1300.000	1400.000	1500.000	1600.000	1700.000	1800.000	1900.000	2000.000	2100.000	2200.000	2300.000	2400.000	2500.000	2600.000	2700.000	2800.000	2900.000	3000.000	3100.000	3200.000	3300.000	3400.000

Q= 68 m³/s Q=54 m³/s Q= 28 m³/s

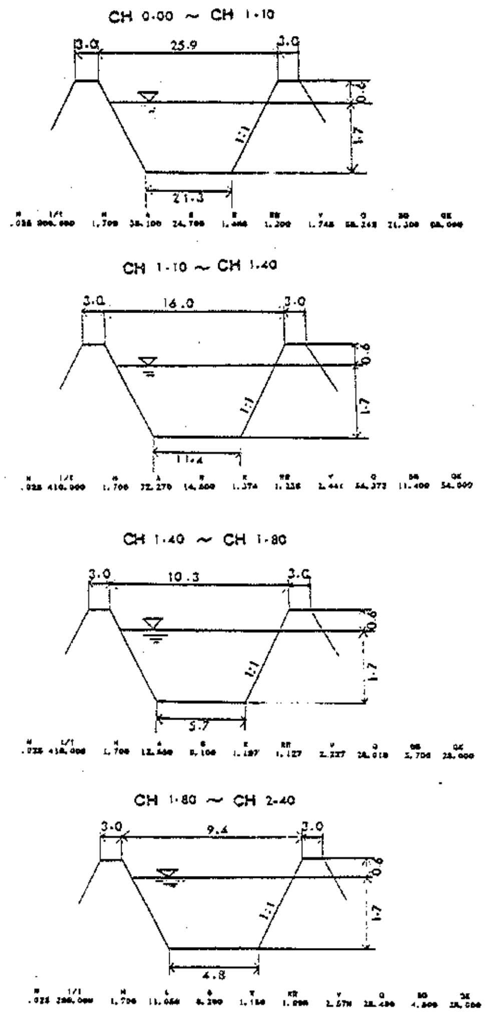
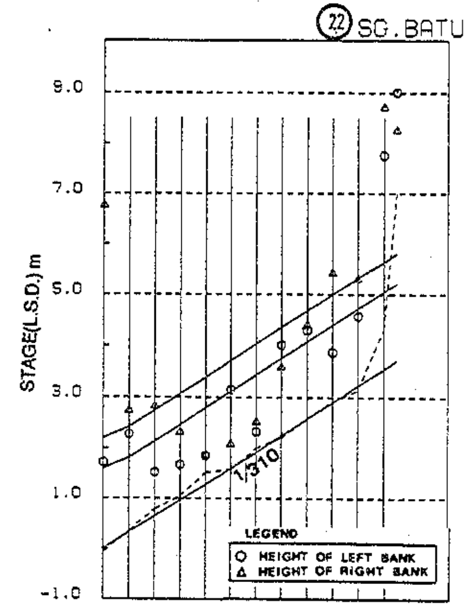


FIG. H-20-24 **PROPOSED LONGITUDINAL PROFILE AND CROSS SECTION OF SG. BAYAN LEPAS**

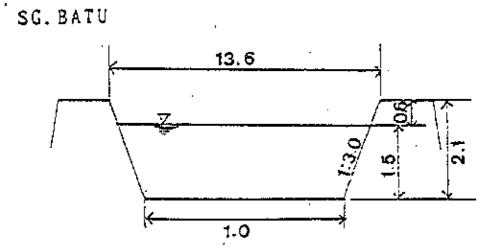
THE STUDY ON FLOOD MITIGATION AND DRAINAGE IN PENANG ISLAND

JICA



PROPOSED BED LEVEL	0.000	0.323	0.645	0.968	1.290	1.613	1.935	2.258	2.581	2.903	3.226	3.548	3.710
DESIGN H.W.L.	1.600	1.823	2.145	2.468	2.790	3.113	3.435	3.758	4.081	4.403	4.726	5.048	5.210
PROPOSED BANK LEVEL	2.200	2.423	2.745	3.068	3.390	3.713	4.035	4.358	4.681	5.003	5.326	5.648	5.810
DISTANCE	0.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	50.000
CHAINAGE	0.000	100.000	200.000	300.000	400.000	500.000	600.000	700.000	800.000	900.000	1000.000	1100.000	1150.000

$Q = 12 m^3/s$

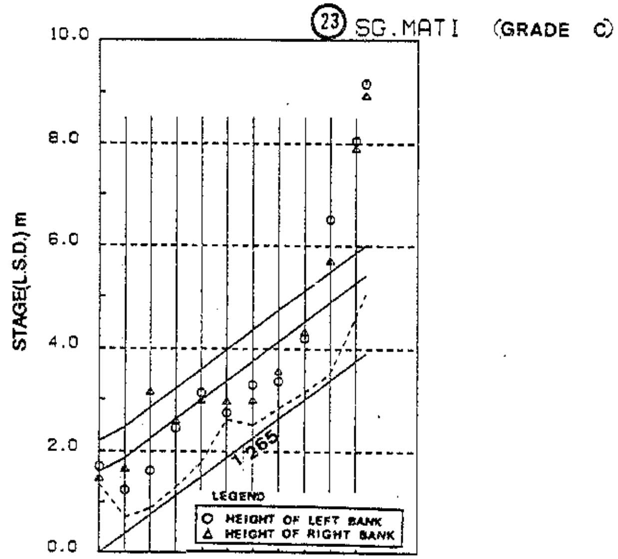


N	I/I	H	A	B	R	RR	V	Q
.030	310.000	1.500	8.250	10.000	.787	.852	1.613	13.310

FIG. H-20-25	**PROPOSED LONGITUDINAL PROFILE AND CROSS SECTION OF SG. BATU**

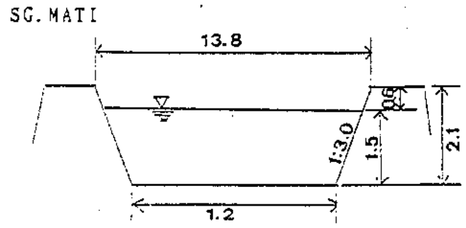
THE STUDY ON FLOOD MITIGATION AND DRAINAGE IN PENANG ISLAND

JICA



PROPOSED BED LEVEL	0.000	0.377	0.755	1.132	1.509	1.887	2.264	2.642	3.019	3.396	3.774 3.916
DESIGN H.W.L.	1.600	1.877	2.255	2.632	3.009	3.387	3.764	4.142	4.519	4.896	5.274 5.416
PROPOSED BANK LEVEL	2.200	2.477	2.855	3.232	3.609	3.987	4.364	4.742	5.119	5.496	5.874 6.016
DISTANCE	0.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000 37.800
CHAINAGE	0.000	100.000	200.000	300.000	400.000	500.000	600.000	700.000	800.000	900.000	1000.000 1037.800

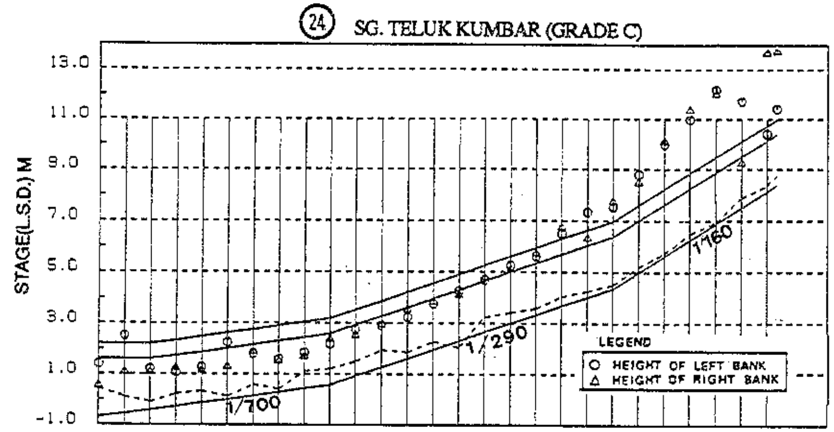
$Q = 25 m^3/s$



N	L/I	H	A	B	R	RR	V	Q
.030	265.000	1.500	8.550	10.200	.800	.862	1.755	15.085

FIG. H-20-26 **PROPOSED LONGITUDINAL PROFILE AND CROSS SECTION OF SG. MATI**

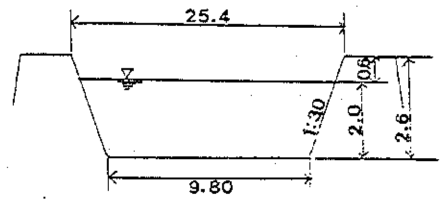
THE STUDY ON FLOOD MITIGATION AND DRAINAGE IN PENANG ISLAND



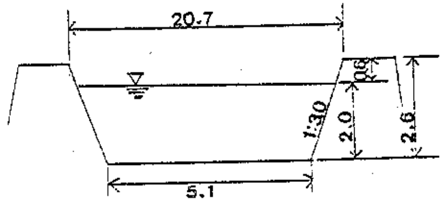
PROPOSED BED LEVEL	-0.700	-0.557	-0.414	-0.271	-0.129	0.014	0.157	0.300	0.443	0.586	0.931	1.275	1.620	1.965	2.310	2.655	3.000	3.344	3.689	4.034	4.379	5.004	5.629	6.254	6.879	7.504	8.129	8.378
DESIGN H.W.L.	1.600	1.600	1.600	1.729	1.871	2.014	2.157	2.300	2.443	2.586	2.931	3.275	3.620	3.965	4.310	4.655	5.000	5.344	5.689	6.034	6.379	7.004	7.629	8.254	8.879	9.504	10.129	10.378
PROPOSED BANK LEVEL	2.200	2.200	2.200	2.329	2.471	2.614	2.757	2.900	3.043	3.186	3.531	3.875	4.220	4.565	4.910	5.255	5.600	5.944	6.289	6.634	6.979	7.604	8.229	8.854	9.479	10.104	10.729	10.978
DISTANCE	0.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000	33.850
CHAINAGE	0.000	100.000	200.000	300.000	400.000	500.000	600.000	700.000	800.000	900.000	1000.000	1100.000	1200.000	1300.000	1400.000	1500.000	1600.000	1700.000	1800.000	1900.000	2000.000	2100.000	2200.000	2300.000	2400.000	2500.000	2600.000	2633.850

Q = 50 m³/s

SG. TELUK KUMBAR



N	1/I	H	A	B	R	RR	V	Q	BB	QK
.030	700.000	2.000	21.600	21.800	1.408	1.256	1.582	50.065	9.800	50.000

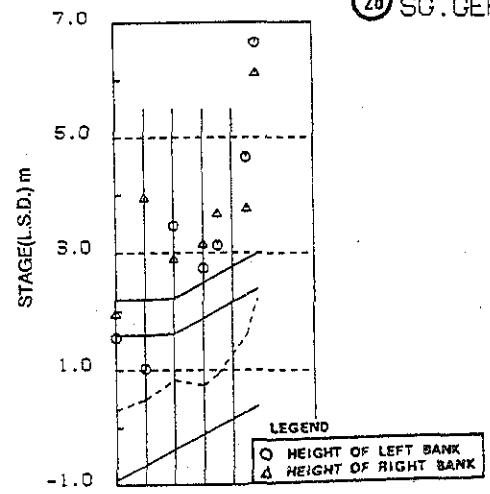


N	1/I	H	A	B	R	RR	V	Q	BB	QK
.030	290.000	2.000	22.200	17.100	1.251	1.161	2.272	50.445	5.100	50.000

FIG. H-20-27 **PROPOSED LONGITUDINAL PROFILE AND CROSS SECTION OF SG. TELUK KUMBAR**

THE STUDY ON FLOOD MITIGATION AND DRAINAGE IN PENANG ISLAND

(26) SG. GERTAK SANGGUL



PROPOSED BED LEVEL	-0.920	-0.650	-0.379	-0.109	0.026	0.296	0.387
DESIGN H.W.L.	1.600	1.600	1.621	1.891	2.026	2.296	2.387
PROPOSED BANK LEVEL	2.200	2.200	2.221	2.491	2.626	2.696	2.987
DISTANCE	0.000	100.000	100.000	100.000	50.000	100.000	33.500
CHAINAGE	0.000	100.000	200.000	300.000	350.000	450.000	483.500

Q=15 m³/s

FIG. H-20-28	PROPOSED LONGITUDINAL PROFILE AND CROSS SECTION OF SG. GERTAK SANGGUL

THE STUDY ON FLOOD MITIGATION AND DRAINAGE IN PENANG ISLAND

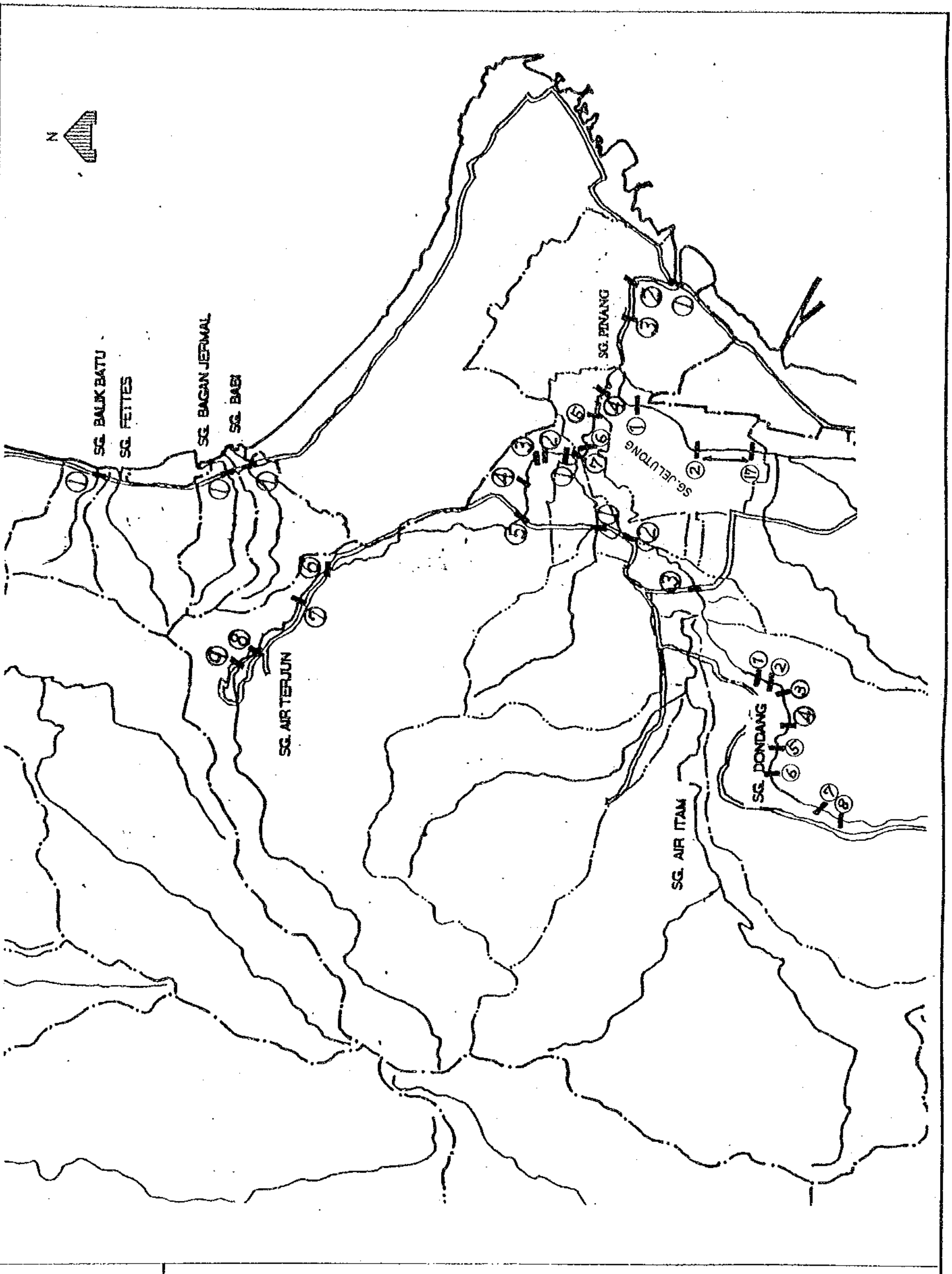


FIG. H-21-1 **LOCATION OF BRIDGES TO BE RECONSTRUCTED IN GEORGETOWN AREA**

THE STUDY ON FLOOD MITIGATION AND DRAINAGE IN PENANG ISLAND

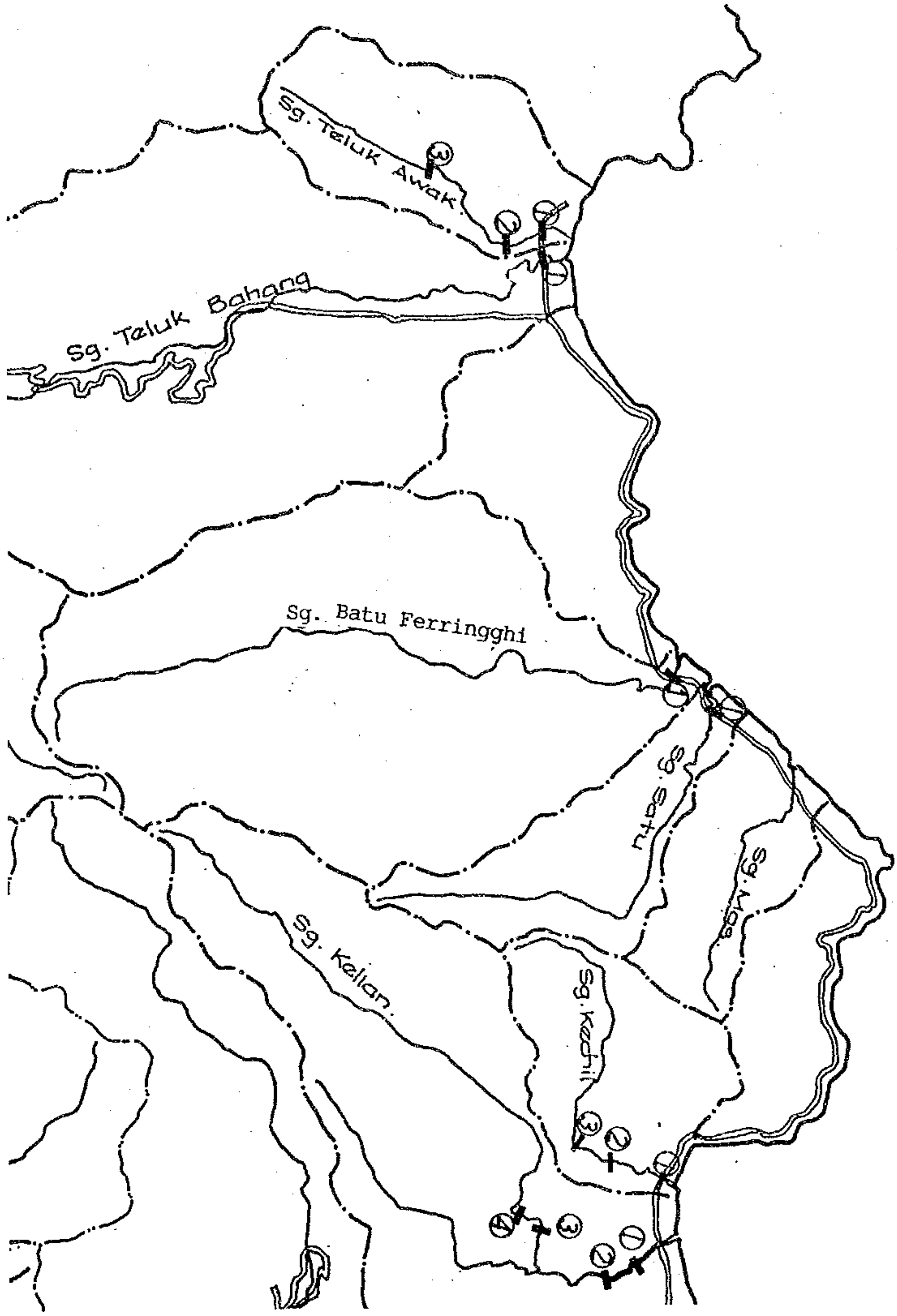


FIG. H-21-2 **LOCATION OF BRIDGES TO BE RECONSTRUCTED IN NORTH COAST**

THE STUDY ON FLOOD MITIGATION AND DRAINAGE IN PENANG ISLAND

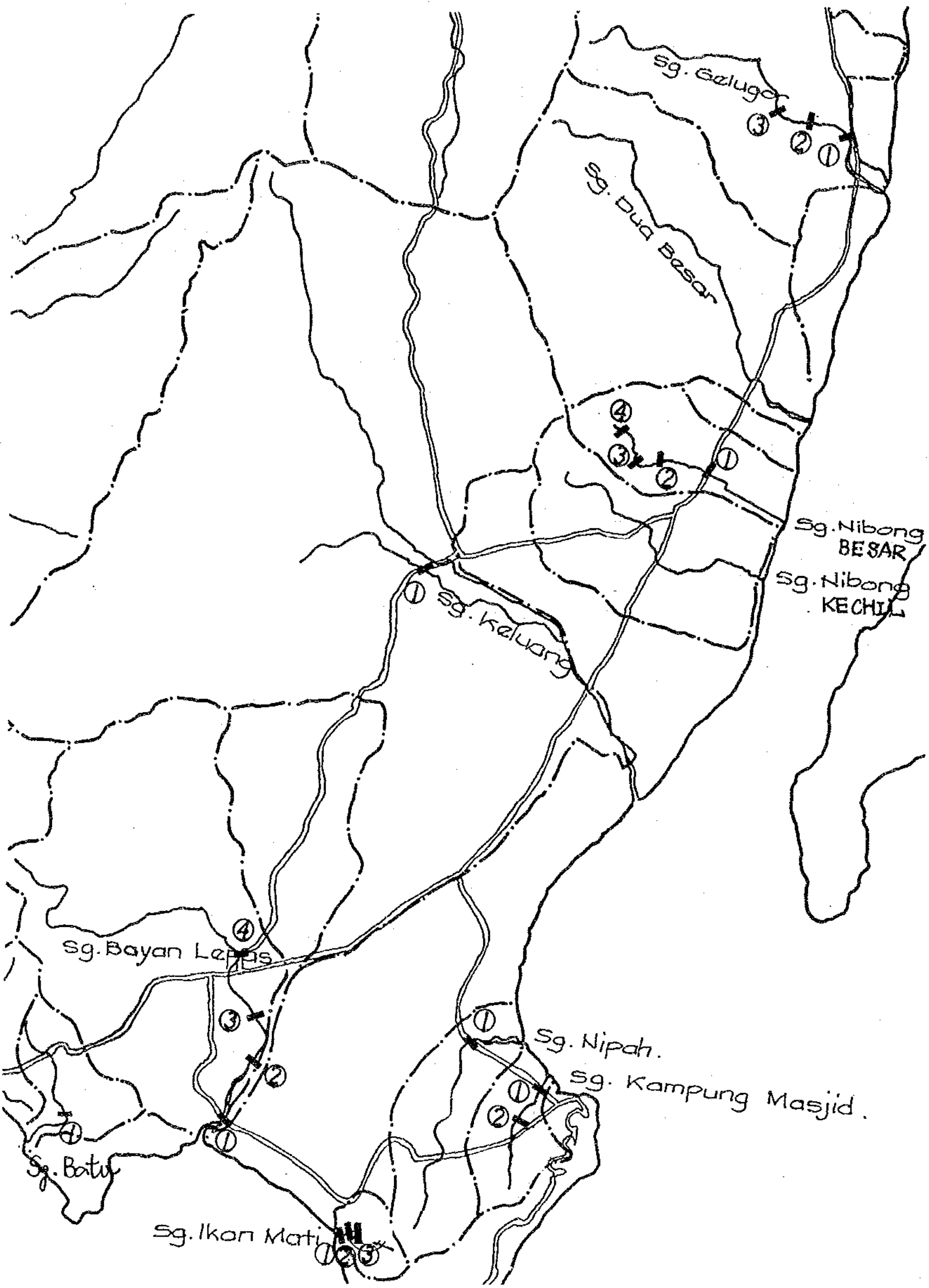


FIG. H-21-3	LOCATION OF BRIDGES TO BE RECONSTRUCTED IN EAST COAST

THE STUDY ON FLOOD MITIGATION AND DRAINAGE IN PENANG ISLAND

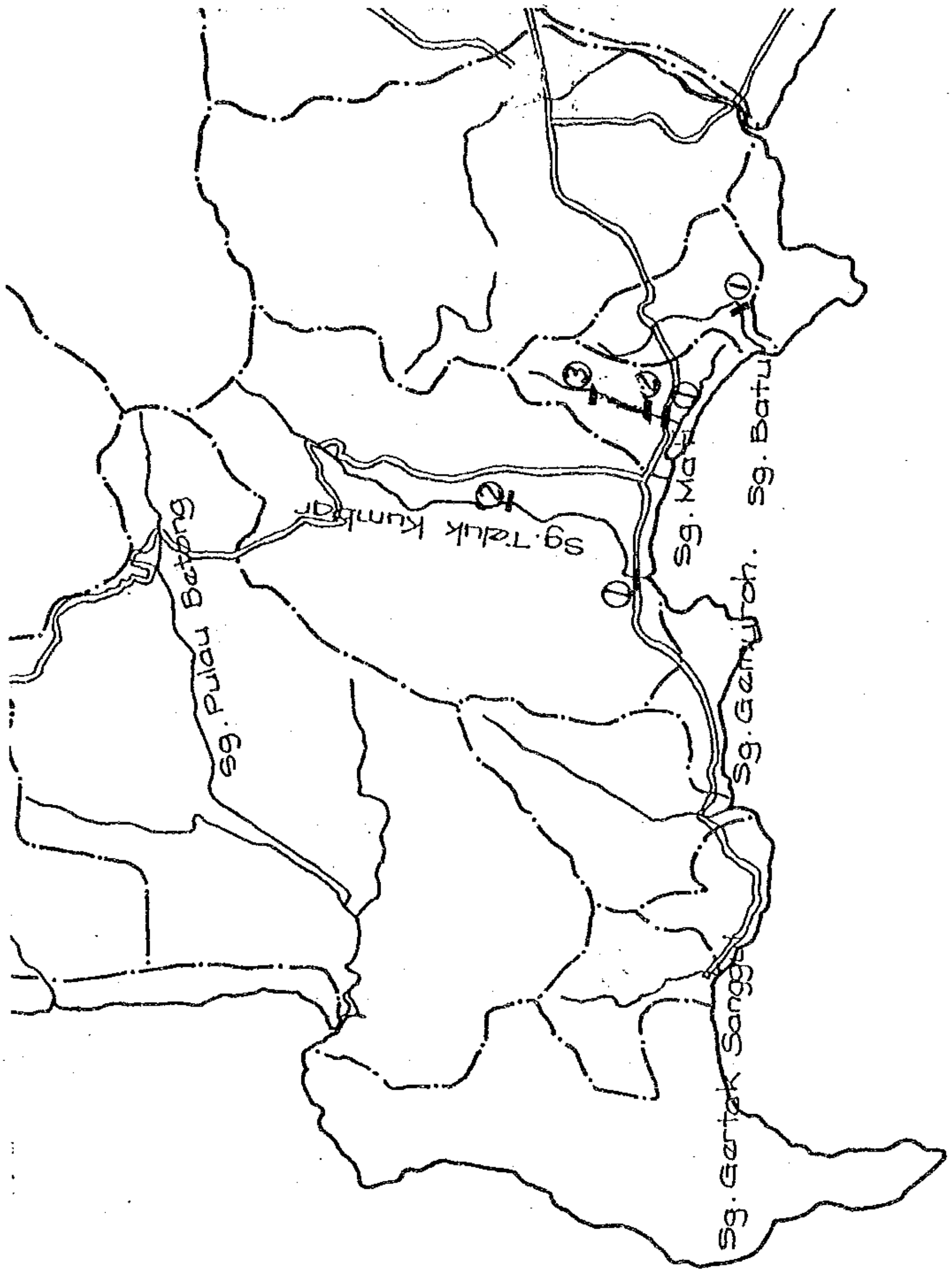


FIG. H-21-4 **LOCATION OF BRIDGES TO BE RECONSTRUCTED IN SOUTH COAST**

THE STUDY ON FLOOD MITIGATION AND DRAINAGE IN PENANG ISLAND

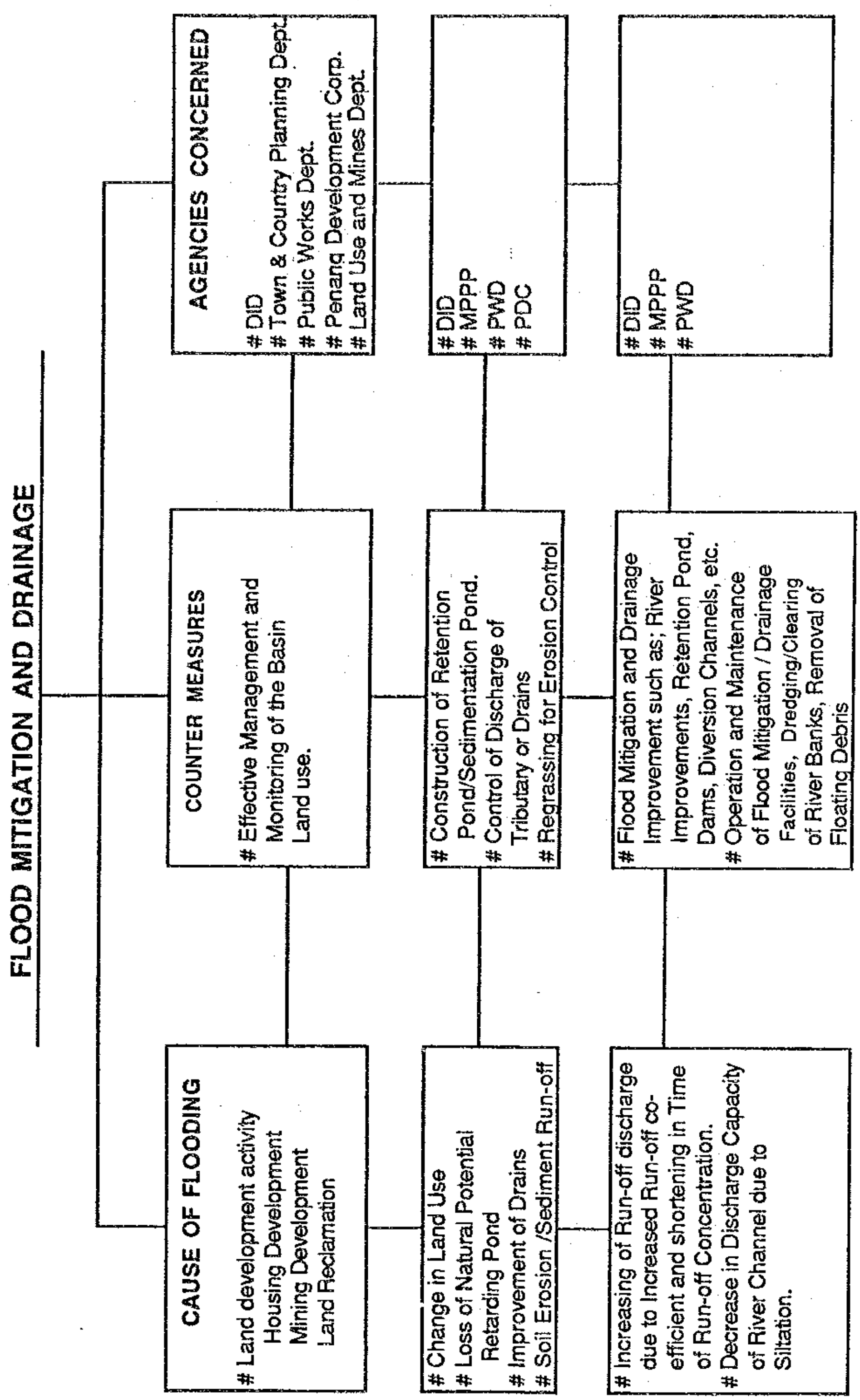


Fig. H-22 ORGANIZATIONAL CONCEPT OF FLOOD MITIGATION AND DRAINAGE

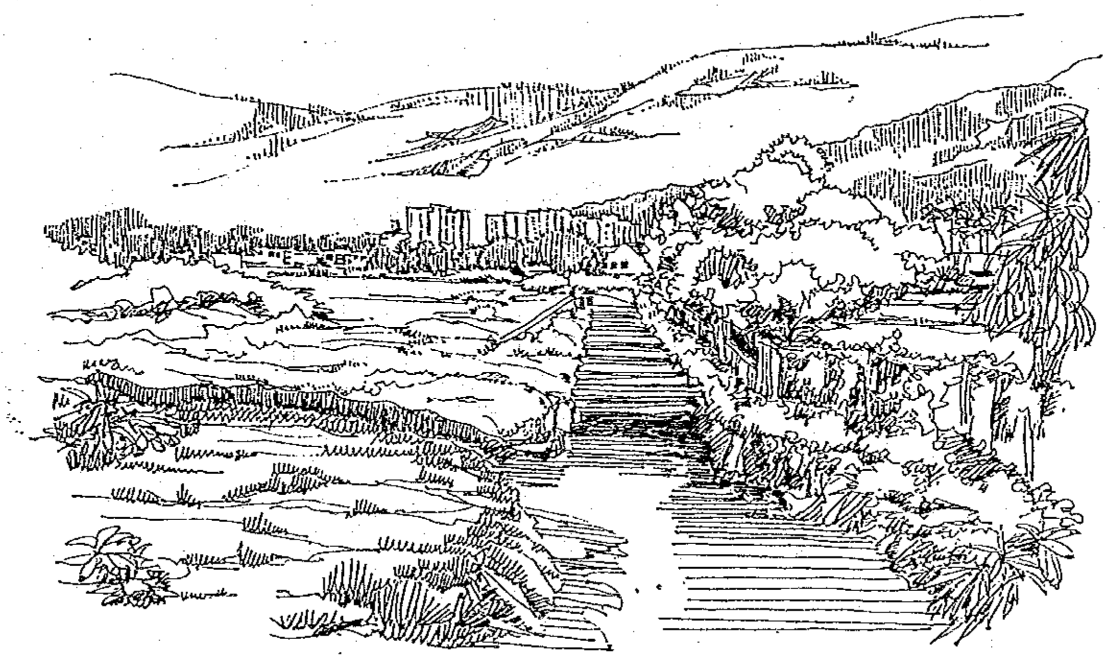
THE STUDY ON FLOOD MITIGATION AND DRAINAGE IN PENANG ISLAND

Fig. H-23 IMPLEMENTATION SCHEDULE FOR FLOOD MITIGATION MASTER PLAN

Phase		I (Urgent Project)					II (Mid-Term Plan)					III (Long-Term Plan)									
Rivers	Year	'91	'92	'93	'94	'95	'96	'97	'98	'99	2000	'01	'02	'03	'04	'05	'06	'07	'08	'09	2010
(Rivers)																					
1. Sg. Pinang,	3.15 km	—	—	—																	
2. Sg. Jelutong,	2.14 km	—	—	—	—	—															
3. Sg. Air Itam,	3.00 km	—	—	—	—																
4. Sg. Dondang,	5.30 km	—	—	—	—		—	—													
- Air Terjun diversion,	1.74 km	—	—	—	—																
- Dondang retention ponds	8.4 ha	—	—	—	—	—															
5. Sg. Keluang	3.38 km	—	—	—	—	—															
6. Sg. Ara	1.87km	—	—	—	—	—															
- Relau diversion channel	1.53 km	—	—	—	—	—															
7. Sg. Air Terjun	2.20 km						—	—													
8. Sg. Gelugor	2.10km					—	—	—													
9. Sg. Dua Besar	3.30 km				—	—	—	—	—	—	—										
10. Sg. Fettes	0.60 km						—	—													
11. Sg. Bayan Lepas	2.40 km							—	—	—											
12. Sg. Teluk Bahang	3.13 km							—	—	—											
13. Sg. Teluk Awak	2.10 km								—	—	—										
14. Sg. Mas	0.60 km								—	—											
15. Sg. Bagan Jermal	0.30 km											—	—								
16. Sg. Nibong Besar	1.05 km											—	—								
17. Sg. Nibong Kecil	0.90 km									—	—										
18. Sg. Kampung Masjid	0.60 km													—	—						
19. Sg. Nipah	1.90 km											—	—								
20. Sg. Batu Ferringghi	0.40 km											—	—								
21. Sg. Satu	0.50 km											—	—								
22. Sg. Kecil	0.70 km												—	—							
23. Sg. Kelian	2.80 km												—	—	—						
24. Sg. Balik batu	0.50 km														—	—					
25. Sg. Babi	1.00 km																—	—			
26. Sg. Ikan Mati	0.15 km																			—	—
27. Sg. Batu	1.00 km																		—	—	
28. Sg. Mati	0.80 km																		—	—	
29. Sg. Teluk Kumbar	1.70 km																		—	—	—

APPENDIX I

FORMULATION OF DRAINAGE MASTER PLAN



APPENDIX I FORMULATION OF DRAINAGE MASTER PLAN

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APPENDIX I FORMULATION OF DRAINAGE MASTER PLAN

1. INTRODUCTION

The Drainage Master Plan Study for Penang Island was carried out by MPPP in 1985, and the results of the study were compiled as technical report No. 14 of Penang Island Structure Plan.

This Master Plan mainly covers the drainage systems in Georgetown which discharge via 45 man-made outfalls to the North and South channels or directly to the rivers. (see Fig. I-1)

Among these main drains, N-20 (Sg. Babi), N-21 (Sg. Bagan Jermal and S-23 (Sg. Jelutong Diversion) were treated as a river.

The reasons for reviewing the previous Drainage Master Plan are as follows:

i) For the drainage outfalls in the North channel, future extension of main drains due to the proposed Outer Ring Road should be considered.

ii) The layout of the outfalls in the land reclamation area in the South Channel should be reviewed based on the latest information.

iii) S-10 and S-18 drains which have lowlying area should be planned taking into account the combination of pump facility and retention pond.

iv) Drainage problems of the main drains in the areas outside Georgetown were not included in the previous Drainage Master Plan. Hence, for the areas where inner water problem still remains after completion of river improvement works, it is necessary to propose some countermeasures.

while, in the areas outside Georgetown, drainage study was carried out for the commonly flooded areas with some flood damage and the future flood prone areas with anticipated inner water problem.

All existing outfalls of the main drains in the areas outside Georgetown discharge into the river or its tributary.

2. EXISTING DRAINAGE SYSTEM

2.1 Existing Conditions of the Main Drains in Georgetown Area.

The field reconnaissance survey for 45 main drains in Georgetown was carried out. Main items of the survey are as follows:

- Types of main drain and outfall
- Conditions of main drain and outfall
- Land use of catchment area
- Screen (or garbage traps)
- Garbage
- Siltation
- Tidal effects

The summary of characteristics of existing outfalls in the North and South Channels is shown in Tables I-1-1 and I-1-2.

The catchment area for each urban drainage system in Georgetown is ranging from 2 to 160 ha. 90% of total urban drainage systems are having the catchment area less than 50 ha. And 70 % of total urban drainage systems are less than 10 ha in area.

2.1.1 Types of Main Drains and Outfalls

Almost all the main drains in North and South Channels are channelized mostly with open drains with the exception of the earth drains, such as S-11, S-12, S-13, S-14, S-19 and S-24.

The type of outfall depends on its location and condition, and remarkably there are some outlets damaged or silted.

- North Channel - N-5 (The slabs on the outlet are fallen. They disturb the flow.)
  - N-9 (The slabs are partly damaged.)
  - N-10 (Totally silted,Unusable) (One of the two pipe culverts is totally silted. Outlet is heavily damaged.)
  - N-15 (Pipe culvert is heavily damaged.)
  - N-17 (Pipe culvert is heavily damaged.)
- South Channel - S-11 (No outlet exists)

2.1.2 Screen

Several screens by wire gauge to screen out the garbage are installed at the following outfalls:

- North Channel - N-1 (One screen is installed at the exit of outfall. But it is slightly damaged.)

  N-4 (One screen is installed at the exit of outfall and two screen are installed at the entrance of outlet.)

  N-8 (One screen is installed at the middle of main drain.)

  N-20 (One screen is installed at the entrance of outlet.)

- South Channel - S-10 (Two screens are installed in front of the pumping station.)

  S-16 (One screen is installed in the secondary drain.)

2.1.3 Garbage

The garbage is seen in most of the open drains and outlets both in the North and South Channels. Among them, the following outfalls are under relatively worse condition than others:

- S-10 (Excessive rubbish at pumping station)
- S-16 (Excessive rubbish along the main drain)
- S-22 (Excessive rubbish along the main drain)

2.1.4 Siltation

The outlets which situations are aggravated by blockage of drain by siltation are as follows:

- North Channel - N-10
  N-12
  N-13
  N-20
  N-21

- South Channel - S-22

2.1.5 Tidal Effects

It is observed that most of the existing outlets are subjected to high tides, as shown in Tables I-1-1 and I-1-2.

The general views were envisaged after the review of the previous urban drainage study and field survey of the existing urban drainage system in the city of Georgetown as follows:

- Rehabilitation of the existing drainage outlets should be urgently necessary to regain the original capacity.
- Regular maintenance and operation shall be needed for any blockage by rubbish and by siltation.

In Georgetown, the major cause of flood along the drains is mostly its undersized facility such as main drain and outlet.

Among those drainage basins in problem, the drain such as S-10 and S-18 needs to have a pumping facility in order to pump up the expected flood water from its internal drainage catchment during the storm, because the drainage catchment has lowlying area which is +1.40 m in elevation.

The only existing pumping station is situated at S-10 drainage outlet of which capacity is about 7.6 m^3/s.

2.2 Existing Flow Capacity of Main Drains in Georgetown Area.

The existing flow capacities of the main drains estimated by previous study are shown in Table I-2.

Based on the flow capacity, 45 main drains are categorized as follows;

Return period of flow capacity of the main drain	No. of the main drain	%
a. $Q_{max} > 1/10$	16	35
b. $1/10 \geq Q_{max} > 1/5$	3	7
c. $1/5 \geq Q_{max} > 1/2$	0	0
d. $1/2 \geq Q_{max}$	18+8	58
Total	45	100

These probable flood discharges were estimated for each revised catchment area as shown in Table I-5.

2.3 Drainage Problems in the Areas Outside Georgetown

Drainage problems in the areas outside Georgetown was recognized in the present and future flood prone areas.

In the 25 river basins outside Georgetown, the following areas are known to be commonly flooded. Fig.I-2 shows the location of these areas.

River Name	Location
13. Sg. Gelugor	- Brown Garden - Minden Heights
14. Sg. Dua Besar	- Pesta Parking Lot and Road - Jalan Pantai Jerjak - Jalan Sungai Nibong
15. 16. Sg. Nibong	- Jalan Mahsuri
17. Sg. Keluang	
Sg. Relau	- PDC Housing area
Sg. Ara	- Jalan Tun Dr. Awang
Ditch-X	- Jalan Bayan Lepas
18. Sg. Nipah	- Lowlying Area (River Mouth)
19. Sg. Kampung Masjid	- Lowlying Area (River Mouth)

21. Sg. Kampung Seronok - Permatang Damar Laut Area (Sg. Bayan Lepas)

Since the return period for the maximum floods experienced throughout the Island is greater than 10 years, as determined from rainfall records, these locations cover all areas which have inundation problem in the present drainage design scale (i.e. 10 year return period)

Almost all these areas are located along the main stream or tributaries and the causes of flooding are mainly due to the inadequate flow capacity of main stream.

Hence, these drainage problems should be basically solved by flood mitigation measures of main stream.

However, it should also be considered that the new drainage problem may be created by future development especially in lowlying area.

Lowlying area are generally defined as these areas which are affected by high tide, or have low ground levels compared with surrounding areas or high river water levels.

For the 25 river basins outside Georgetown, lowlying areas were determined based on the river survey maps, urban planning maps, etc.

Table I-8 shows the features of the existing lowlying areas.

As a basic concept for solving the drainage problems in these lowlying areas, the high water level of the rivers should be lowered as much as possible.

For lowlying areas affected by high tide near the river mouth, the basic strategy is to fill the area to a ground level high enough to permit future development, instead of installing pumping facilities.

3. REVIEW OF THE PREVIOUS STUDIES

The survey and study for the Penang Island Structure Plan were carried out by MPPP in 1985, covering all aspects including infrastructure matters of Penang Island and the implementation and implications of the plan. As one of the infrastructure studies, public utilities study was carried out, including such matters as water supply, electricity supply, telecommunication, sewerage, solid waste disposal and management and drainage. The results of the study were compiled as technical report No.14 of Penang Island Structure Plan.

Findings and main issues which were elucidated by this study concerning drainage can be summarized as follows:

3.1 Findings and Main Issues

3.1.1 Findings

The existing drainage system in Georgetown consists of a network of drains which discharge via 45 man-made outfalls to the North and South Channels or directly to rivers and streams. Outside Georgetown all drainage outlets are natural rivers. Fig.I-2 shows the existing outfalls and their respective catchment areas.

3.1.2 Main Issues

- There have been high occurrences of indiscriminate dumping of garbage into open drains, streams, vacant lands and the sea by the general populace.

- There are at present many government departments responsible for drainage in the island working without sufficient coordination and cooperation.

- The existing man-made as well as natural drainage outlets in many areas are inadequate to cater for the drainage. The situation is aggravated by blockage of the drains by garbage and siltation, by high tides and by the faster run-off caused by excessive land-cutting.

- Drainage improvement measures would require tremendous amount of funds.

- Some structural measures were planned and recommended by MPPP in order to alleviate the flooding caused by inadequate urban drainage system.

3.2 Proposed Plans by MPPP

Since almost all the drainage outlets are subjected to tidal inundation, the provision of a drainage system with tide controlled flap gate outlets is warranted. If necessary, pumping facilities should be installed at the outlets. The construction of pumping station and installation of pumps and a new tide gate were proposed at the existing Brick Kiln road outfall (S-18). Also the installation new pumps to replace the present ones were proposed at Prangin Canal outlet (S-10).

To improve the drainage system of the City of Georgetown and alleviation of flooding, a reorganization of the existing outfalls and catchment areas along the north and south channels is necessary.

Fig. I-3 shows the previously proposed outfalls.

Adequate drainage access reserve must be provided to all drainage outlets to regularly maintain the outlets.

3.3 Review of the Previous Studies

The Drainage Master Plan prepared by MPPP was reviewed taking into account the present land use and future land use conditions (especially, Land Reclamation Plan by PDC in the South Channel and Outer Ring Road Plan in the North Channel), and the following items will be necessary to be changed.

a) N-20 (Sg. Babi), N-21 (Sg. Bagan Jermal) and S-23 (Jelutong Diversion Channel) should be treated as a river and planned for the 50 year design discharge.

b) The outfalls along the Gurney Drive in the North Channel will be reorganized after taking into consideration landscape, environment and maintenance.

c) S-11, S-12 and S-13 drains will be connected to S-10 drain because of their low ground levels. S-14 drain will be extended directly into the sea.

d) S-18 drain will be extended up to proposed Coastal Road without discharging into Sg. Pinang, and discharge via retention pond or pump facility into the sea.

e) For the drainage in S-10 and S-18 catchment, the combination of pumping and retention pond will be adopted.

f) The extension of existing drains of S-19 - S-22 and S-24 will be quite difficult due to their low ground level in the catchments. Hence, the open waterway will be planned near the existing outfalls.

g) S-23 drains (Sg. Jelutong Diversion Channel) will be planned for 50 year design discharge and extended to the new coastal line in box culvert or concrete open channel.

h) In the areas outside Georgetown, previous Master Plan for drainage by MPPP covers only main stream of the river. Hence, in this study, drainage problems in commonly flooding areas and lowlying areas will be studied.

4. PLANNING CONCEPT AND CONDITIONS FOR DRAINAGE MASTER PLAN

4.1 Planning Concept

The following concept was contemplated for planning the drainage master plan.

- Considering the fact that the flow capacity of the river in Georgetown is relatively small, the distance from main drain to the sea is short and the availability of relatively steep slope of the main drain, it is presumed to be reasonable that the outlets be located towards the sea as much as possible.

- The flap gate or the pumping station will be considered for the drainage basins with a lowlying area or high tide level, where the drainage by gravity is physically impossible.

- It is presumed that the drainage problem along the Sg. Dondang at upstream of Sg. Pinang will be solved by lowlying the design water level of the main stream of Sg. Dondang.

- Number of the outlet of main drains along the Gurney Drive in the North Channel will be reduced by means of reorganization after taking into consideration the landscaping and environmental aspects, and their maintenance.

- In the South Channel area of Georgetown, the extension of the existing drain in land reclamation area is seemed to be extremely difficult.

  Hence, the existing outfalls will be connected to new waterway.

- For drainage in S-10 and S-18 catchments combination of pumping facility and retention pond will be contemplated.

- For the undeveloped lowlying areas near the river mouth, the basic strategy will be to fill up the area with a ground level suitable for future development instead of installing any pumping facility.

- The drain reserve shown in Table I-3 should be kept taking into account access requirement for maintenance works.

4.2 Run-off Analysis

The run-off formula adopted for the study is explained below;

$$Q = 1/360\ Cs.C\ I\ A$$

where,
- Q = the peak discharge (m3/s)
- I = the average intensity of rainfall (mm)
- A = the catchment area (ha)
- C = a run-off coefficient
- Cs = a storage coefficient
  - $Cs = 2tc/(2tc + td)$
  - where,
  - tc = the time of concentration
  - td = the time of flow in the main drain

The application of a run-off formula modified by a storage coefficient is preferable in the project area which is totally flat and lowlying.

The relationship between Cs, tc and td in Malaysian Standard is designed on the basis of the theory acceptable internationally,and the result of its practical application on some drainage basins in Kuala Lumpur (KL) coincide with those obtained by the more elaborate routing procedure by way of computer calculation. The derivation of Cs as a function of tc and td is explained in "Flood Procedure No. 16", published by Ministry of Agriculture and Rural Development, Malaysia. With the background above, "Rational Method" with storage coefficient Cs, i.e., Q=1/360 Cs C I A is adopted for the Project.

A run-off coefficient C adopted in this drainage plan is shown in Table I-4. The rainfall intensity curve is shown in Fig.I-4.

The Tables I-5 and I-6 show the result of run-off calculation for probable floods.

4.3 Flood Protection Level

The design flood protection level of 10-year return period, which is the same value adopted for the Master Structure Plan MPPP, was adopted based on the following reasons:

i. The catchment area of the drainage basin is comparative small and duration of peak discharge is quite short.

ii. Generally, as a design scale of drainage plan, 5 to 10 year return period flood is adopted.

iii. Construction of drainage facilities in the built-up urban area is quite costly and adoption of large scale of protection level is not feasible.

5. FORMULATION OF MASTER PLAN

5.1 Drainage Plan in Georgetown Area

5.1.1 Drainage Plan in the Coastal Zone

In the coastal zone of Georgetown, there are proposed projects of Outer Ring Roads, Coastal Roads and land reclamation. By these projects many of the existing drainage outfalls will be affected.

In the coastal zone of the North Channel, the only proposed project is the Outer Ring Road.

The required extension length of the existing drain is expected to be about 40 - 100 m.

In the South Channel, a large scale land reclamation project is now under consideration. The maximum length of drains to be extended would be 685 m. However, it may be rather difficult to construct such a long extension drain without any technical or hydraulic problem.

Hence, the following two cases were contemplated.

CASE - 1 Filling without any waterway in reclaimed area.
CASE - 2 Filling leaving some portion to keep the waterway in reclaimed area.

1). Comparison of Drainage Plan in Reclaimed Area in Case of With and Without Waterway.

(1) CASE-1 Without waterway

In this case, all the drains located in the land reclamation area would have to be extended from 450 m to 685 m, with or without realignment. The type of drain would be closed channel which would be installed under a roadway in the future reclaimed area.

The maintenance of such a drain extension would be very difficult because it would have a very flat slope and be affected by high tide resulting in clogging by sedimentation.

Construction cost would be costly because of poor foundation conditions.

The major problem would be inundation in inland areas during floods exceeding the 10 year design flood.

(2) CASE-2 With waterway

In this case, by installing the waterway in the reclaimed area, the hydraulic effect for the existing outfalls due to land reclamation would probably be negligible.

The waterway will be composed of a waterway along the existing coastal line and one connecting to the sea.

These waterways serve as a retention pond for storing flood waters even if the sea water level rises due to the effects of global warming in future.

To use such a waterway as a retention pond, a tidal gate is necessary at the new outfall.

Installation of a pumping station may reduce the necessary retention pond capacity.

CASE 2 has considerable merit as compared to CASE 1 and is recommended as the drainage plan for reclaimed areas.

Regarding internal drains in the reclaimed area, those outfalls should discharge directly to the sea without flowing into the waterway.

2). Basic concept of proposed waterway

The proposed waterway was planned based on the following conditions;

- The cross section of waterway should have sufficient capacity so that the backwater effect would be negligible.

- The waterway should have enough capacity to storage the 50 year flood discharge for 6 hours.

Fig.I-5 shows the relationship between the necessary volume of channel, (length and width), catchment area and accumulated rainfall.

The proposed waterways in the South Channel are shown in Fig. I-6. These two waterway systems have a minimum necessary volume and it is desirable to keep the larger volume.

The continuous waterway connecting W_1 waterway with W_2 waterway was considered for the purpose of navigation use and improvement of water stagnation in the waterway.

Tentatively, it is possible to make the continuous waterway.

However, there exist the outfall of the Jelutong Diversion Channel and sewage outfall between proposed W_1 and W_2 waterways.

Especially, the outfall of diversion channel will be major obstruction in the future.

Because the level of the existing outfall of the diversion channel is high enough to discharge directly to the sea by gravity even with long extension of the channel in the reclaimed area and this diversion channel does not need any retention pond nor pump facility for drainage even if the sea water level rises in the future.

Since the catchment area of this diversion channel is rather big (490 ha), about 900,000 m^3 of additional storage capacity will be required in the waterway (i.e. 30 m in width and 16 km in length), when the run-off discharge from the diversion channel catchment flows into the continuous waterway.

Hence, for the outfall of the Jelutong Diversion Channel, it is recommended to construct separately the water channel from the proposed W_1 and W_2 waterways.

5.1.2 Layout of Outfalls of Main Drains in Georgetown

The outfalls of main drains in Georgetown area are divided into several groups according to the future land use plan of coastal zone.

(1) North Channel

In the North Channel, there is an Outer Ring Road project for which a study was carried out by JICA in 1981.

This proposed road runs along the existing coastal line and was planned to have the same formation level as the existing road except at the river mouth of Sg. Babi where the road crosses the river by means of a bridge.

(i) Outfalls N-1 to N-11

For the outfalls N-1 and N-2 there has been no consideration for future extension.

The outfalls of N-3 ~ N-11 will be extended straight up to new coastal line outside the proposed Outer Ring Road.

The required extension length will be as follows.

Original No.	Original Catchment Area (ha)	Increase / decrease of Area by Re-organization (ha)	Additional Catchment Area by Land Reclamation	Proposed Catchment Area (ha)	Extension Length (m)	Proposed outfall No.
N-3	1.5	0.0	1.2	2.7	45	PN3
N-4	1.4	0.0	1.6	3.0	70	PN4
N-5	20.5	0.0	5.0	25.5	150	PN5
N-6	34.1	0.0	5.3	39.4	90	PN6
N-7	4.8	0.0	1.8	6.6	90	PN7
N-8	10.4	+5.0	1.9	17.3	95	PN8
N-9	54.2	-11.7	1.2	43.7	75	PN9
N-10	3.3	-5.0	1.7	0.0	80	-
N-11	12.0	+11.7	2.1	25.8	60	PN10

(ii) Outfalls of N-12 to N-19

For the section of Gurney Drive, realignment reducing the number of outfalls is expected after taking into consideration the maintenance and environmental aspects.

However, this Ring Road project is not definitive nor has it been authorized. The distance between the new road and existing one is still flexible.

Hence, at this stage, only the layout of the realignment will be considered.

The realignment of outfalls of the Gurney Drive section in the North Channel is as follows;

N-19, N-18, N-17 — PN-14
N-16 — PN-13
N-15, N-14 — PN12
N-13, N-12 — PN11

(2) South Channel

In the South Channel, about 210 ha of coastal area along Prangin Street Ghaut and Penang Bridge is proposed for reclamation, and a coastal road is proposed along the same stretch. This coastal road will run almost parallel to the existing coastal line about 500 m offshore.

Also, about 25 ha of coastal area along Market Street Ghaut and Prangin Street Ghaut is proposed for reclamation.

Nineteen outfalls, from S-6 to S-24, will be affected by this land reclamation project.

(i) S-1 - S-9 Outfalls

S-1 to S-5 outfalls are not affected by land reclamation and will remain at the existing sites.

S-6, S-7, S-8 and S-9 will be extended due to land reclamation as follows.

Original No.	Original Catchment Area (ha)	Additional Catchment Area by Land Reclamation	Proposed Catchment Area (ha)	Extension Length (m)	Proposed Outfall No.
S-6	5.1	3.9	9.0	160	PS6
S-7	7.3	4.9	12.2	220	PS7
S-8	4.7	4.5	9.2	280	PS8
S-9	7.6	9.0	16.6	130	PS9

(ii) S-10 - S-18 Outfalls

S-10 will be extended up to new retention pond which is planned outside proposed coastal road.

S-11, S-12 and S-13 will be connected to S-10 outfall because their existing ground level is not high enough to extend directly those drains up to the future outfalls outside the proposed coastal road.

S-14 drain will be solely extended to the new coastal line.

S-15 and S-16 drains will be reorganized and discharged directly to the sea.

The outfall of S-17 will remain at the existing site.

S-18 drain, which flows into the Sg.Pinang at present, will be extended and connected to the retention pond located outside coastal road.

The proposed catchment areas and extension length of each outfall are as follows:

Original No.	Original Catchment Area (ha)	Additional Catchment Area by Land Reclamation	Proposed Catchment Area (ha)	Extension Length (m)	Proposed Outfall No.
S-10	120.0	0.0	120.0	0.0	PS10
S-11	0.0	0.0	0.0	0.0	-
S-12	1.6	0.0	1.6	240	-
S-13	4.7	0.0	4.7	143	-
S-14	5.3	2.7	8.0	300	PS11
S-15	6.6	0.0	6.6	32.0	-
S-16	12.0	6.7	18.7	150.0	PS12
S-17	6.8	0.0	6.8	0.0	PS13
S-18	104.0	0.0	104.0	0.0	PS14

(iii) S-19 - S-22 outfalls

These outfalls will be treated as the third group to be surrounded by Sg. Pinang, the coastal road and the extended Jelutong diversion channel.

The total catchment area of these drains is 44.7 ha. The minimum required capacity of waterway will be about 80,500

m^3, with a length of 1,150 m, width of 35 m and effective depth of 2 m. However, taking into account the future demand for retention volume, 1,450 m of waterway having a capacity of 101,500 m^3 as shown in Fig. I-6 is recommended.

(iv) S-23 outfalls (Jelutong Diversion Channel)

The existing diversion channel has enough capacity to handle the discharge of a flood with a 50 year return period. And it has adequate design water level to discharge by gravity flow to the future extended outfall.

Hence, this outfall should be extended straight to the outside of the Coastal Road.

(v) S-24 outfall

This outfall has a catchment area of 42.2 ha; the necessary volume of waterway will be 75,960 m^3, with a length of 1,100 m, width of 35 m and effective depth of 2 m. However, for the same reason of S-19 - S-22 outfalls, 1,550 m waterway having a capacity of 108,500 m^3 is proposed. The alignment of the waterway is shown in Fig. I-6.

3) Future extension plan of Jelutong Diversion Channel

(1) Verification of existing flow capacity of Jelutong Diversion Channel

This diversion channel is one of tributary of Sg. Pinang and should be planned for 50 year flood as a river.

The Jelutong Diversion Channel was completed in 1976 for the purpose of diverting the discharge from the 74% of the catchment area (4.9km^2) of the whole basin of Sg. Jelutong to South Channel.

This diversion channel consists of a rectangular reinforced concrete channel with a width ranging from 8.22 to 9.52 m, an average depth of 3.05 m and a gradient of 1 in 833.33.

Fig.I-7 shows a typical cross section of the channel.

An estimation of run-off discharge using formula shown in Hydrological Procedure No. 16 was carried out under the projected land use conditions in 2010 to evaluate the existing flow capacity.

Existing Flow Capacity,

Using Manning's Formula:

$Q = 1/n\ R^{2/3}.I^{1/2}.A$ $N = 0.014$:concrete culvert

$I^{1/2} = (1/833.33)^{1/2} = 0.0346$

A (m^2)

a) Case I — without any freeboard (H.W.L. is the bottom of beam)

$A = 22.16\ m^2$, $R = 1.682$, $R^{2/3} = 1.414$

$Q = 77.4\ m^3/s$

b) Case II — with 15% of the sectional area as a necessary clearance under the bottom of top slab
(H.W.L. is 0.489m below the bottom of top slab
i.e. 0.083m below the bottom of beam)

$A = 21.52\ m^2,\ R = 1.654,\ R^{2/3} = 1.399$
$Q = 74.3\ m^3/s$

c) Case III — with 15% of the sectional area as a necessary clearance below the bottom of beam (H.W.L. is 0.428m under the bottom of beam)
$A = 18.84\ m^2,\ R = 1.53,\ R^{2/3} = 1.328$
$Q = 61.8\ m^3/s$

d) Case IV — with 0.6m freeboard under the bottom of beam

$A = 17.50m^2,\ R = 1.462,\ R^{2/3} = 1.288$
$Q = 55.7\ m^3/s$

<u>50 year discharge estimated by specific discharge of Storage Function Model</u>

The design discharge was estimated by Storage function Model as one of Tributary of Sg. Pinang System.

$Q = 10 \times 4.9 = 50\ m^3/s$

For refevence, the run-off discharges for 50 year and 100 year floods were estimated by Retional Method.

<u>50 year discharge by Rational Method</u>

a) Areal average run-off coefficient

f = 0.308 = 0.31 (see Table I-6.)

b) Run-off discharge

$Q = 1/3.6.f.I_{1/50}.A$
Time of concentration: to + td
= 15 + 32 (min.)
$I_{1/50} = 125$ mm/hr
$Q = 1/3.6 \times 0.31 \times 125 \times 4.90 = 52.7\ m^3/s$

<u>100 year discharge by Rational Method</u>

$I_{1/100} = 136$ mm/hr
$Q = 1/3.6 \times 0.31 \times 136 \times 4.9 = 57.4\ m^3/s$

<u>Conclusion</u>

The diversion channel section with a 0.6 m freeboard under the bottom of beam can handle the 50 year discharge.

Q_{50} 52.7 m^3/s < 55.7 m^3/s CASE IV

The diversion channel section with 15% of the sectional area as the basis for the necessary clearance under the bottom of the top slab can handle 130% of the 50 year discharge.

1.3.Q_{50} 68.5 m^3/s < 74.3 m^3/s CASE II

The diversion channel section without any freeboard under the bottom of beam can handle the 100 year discharge.

Q_{100} 57.4 m^3/s < 77.4 m^3/s CASE I

Finally, the Jelutong Diversion Channel flow capacity is adequate to handle the run-off discharge for 50 year flood of Sg. Jelutong under the projected land use conditions in the year 2010.

(2) Future extension plan for Jelutong Diversion Channel

Due to land reclamation the extension of existing Jelutong Diversion Channel will be necessary. The length of the extension is expected to be about 510 m.

The alignment shown in Fig.I-8 is proposed. At present there are no definitive land use plans or constraints related to the reclaimed area. The gradient and cross section of the proposed channel will be the same as for the existing one.

The design discharge was set to be 65 m^3/s (1.3 x 50 m^3/s).

The invert level and design high water level at the new outfall will be -0.692 and 1.99 m, respectively: (see Fig.I-9)

The design H.W.L. would not be affected even by the E.H.W.S. (+1.60 m).

The existing 1.2 m diameter water supply pipe line about 200 m off shore should be reconstructed because it is a major obstruction crossing the route of the proposed diversion channel.

5.1.3 Proposed Drainage Plan in Georgetown Area

Based on the above mentioned planning concept and conditions, proposed drainage facilities were studied. The major components of the drainage master plan in Georgetown consist of realignment of the existing main drains including extension of the outfall in the future reclamation area, reorganization of drainage catchment of existing drains and pump drainage plan for lowlying area.

Fig I-10 shows the proposed outfall and catchment areas in Georgetown.

The required improvement works for each drainage system are summarized in Table I-7-1 and I-7-2.

The proposed plan of drainage route, longitudinal profile and typical cross sections are shown in the drawing "DRAINAGE MASTER PLAN IN GEORGETOWN" compiled separately.

For drainage catchment S-10 and S-18, simulation study on the relationship between given run-off hydrograph, required pump capacity and storage capacity of retention pond were carried out for determination of optimum combination system of pump and retention pond.

Proposed sites of retention ponds and pump stations are shown in Fig.I-10.

5.1.4 Construction Cost of Georgetown Drainage Master Plan

The financial cost for the Georgetown Drainage Projects are summarized in Table I-11 to I-13. Total cost was estimated to be about M$ 64.4 million.

5.2 Drainage Plan in the Areas Outside Georgetown

5.2.1 General

1) Objective Areas

Drainage study in the areas outside Georgetown was carried out for those areas that are flood prone at present or in future.

In the 25 river basins outside Georgetown, the following areas are known to be commonly flooded or flood prone and lowlying. Fig.I-2 shows the location of these areas.

No.	River Name		Location
13.	Sg. Gelugor	-	Brown Garden
		-	Minden Heights
14	Sg. Dua Besar	-	Pesta Parking Lot and Road
		-	Jalan Pantai Jerjak
15.	16. Sg. Nibong	-	Jalan Mahsuri
17.	Sg. Keluang		
	Sg. Relau	-	Jalan Tun Dr.Awang
	Ditch-x	-	Jalan Bayan Lepas
18.	Sg. Nipah	-	Lowlying Area (river Mouth)
19.	Sg. Kampung Masjid	-	Lowlying Ares (river mouth)
21.	Sg. Kampung Seronok	-	Permatang Damar Laut Area (Sg. Bayan Lepas)

2) Causes of Flooding

All objective drains in the area outside Georgetown flow into the river or its tributary. In almost all cases drainage problems are related to the water level or flow capacity of a main stream. The major causes of the existing flood problem in the objective areas are considered as follows:

i) Overflowing from the river due to insufficient flow capacity.

ii) Impossibility of drainage by gravity due to higher water level of the river in comparison to the ground elevation of the objective area.

iii) Inadequate flow capacity of drains.

iv) Combination of the above mentioned (i~iii) causes.

The causes of inundation and countermeasures in each objective area are summarized in Table I-9.

3) Drainage Plan

As the basic concept for solving the drainage problems in the objective areas, the high water level of the river should be lowered as much as possible.

For lowlying areas affected by high tide near the river mouth, the basic strategy is to fill the area to a ground level high enough to permit future development, instead of installing pumping facilities.

The features of proposed drainage facilities are shown in Table I-10.

5.2.2 Sg. Gelugor

1) Existing Conditions of Sg.Gelugor

Downstream of Sg. Gelugor has two streams. One is original river, another is Sg. Gelugor Diversion Channel. In accordance with this condition, the catchment of Sg. Gelugor is divided into Sg. Gelugor original river area (GO) and Sg. Gelugor Diversion Area (GD).

Almost all the catchment area of Sg. Gelugor were already developed for housing. Two flood prone areas, Minden Heights and Brown Garden, are identified along Sg. Gelugor as shown in Fig.I-10.

Run-off from Minden Heights and some part of U.S.M. are discharged to the original river. Run-off from the area along Sg. Gelugor main stream is directed to the diversion channel.

2) Causes of Inundation

The causes of inundation of each area are as follows:

i) Brown Garden area

Overflowing of the river water due to inadequate discharge capacity at Jln. Permai (CH1200) and existence of low ground level (EL. 3.0 m) at Brown Garden.

ii) Minden Heights area

Overflowing of the drain due to inadequate flow capacity of the drain and original Sg. Gelugor.

3) Drainage Plan

(1) Brown Garden area

Required flow capacity of the main stream is 29 m^3/s and improvement of the existing concrete channel at upstream of Jln. Gelugor will lower the Design High Water Level (D.H.W.L.) at Jln. Permai (CH1200) to 3.3 m. This would mitigate the inundation due to river overflow, considering the ground level along the stream.

However the inner drainage problem in the lowlying area (ground level +3.0 m) still remains.

Hence, reorganization of catchment area and replacement of the existing outfall will be necessary. The outfall of the drain is recommended to be shifted to the lower portion as shown in Fig.I-12. The design high water level (50 year flood) of the Sg. Gelugor at Jln. Gelugor is +2.93 m.

The proposed drain route is shown in Fig. I-12 and water level for 10 year flood is + 2.65 m at the same point. This proposed drain route might encounter land acquisition problems due to the presence of terrace houses, in which case an alternative like a pumping scheme should be investigated.

(2) Minden Heights area

To mitigate the inundation of this area, it is recommended that not only improvement of Sg. Gelugor Original Stretch but also improvement of drains.

i) Improvement of Original Sg. Gelugor

The existing flow capacity of the original Sg. Gelugor, except some sections, is smaller than the design discharge of 25 m^3/s calculated for 1/50 year floods. The proposed cross section and longitudinal profile are shown in Fig.I-13.

ii) Improvement of drains

Drains which cause inundation due to inadequate flow capacity shall be improved for 10 year floods. The proposed cross section and longitudinal profile for the design discharge (17 m^3/s) are shown in Fig.I-14.

Due to the inadequate capacity of the internal drain downstream of the U.S.M. area, and also constraints in upgrading these drains, the retention pond at U.S.M. compound should be investigated at detailed design stage. The proposal for Sg. Gelugor section therefore should be reviewed due to possible attenuation effect.

This retention pond should be made by excavating the existing ground to protect the houses nearby. The downstream stretch of this pond also be improved to lowering the design bed level to obtain the effective enough depth of the pond.

5.2.3. Sg. Dua Besar

1) Existing Conditions of Sg.Dua Besar

The catchment area of the Sg. Dua Besar is 6.72 km^2, including about 50% of the upper catchment of the Sg. Dua Kecil which is diverted to the Sg. Dua Besar. (Fig. I-15)

Since the land is very flat in this area, particularly in PESTA, inundation spread widely.

2) Design Discharge of Sg. Dua Besar

The design discharges of Sg. Dua Besar, which was calculated for a 1/50 year floods, are 55 m3/s at the river mouth including the discharge from the Sg. Dua Kecil Diversion and around 30 m3/s at Jln. Sg. Dua.

3) Causes of Inundation

The causes of inundation along Sg. Dua Besar are;

i) rising river water level and overflow due to inadequate discharge capacity along the entire stretch, and

ii) failure of drainage from lowlying area due to rising of river water level.

4) Drainage Plan

After completion of widening and deepening the main stream by Urgent Project for flood mitigation, the D.H.W.L. of the main stream is expected to be reduced to 1.7 m at Jln. Sg. Nibong, 1.9 m at the diversion confluence and 3.2 m at Jln. Sg. Dua.

(1) Downstream of Jln. Sg. Nibong

After completion of the river improvement works of the Sg. Dua Besar, the areas along the main stream will be protected from the flooding due to overflowing from the main stream.

However, the inner water problem during high tides (1.6 m) still remains in the lowlying area shown in Fig.

I-16. As a countermeasures for these areas reorganization of the catchment, realignment of existing drain and installation of the small scale pump facilities are recommended. The proposed drainage system is shown in Fig. I-16.

(2) Area around PESTA

The existing ground level of PESTA is around 2.2 m, which is 0.3 m higher than D.H.W.L. (1.9 m). The cause of the existing inundation is considered to be mainly due to the reduced cross section at the bridge crossing Jln. Sg. Nibong. This inundation problem is expected to be solved after completion of the river improvements which will include reconstruction of the bridge.

(3) To Jln. Sg. Dua

D.H.W.L. at the point of Jln. Sg. Dua is 3.2 m, which is about 0.5 m lower than the existing bank elevation (about 3.7 m). Most areas at the downstream of Jln. Sg. Dua, where the stream is partially embanked, are lowlying with existing ground levels between 2.5 m and 2.7 m.

Since the major land use in this lowlying area is cultivation, the inundation problem of this area will be solved by rising the ground level above the design bank level which is given in the Flood Mitigation Master Plan for the rivers.

(4) Diversion Areas

It is presumed that existing flooding is caused by the backwater due to the bottle neck resulted by the bridge crossing Jln. Sg. Nibong and the inadequate discharge capacity of the diversion channel (5 - 14 m3/s), due to the too gentle slope of the channel. (1/600 and 1/1000.)

Inundation due to river water overflow would be prevented by expanding the cross section of the diversion channel to a design discharge of 9 m3/s and removing the bottle neck by bridge reconstruction. The proposed cross section and longitudinal profile are shown in Fig. I-13.

There exists small lowlying area in the left bank near upper diversion channel. While it is difficult to fill the land because a clinic is located there, the inundation problem in this area can be solved by installing a small scale pump.

5.2.4. Sg. Nibong Kecil

1) Existing Condition of Sg. Nibong Kecil Catchment

Downstream of Sg. Nibong Kecil (from river mouth to Jln. Sungai Nibong) has already been improved by D.I.D. and construction of road side drain along Jln. Sg. Nibong was also completed.

However, inundation or drainage problem is reported in the lowlying area along Jln. Relau and in the area along Jln. Mahsuri which is Sg. Relau catchment area.

Existing conditions of inundation are shown in Fig.I-18.

(1) Along the area of Jln. Relau

While run-off from Bukit Jambul is to flow along the Jln. Relau, the run-off presently flows into the lowlying area opposite side of Jln. Relau because the drain along the Jln. Relau is only small earth channel. The peak run-off discharge from Bukit Jambul is relatively high due to short time of concentration because of steep slope in the catchment. Accordingly the run-off exceeds the flow capacity of the existing small stream in the lowlying area and causes flooding there.

(2) Along the area of Jln. Mahsuri

The run-off along Jln. Bukit Jambul is to flow down along the Jln. Relau, but it presently flows into the area opposite side of the road, finally reaching to the drain along the Jln. Mahsuri, since there is no trunk drain running along the Jln. Relau to Sg. Nibong Kecil outlet. The flow capacity of the drain along the Jln. Mahsuri is 8 m^3/s and is not enough to cater the extra discharge from Jln. Bukit Jambul.

Since the area is flat, the inundation spreads widely through the area.

2) Design Discharge of the Drain along Jln. Relau

The channel along Jln. Relau is treated as a drain because; i) there is no natural river in the catchment along Jln. Relau, and ii) its catchment (55 ha) is smaller than 2 km2.

Design discharges at the target points (see Fig.I-18) for 10 year flood are;

Point A 2 m^3/s,

Point B 4 m^3/s.

3) Causes of Inundation

(1) Along the area of Jln. Relau

The extra run-off from Bukit Jambul is considered to be the major cause of the inundation.

However, even if it is possible to cut off the run-off from Bukit Jambul by the drain along Jln. Relau, the flow capacity of the stream in the area is suspected to be not enough to cater the run-off from own catchment.

(2) **Along the area of Jln. Mahsuri**

Main cause of inundation around Jln. Mahsuri is due to the run-off from Bukit Jambul area and part of Sg. Relau catchment which infact must be drained to Sg. Nibong Kecil and Sg. Relau outlets.

4) **Drainage Plan**

(1) **Along the area of Jln. Relau**

To cut off the run-off from Bukit Jambul area to the opposite side area, it should be constructed the proposed road side drain along the Jln. Relau. The existing flood area is expected to be solved after completion of this drain. The proposed drain alignment is shown in Fig. I-19. The proposed cross section, longitudinal profile are shown in Fig.I-20.

However it is recommended to consider the land filling and improvement of the existing small stream when the area is developed in future.

(2) **Along the area of Jln. Mahsuri**

Inundation problems of this area are expected to be solved by the construction of proposed drain along Jln. Relau, because inundation problem of this area is caused by run-off from Jln. Bukit Jambul area which is to be discharged through the proposed drain.

(3) **The catchment in the Westside of the Pesiaran Bukit Jambul Road**

The run-off from the catchment in the westside of the Pesiaran Bukit Jambul road shown in Fig. I-21 is reported to flow down to the area of the Jln. Mahsuri. This catchment (A = 51 ha) initially belongs to the Sg. Relau catchment.

However, due to luck of proper drainage system to the Sg. Relau, run-off from this catchment causes flooding in the area along the Jln. Mahsuri.

The run-off discharge for 10 year flood from this catchment is 12 m^3/s.

To drain the run-off from this catchment, the construction of new drain connecting point A to the Sg. Relau as shown in Fig. I-21 is proposed.

5.2.5 Sg. Keluang (Sg. Relau, Sg. Ara)

1) **Existing Condition**

Sg. Keluang and downstream stretch of Sg. Ara has been improved by ongoing SDID's project.

Relau Diversion Channel (from CH.2410 of Sg. Relau to CH.2200 of Sg. Ara) is being proposed by DID and the Master Plan has incorporated this idea.

Areas along the downstream of these rivers are new industrial and housing area.

Inundation is reported for the following three areas as shown in Fig.I-22.

(1) Sg. Ara (Jln. Tun Dr. Awang)

(2) Sg. Relau (PDC housing area)
(Jln. Tengah)

(3) Jln. Bayan Lepas

2) Design Discharges of Main Stream

Design discharges of each point (see Fig.I-17) are as follows:

Point A (Sg. Ara)	:	40 m^3/s
Point B (Sg. Relau)	:	65 m^3/s
Point C (Sg. Relau)	:	12 m^3/s

3) Causes of Inundation

(1) Sg. Ara (Jln. Tun Dr. Awang)

Inundated area of Sg. Ara is limited alongside of the main stream. Causes of inundation in this area are considered to be rising of the water level due to inadequate discharge capacity of main river.

(2) Sg. Relau (PDC housing area, Jln. Tengah)

Existing flow capacity of Sg. Relau of this area is about 10 - 30 m^3/s. It is 55 - 35 m^3/s less than design discharge (65 m^3/s) without Relau Diversion Channel.

It can be judged that inundation is mainly caused by overflow from main stream, because drainage system of this area mostly discharges to the other river system through the road side drain of Jln. Sg. Nibong.

(3) Jln. Bayan Lepas

The existence of inundation was reported along the Jln. Bayan Lepas. But the actual area was not identified in the study.

4) Drainage Plan

Inundation or drainage problem of these areas will be solved by flood mitigation measures (river improvement, construction of diversion channel etc.) for the river.

(1) Sg. Ara

Improvement of Sg. Ara by deepening and widening will lower the water level and resolve the inundation problem in the areas along the river.

(2) Sg. Relau

Design discharge of the Sg. Relau downstream stretch is 12 m^3/s after completion of Relau Diversion Channel. Inundation problem will be solved by construction of Relau Diversion Channel and improvement of Sg. Relau downstream stretch.

Though some development plans are proposed in this area, it is recommended that existing river channel should be remained, because it will be necessary for the drainage in this area.

5.2.6 Sg. Nipah and Sg. Kampung Masjid

1) Existing Conditions of Sg. Nipah and Sg. Kg. Masjid

Both river basins, Sg. Nipah and Sg. Kg. Masjid, are located next to each other and areas comprise lowlying and filling up areas. Therefore the catchment can be easily changed by development works, such as housing development, road construction etc. For example several tributaries of Sg. Nipah was changed to Sg. Kg. Masjid by housing development.

There exist lowlying areas around both river mouths and the upstream. But presently, inundation was reported only upperstream area.

Upper catchments of the rivers have been rapidly developed and river channels are sometimes found to be filled without construction of any alternative drainage system. The inundation problems are observed in such ares.

Both downstream of Sg. Nipah and Sg. Kg. Masjid are affected by tide, but there is no flap gate, or any prevention structure for back flow, installed at the outlet of drains.

Existing conditions of these rivers are shown in Fig.I-23.

2) Causes of Inundation

Inundation of upper catchment area is mainly considered to be caused by filling the land without any proper alternative drains or due to inadequate flow capacity of the rivers.

3) Drainage Plan

The division of the catchment of Sg. Nipah and Sg. Kg. Masjid is shown in Fig. I-24.

The river improvement works for these two rivers are proposed in the Flood Mitigation Master Plan, and flooding Problems in these basins would be solved after completion of the river improvement works.

There exists the small drainage catchment (A≈15 ha) in the Sg. Kg. Masjid basin.

For the drainage of this area, the road side drains along the Lebuhraya Batu Maung (B - C) and the Jln. Batu Maung (B - A) have been constructed.

In order to solve the flooding problem in the upstream basin of this catchment, the existing flow capacities of these drains were examined.

The cross section of these drains are shown in Fig. I-25

- The flow capacity

Main drain type A	2.2 m^3/s x 2 = 4.4 m^3/s
Main drain type B	4.2 m^3/s x 1 = 4.2 m^3/s

While, the run-off discharge of this catchment was estimated to be 3.54 m^3/s for 10 year flood.

Accordingly, This drainage system has enough capacity to cater 10 year flood, and does not need to be improved.

However, in the upper reaches of this basin, it is necessary to construct the secondary drains which shall be properly connected to the existing these main drains or rivers.

5.2.7 Sg. Kampong Seronok (Permatang Damar Laut)

1) Existing Conditions of Permatang Damar Laut Area

The Sg. Kampong Seronok flows into the tributary of Sg. Bayan Lepas passing through Permatang Damar Laut area. Under the ordinal flow condition, most of the river flow is diverted to irrigate Permatang Damar Laut paddy area and remaining water flows through small concrete channel of 200 m in length along to Jln. KEPERMATANG DAMAR LAUT and enter natural stream of secondary tributary of Sg. Bayan Lepas as shown in Fig. I-26.

In recent few years, floods occurred 3 to 5 times a year, duration of floods were 1 to 2 hours, and depth of inundation was about 1 or 2 ft.

2) Design Discharge of Sg. Kg. Seronok at Upstream of Permatang Damar Laut Area

Since Sg. Kg. Seronok is the one of tributaries of Sg. Bayan Lepas, this river was planned for 50-year flood.

Design discharge at target point is 21 m^3/s.

3) Causes of Inundation

Major causes of the inundation in Permatang Damar Laut area are due to inadequate flow capacity of the road side concrete channel (2 m^3/s) and improper operation of water gate for paddy field water intake.

4) Drainage Plan

The proposed countermeasures in this flooding area are river improvements for the 600 m stretches between the irrigation intake point A and confluence with tributary of the Sg. Bayan Lepas (point B) shown in Fig. I-27.

The proposed alignment for the stretch between point A and point B was set on the east side of the road considering the existing land use condition.

The alignment of the downstream stretch from B to C basically follows the existing natural river course.

The features of the proposed river improvement works are as follows:

Concrete open channel: Rectangular
width 5.0 m, depth 2.1 m, L = 200 m

Rubble pitching channel: Trapezoidal
width 2.5 m, depth 2.1 m, L = 380 m

The proposed cross sections and longitudinal profile are shown in Fig. I-28.

Tables

TABLE I-1-1 CHARACTERISTICS OF EXISTING OUTFALLS IN NORTH CHANNEL

Existing Outfall	Catchment Area** (acres)	(ha)	Type * of Main Drain	Type * or Outfall	Land Use of Catchment Area	Remarks - (Conditions of outfall) - Screen	Garbage	Siltation	Tidal Effect
N 1	50.0	20.2	CD	CB	Park commercial	One (exit of outlet)	NONE	NONE	Some
N 2	4.9	2.0	CD	CP	Park	NONE	NONE	NONE	rarely
N 3	3.6	1.5	CD	CB	Park	NONE	NONE	NONE	rarely
N 4	3.4	1.4	CD	CP 1	Commercial	One (exit of outlet)	NONE	NONE	Some 1
			+CD	+CD 2		Two(entranc e of outlets)			Some 2
N 5	50.6	20.5	CD	CD (heavily damaged)	Commercial	NONE	NONE	slightly	some
N 6	84.3	34.1	CD	CD	Mostly commercial	NONE	NONE	Outlet could be seen	
N 7	11.8	4.8	CD	CD	Mostly residential	NONE	NONE	NONE	rarely
N 8	25.8	10.4	CD	CD	Residential	One (middle of main drain)	Some at screen	NONE	NONE
N 9	134.0	54.2	CD	CD (partly damaged)	Mostly residential	NONE	Some along main drain	slightly	heavily
N10	8.2	3.3	CD (no flow)	DC (totally silted)	Residential	NONE	Some	Totally silted up	heavily
N11	29.7	12.0	CD	CB	Residential	NONE	NONE	slightly	some
N12	396.0	156.2	CD	CP (one of two pipes totally silted)	Residential	NONE	Very few	Half silted up	heavily
N13	9.0	3.2	CD	CB	Residential	NONE	NONE	Totally silted up	heavily
N14	97.7	39.5	CD	CB	Residential	NONE	NONE	slightly	Some
N15	8.6	3.5	CD	CP (partly damaged)	Residential	NONE	NONE	NONE	rarely
N16	395.0	159.9	CD	CP	Residential	NONE	Very few	slightly	rarely
N17	10.0	4.0	CD	CP (partly damaged)	Residential	NONE	NONE	NONE	NONE
N18	27.3	11.0	CD	CP	Residential	NONE	Very few	NONE	rarely
N19	64.0	25.9	CD(partly CP)	CP	Residential	NONE	Some	NONE	some
N20	241.0	97.5	CD partly	CD	Residential	One near Jin Kelawai	Some	heavily	some
N21	181.0	73.3	CD partly	Earth	Residential	NONE	Some	heavily	some

Note: (*) CD = Channelized open drain
CP = Channelized pipe culvert
CB = Channelized box culvert
(**) Source = Technical Report No. 14, Penang Island Structure Plan, 1985

TABLE I-1-2 CHARACTERISTICS OF EXISTING OUTFALLS IN SOUTH CHANNEL

Existing Outfall	Catchment Area** (acres)	(ha)	Type * of Main Drain	Type * or Outfall	Land Use of Catchment Area	Screen	Remarks - (Conditions of outfall) - Garbage	Siltation	Tidal Effect
S 1	13.6	5.5	CD	C	Commercial	NONE	NONE	NONE	NONE
S 2	8.7	3.5	CD	C	Commercial	NONE	NONE	NONE	NONE
S 3	15	6.1	CD	CD	Commercial	NONE	Some	sightly	Some
S 4	15	6.1	CD	CD	Commercial	NONE	Some	sightly	Some
S 5	17.2	7	CD	CB	Commercial	NONE	Some	NONE	rarely
S 6	12.6	5.1	CD	Earth	Commercial	NONE	Some	NONE	Some
S 7	18.1	7.3	CDCD	Earth	Commercial	NONE	Some	NONE	Some
S 8	11.5	4.7	CD	Earth	Commercial	NONE	Some	NONE	Some
S 9	10	4	CD	Earth	Commercial	NONE	SomA lote	NONE	Some
S10	279	113	CD	Earth	Commercial	Two at pumpint station		NONE	Some
S11	3	1.2	(almost not usable) E	(almost not usble) E	Residential	NONE	Some	sightly	Some
S12	3.9	1.6	CD, E	E	Residential	NONE	Some	sightly	Some
S13	11.5	4.7	E	E	Residential	NONE	Some	sightly	Some
S14	2.6	1.1	E	E	Mostly residential	NONE	Some	sightly	Some
S15	2.6	1.1	Mostly CD	CD	Residential	NONE	Some	NONE	NONE
S16	30	12.1	CD	CD	Residential	One at the secondary drain	A lot	NONE	NONE
S17	13.1	5.3	CD	CD	Residential	NONE	Some	NONE	NONE
S18	292	118.2	CD	CD	Commercial (mostly)	NONE	Some	NONE	Some
S19	2.3	0.9	CD	CD	Residential	NONE	Some	NONE	rarely
S20	5.8	2.3	CD	CD	Residential	NONE	NONE	NONE	rarely
S21	5.9	2.4	CD	CD	Residential	NONE	NONE	Some	Some
S22	96.5	39.1	CD	E	Residential	NONE	A lot	heavily	Some
S23	1305	528	CD	CB	Residential	NONE	Some	NONE	NONE
S24	104.3	42.2	Mostly E	E	Residential	NONE	Some	NONE	rarely

Note: (*) CD = Channelized open drain
CP = Channelized pipe culvert
CB = Channelized box culvert
E = Earth
(**) Source = Technical Report No. 14, Penang Island Structure Plan, 1985

TABLE I-2 EXISTING DISCHARGE CAPACITY OF MAIN DRAIN AND OUTFALL IN NORTH AND SOUTH CHANNEL

No.of Existing Outfall	Discharge Capacity (m3/s)		Remark
	Main Drain	Outfall	
N1	-	5.2	a
N2	-	0.7	a
N3	0.1	0.2	d
N4	0.4	0.4	b
N5	0.4	0.4	d
N6	1.8	2.6	d
N7	0.4	0.4	d
N8	2.0	2.0	b
N9	3.7	1.9	d
N10	-	-	-
N11	1.1	-	-
N12	6.2	6.7	d
N13	3.1	3.1	a
N14	1.1	3.0	d
N15	1.8	1.8	a
N16	12.8	8.4	d
N17	1.2	1.2	a
N18	2.9	7.6	a
N19	6.8	4.1	a

No.of Existing Outfall	Discharge Capacity (m3/s)		Remark
	Main Drain	Outfall	
S1	-	-	-
S2	0.2	1.4	d
S3	2.2	2.7	a
S4	1.6	1.6	a
S5	0.7	3.7	d
S6	0.3	4.3	d
S7	1.2	0.5	d
S8	1.2	1.7	d
S9	1.8	1.8	d
S10	15.4	15.4	b
S11	-	-	-
S12	0.5	0.5	a
S13	-	-	-
S14	-	-	-
S15	0.4	0.4	d
S16	5.7	12.3	a
S17	3.7	3.7	a
S18	2.8	2.8	d
S19	0.7	0.7	a
S20	2.4	2.4	a
S21	0.6	0.6	a
S22	3.4	2.3	d
S23	80.7	80.7	a
S24	0.6	0.6	d

Remark
The existing flow capacity was categorized by return periods as follow ;
a > 10 year return period, 10 > b > 5, 5 > c > 2, 2 > d

TABLE I-3 DRAIN RESERVE

Width of Drain	Drain Reserve
W < 3m	One side 1.5 m
3m < W < 7.5 m	One side 4.5 m
7.5 m < W < 15 m	One side 3 m, another side 6.0 m
15 m < W	Both sides 6 m

TABLE I-4 RUN-OFF COEFFICIENTS FOR URBAN CENTRE

Landuse	Run-off Coefficient
Business:-	
City Areas Fully Built-up and Shophouses	0.90
Industrial:-	
Fully Built-up	0.80
Residential:-	
4 houses/acre	0.55
4 - 8 houses/acre	0.65
8 - 12 houses/acre	0.75
12 houses/acre	0.85
Pavement	0.95
Parks (normally flat in urban areas)	0.30
Rubber	0.45
Jungle (normally steep in urban areas)	0.35
Mining Land	0.10
Bare Earth	0.75

Source:Hydrological Procedure No.16

TABLE I-5 PROBABLE FLOOD DISCHARGE OF DRAINAGE BASINS IN NORTH AND SOUTH CHANNEL

NORTH CHANNEL

NO. of Existing Outfall	Revised Catchment Area (ha)	Peak Discharge (m3/s)		
		Q2	Q5	Q10
N1	20.2	2.29	2.78	3.27
N2	2.5	0.22	0.26	0.31
N3	1.5	0.19	0.23	0.27
N4	1.4	0.17	0.20	0.24
N5	20.5	2.69	3.26	3.84
N6	34.1	3.96	4.80	5.65
N7	4.8	0.69	0.84	0.99
N8	10.4	1.43	1.73	2.04
N9	54.2	5.66	6.88	8.09
N10	3.3	0.29	0.36	0.42
N11	12.0	1.55	1.89	2.22
N12	156.0	12.50	15.17	17.85
N13	3.2	0.46	0.56	0.66
N14	39.5	5.49	6.66	7.84
N15	3.5	0.52	0.63	0.74
N16	125.5	11.00	13.35	15.71
N17	4.0	0.29	0.35	0.41
N18	11.0	0.95	1.16	1.36
N19	25.9	2.51	3.04	3.58

SOUTH CHANNEL

NO. of Existing Outfall	Revised Catchment Area (ha)	Peak Discharge (m3/s)		
		Q2	Q5	Q10
S1	5.5	0.78	0.94	1.11
S2	3.5	0.48	0.59	0.69
S3	6.1	0.85	1.03	1.21
S4	6.1	0.85	1.03	1.21
S5	7.0	0.99	1.21	1.42
S6	5.1	0.70	0.85	1.00
S7	7.3	1.00	1.22	1.43
S8	4.7	0.63	0.77	0.90
S9	7.6	1.00	1.22	1.43
S10	120.0	12.71	15.44	18.16
S11	-	-	-	-
S12	1.6	0.19	0.23	0.27
S13	4.7	0.59	0.71	0.84
S14	5.3	0.58	0.71	0.83
S15	6.6	0.76	0.93	1.09
S16	12.0	1.38	1.67	1.97
S17	6.8	0.62	0.76	0.89
S18	104.0	10.42	12.65	14.88
S19	0.9	0.09	0.11	0.13
S20	2.3	0.28	0.34	0.4
S21	2.4	0.26	0.31	0.37
S22	48.8	5.06	6.15	7.23
S23	-	-	-	-
S24	42.2	5.12	6.22	7.32

TABLE I-6-1 10 YEAR FLOOD DISCHARGE AT EXISTING OUTFALL IN THE NORTH CHANNEL

	(ha)	(m)		(%)	sq. mile	(mile)	(HOUR)	(HOUR)			(inch)	(mm)	(m^3/s)	(m^3/s)	(m^3/s)
NORTH	a	l	s	S	A	L	Tc	Td	Cs	C	II	I	Q	EX. MAIN	EX. OUTFALL
1	20.2	185	0.00100	0.10	0.078	0.1150	0.11	0.09	0.72	0.667	4.8	121.92	3.27	-	5.2
2	2.5	18	0.00100	0.10	0.010	0.0112	0.01	0.01	0.66	0.546	4.8	121.92	0.31	-	0.7
3	1.5	225	0.00100	0.10	0.006	0.1398	0.10	0.10	0.65	0.778	5	127	0.27	0.1	0.2
4	1.4	210	0.00100	0.10	0.005	0.1305	0.09	0.10	0.65	0.792	4.8	121.92	0.24	0.4	0.4
5	20.5	345	0.00100	0.10	0.079	0.2144	0.20	0.16	0.72	0.772	4.8	121.92	3.84	0.4	0.4
6	34.1	680	0.00100	0.10	0.132	0.4226	0.42	0.31	0.73	0.786	4.1	104.14	5.65	-	-
7	4.8	270	0.00100	0.10	0.019	0.1678	0.13	0.13	0.68	0.876	4.9	124.46	0.99	0.4	0.4
8	10.4	315	0.00167	0.17	0.040	0.1958	0.13	0.15	0.65	0.873	4.9	124.46	2.04	2	2
9	54.2	815	0.00100	0.10	0.209	0.5065	0.54	0.38	0.74	0.773	3.7	93.98	8.09	-	-
10	3.3	260	0.00100	0.10	0.013	0.1616	0.12	0.12	0.67	0.540	5	127	0.42	-	-
11	12	240	0.00100	0.10	0.046	0.1492	0.13	0.11	0.70	0.758	4.9	124.46	2.22	1.1	-
12	156	1240	0.00100	0.10	0.602	0.7707	0.92	0.57	0.76	0.759	2.8	71.12	17.85		
13	3.2	250	0.00100	0.10	0.012	0.1554	0.12	0.12	0.67	0.876	5	127	0.66	1.1	-
14	39.5	380	0.00100	0.10	0.153	0.2362	0.24	0.18	0.73	0.817	4.7	119.38	7.84	1.1	3
15	3.5	75	0.00100	0.10	0.014	0.0466	0.04	0.03	0.67	0.885	5	127	0.74	1.8	1.8
16	125.5	1635	0.00390	0.39	0.485	1.0162	0.63	0.76	0.62	0.812	3.5	88.9	15.71	-	-
17	4	230	0.00909	0.91	0.015	0.1429	0.04	0.11	0.43	0.672	5	127	0.41	1.2	1.2
18	11	400	0.00820	0.82	0.042	0.2486	0.08	0.19	0.47	0.747	5	127	1.36	2.9	7.6
19	25.9	500	0.00990	0.99	0.100	0.3108	0.10	0.23	0.47	0.830	5	127	3.58	-	-

TABLE I-6-2 10 YEAR FLOOD DISCHARGE AT EXISTING OUTFALL IN THE SOUTH CHANNEL

	(ha)	(m)		(%)	sq. mile	(mile)	(HOUR)	(HOUR)			(inch)	(mm)	(m^3/s)	(m^3/s)	(m^3/s)
SOUTH	a	l	s	S	A	L	Tc	Td	Cs	C	II	I	Q	Trunk Drain	Outfall
1	5.5	200	0.00100	0.10	0.021	0.1243	0.10	0.09	0.69	0.869	4.8	121.92	1.11	-	-
2	3.5	76	0.00100	0.10	0.014	0.0472	0.04	0.04	0.67	0.869	4.8	121.92	0.69	0.2	1.4
3	6.1	254	0.00100	0.10	0.024	0.1579	0.13	0.12	0.69	0.870	4.7	119.38	1.21	2.2	2.7
4	6.1	228	0.00100	0.10	0.024	0.1417	0.12	0.11	0.69	0.870	4.7	119.38	1.21	1.6	1.6
5	7	215	0.00100	0.10	0.027	0.1336	0.11	0.10	0.69	0.869	4.8	121.92	1.42	0.7	3.7
6	5.1	463	0.00100	0.10	0.020	0.2878	0.23	0.21	0.68	0.869	4.7	119.38	1.00	0.3	4.3
7	7.3	512	0.00100	0.10	0.028	0.3182	0.27	0.24	0.69	0.869	4.6	116.84	1.43	1.2	0.5
8	4.7	523	0.00100	0.10	0.018	0.3250	0.26	0.24	0.68	0.868	4.6	116.84	0.90	1.2	1.7
9	7.6	575	0.00100	0.10	0.029	0.3574	0.30	0.27	0.69	0.854	4.5	114.3	1.43	1.8	1.8
10	120	960	0.00100	0.10	0.463	0.5966	0.69	0.44	0.76	0.858	3.3	83.82	18.16		
11															
12	1.6	420	0.00207	0.21	0.006	0.2610	0.13	0.19	0.57	0.881	4.7	119.38	0.27	0.5	0.5
13	4.7	336	0.00207	0.21	0.018	0.2088	0.12	0.16	0.60	0.876	4.8	121.92	0.84	-	-
14	5.3	800	0.00100	0.10	0.020	0.4972	0.40	0.37	0.68	0.773	4.2	106.68	0.83	-	-
15	6.6	642	0.00100	0.10	0.025	0.3990	0.33	0.30	0.69	0.768	4.4	111.76	1.09	0.4	0.4
16	12	762	0.00100	0.10	0.046	0.4736	0.42	0.35	0.70	0.805	4.1	104.14	1.97	-	-
17	6.8	130	0.00680	0.68	0.026	0.0808	0.03	0.06	0.48	0.809	4.8	121.92	0.89	3.7	3.7
18	104	920	0.00100	0.10	0.402	0.5718	0.65	0.43	0.75	0.791	3.4	86.36	14.88	-	-
19	0.9	15	0.00267	0.27	0.003	0.0093	0.00	0.01	0.53	0.794	4.8	121.92	0.13	0.7	0.7
20	2.3	210	0.00133	0.13	0.009	0.1305	0.08	0.10	0.63	0.820	4.8	121.92	0.40	2.4	2.4
21	2.4	250	0.00267	0.27	0.009	0.1554	0.07	0.12	0.56	0.821	4.8	121.92	0.37	0.6	0.6
22	48.8	1050	0.00238	0.24	0.188	0.6526	0.46	0.49	0.65	0.805	4	101.6	7.23	3.4	2.3
23															-
24	42.2	460	0.00238	0.24	0.163	0.2859	0.20	0.21	0.65	0.807	4.7	119.38	7.32	0.6	0.6

TABLE I-7-1 REQUIRED IMPROVEMENT WORKS OF DRAINS IN NORTH CHANNEL

No.of Existing Outfall	Catchment Area (ha)	Flow Capacity	MEASURE OF IMPROVEMENT			No. of New Outfall	Proposed C.A.(ha)	Q_{10} Discharge (m3/s)
			Reorganization	Realignment	Widening/ Deepening			
N1	20.2	a				PN1	20.2	3.30
N2	2.5	a				PN2	2.5	0.30
N3	1.5	d		EXT.	o	PN3	2.7	0.46
N4	1.4	a		EXT.		PN4	3.0	0.54
N5	20.5	d		EXT.	o	PN5	25.5	4.70
N6	34.1	d		EXT.	o	PN6	39.4	6.56
N7	4.8	d		EXT.	o	PN7	6.6	1.32
N8	10.4	b	o	EXT.	o	PN8	17.3	2.33
N9	54.2	d	o	EXT.	o	PN9	43.7	9.04
N10	3.3	-	o	EXT.	o	-	-	
N11	12.0	-	o	EXT.	o	PN10	25.8	2.56
N12	156.0	d	o	EXT.	o	PN11	161.8	17.60
N13	3.2	a		o EXT.		-	(4.9)	
N14	39.5	d		o EXT.	o	PN12	45.6	9.40
N15	3.5	a		o EXT.		-	(4.8)	
N16	125.5	d		EXT.	o	PN13	126.3	16.30
N17	4.0	a		o EXT.		-	(4.9)	
N18	11.0	a		o EXT.		-	(12.4)	
N19	25.9	a		o EXT.		PN14	46.0	6.39

Remark
The existing flow capacity was categorized by return periods as follow ;
a > 10 year return period, 10 > b > 5, 5 > c > 2, 2 > d

TABLE I-7-2 REQUIRED IMPROVEMENT WORKS OF DRAINS IN SOUTH CHANNEL

No.of Existing Outfall	Catchment Area (ha)	Flow Capacity	MEASURE OF IMPROVEMENT Reorganization	Realignment	Widening/ Deepening	No. of New Outfall	Proposed C.A.(ha)	Q 10 Discharge (m3/s)
S1	5.5	-				PS1	5.5	1.10
S2	3.5	d			o	PS2	3.5	0.69
S3	6.1	a				PS3	6.1	1.21
S4	6.1	a				PS4	6.1	1.21
S5	7.0	d			o	PS5	7.0	1.42
S6	5.1	d		EXT.	o	PS6	9.0	1.77
S7	7.3	d		EXT.	o	PS7	12.2	2.37
S8	4.7	a		EXT.		PS8	9.2	1.77
S9	7.6	a		EXT.		PS9	16.6	3.14
S10	120.0	b		o EXT.	o	PS10	126.3	18.37
S11	-	-		o EXT.	o	-	-	
S12	1.6	a		o EXT.		-	(1.6)	
S13	4.7	-		o EXT.	o	-	(4.7)	
S14	5.3	-		EXT.	o	PS11	8.0	1.24
S15	6.6	d		o EXT.	o	-	(6.6)	
S16	12.0	a		o EXT.		PS12	25.3	4.05
S17	6.8	a				PS13	6.8	0.89
S18	104.0	d		EXT.	o	PS14	104.0	14.85
S19	0.9	a				PS15	0.9	0.13
S20	2.3	a				PS16	2.3	0.40
S21	2.4	a				PS17	2.4	0.37
S22	48.8	d			o	PS18	48.8	7.23
S23	528.0	a	Jelutong Diversion EXT.			-	-	
S24	42.2	d			o	PS19	42.2	7.32

Remark
The existing flow capacity was categorized
by return periods as follow ;
a > 10 year return period, 10 > b > 5, 5 > c > 2, 2 > d

TABLE I-8 FEATURES OF LOWLYING AREAS IN PENANG ISLAND

CATCH-MENT NO.	NAME	Lowest Ground Level	Existence of Lowlying Areas	Experienced Flooding Area (ha)	Location	Causes of Inundation	Counter Measures
13	Sg. Gelugor	3 m	exist		Brown Garden	Inadequate flow	Realignment of Outfall
					Minden Heights	capacity of	
						main stream	
14	Sg. Dua Besar	1.5m	exist		Ch 00-800	Hightide	Pumping
16	Sg. Nibong Kecil		exist				
17	Sg. Keluang	1.4 m	exist		Mid & downstream	Lowlying hightide	Filling up
18	Sg. Nipah	1.0 m	exist		Downstream	Lowlying hightide	Filling up
19	Sg. Kampung Masjid	1.0 m	exist		Downstream	Lowlying hightide	Filling up
22	Sg. Batu	1.6 m	exist		Downstream		Filling up
24	Sg. Teluk Kumbar	1.2 m	exist		Downstream		Filling up

2	Sg.Teluk Awak	
3	Sg. Teluk Bahang	
4	Sg. Batu Ferringghi	
5	Sg. Satu	
6	Sg. Mas	
7	Sg. Kecil	In these river basins
8	Sg. Kelian	existing ground level is higher than 2.00 m.
9	Sg. Balik Batu	(except for Sg. Ikan Mati 1.8 m
10	Sg. Fettes	and Sg. Bayan Lepas 1.7 m).
11	Sg. Bagan Jermal	There is no lowlying areas and also there is
12	Sg. Babi	no experienced flooding area in these catchments
15	Sg. Nibong Besar	
20	Sg. Ikan Mati	
21	Sg. Bayan Lepas	
23	Sg. Mati	
25	Sg. Gemuroh	
26	Sg. Gertak Sanggul	

Table I-9 CAUSES OF INUNDATION AND COUNTERMEASURES IN THE AREAS OUTSIDE GEORGETOWN

Location	Causes of inundation	Flooding Area (ha)	Countermeasures
Sg. Gelugor			
- Brown Garden	Inadequate flow capacity of main stream	5	1; Improvement of Sg. Gelugor 2; Realignment of existing drainage outfall
- Minden Heights	Existence of lowlying area Inadequate flow capacity of drain.	4	1; Improvement of original Sg. Gelugor 2; Improvement of drainage channel 3; Recommendation of retention pond near USM
Sg. Dua Besar			
- Downstream of Jln. Sg. Nibong	Existence of lowlying area Failure in drain from lowlying area along the main stream		1; Improvement of main river 2; Installation of small scale drainage pump
- Area around PESTA	Inadequate flow capacity of main stream	Total	Improvement of main river
- to Jln. Sg. Dua	Inadequate flow capacity of main stream	60	Improvement of main river and land fill
- along the Diversion	Inadequate flow capacity of the Diversion Channel Existence of lowlying area		1; Improvement of the existing diversion channel 2; Installation of small scale drainage pump
Sg. Nibong			
- Jln. Relau	Extra run-off from Bukit Jambul	Total	1; Construction of road side drain 2; Construction of new drain to Sg. Relau
- Jln. Mahsuri	Inadequate flow capacity of drain along Jln. Relau	20	Expected to be solved by countermeasures for the area along Jln. Relau
Sg. Keluang			
- Jln. Tun Dr. Awang	Inadequate flow capacity of main stream	10	Improvement of Sg. Ara
- PDC Housing Area	Inadequate flow capacity of main stream	30	Expected to be solved by construction of Relau Diversion Channel
- Jln. Bayan Lepas	Actual area was not identified		
Sg. Nipah	Filling land without proper alternative drains Existence of lowlying area		1; Construction adequate alternative drain 2; Land filling of lowlying area
Sg. Kg. Masjid	Existence of lowlying area		Land filling of lowlying area
Sg. Kg. Seronok (Sg.Bayan Lepas)	Inadequate flow capacity of concrete channel stretch along the road and improper operation of intake gate for irrigation	6	Improvement of Sg. Kg. Seronok downstream reaches

TABLE I- 10 FEATURES OF PROPOSED DRAINAGE FACILITIES IN THE AREAS OUTSIDE GEORGETOWN

Location	Countermeasures	Length	Size	Remarks
Sg. Gelugor				
- Brown Garden	1; Improvement of main river	-	-	
	2; Realignment of existing drainage outfall	-	-	
- Minden Heights	1; Improvement of original Sg. Gelugor	650 m	5.3 m (Bed W) x 1.7 m (Water D) Slope=1/450	Earth Channel (1:1)
	2; Improvement of drainage channel	85m	3.0 m (Bed W) x 1.5 m (Water D) Slope=1/200	Concrete Drain
	3; Recommendation of retention pond near USM	-		
Sg. Dua Besar				
- Downstream of Jln. Sg. Nibong	1; Improvement of main river	-	-	
	2; Installation of small scale drainage pump	-	-	
- Area around PESTA	Improvement of main river	-	-	
- to Jln. Sg. Dua	Improvement of main river and land fill	-	-	
- along the Diversion	1; Improvement of the Diversion Channel	about 1,600 m	1.9 m (Bed W) x 1.7 m (Water D) Slope=1/650	Earth Channel (1:1)
	2; Installation of small scale drainage pump	-		
Sg. Nibong				
- Jln Sg. Relau	Construction of road side drain	1,200 m	Upstream 1.2 m (W) x 1.0 m (D) S=1/300	Concrete Drain
			Downstream 1.5 m (W) x 1.2 m (D) S=1/300	Concrete Drain
	Construction of new drain to Sg. Relau	450 m	3.5 m (W) x 1.5 m (D) S=1/600	Concrete Drain
- Jln Mahsuri	Expected to be solved by countermeasures for the area along Jln. Relau	-	-	
Sg. Keluang				
- Jln Tun Dr. Awang	Improvement of Sg. Ara	-	-	-
- PDC Housing Area	Expected to be solved by construction of Relau Diversion Channel	-	-	-
Sg. Nipah	1; Construciton adequate alternative drain	-	-	-
	2; Land filling of lwlying area	-	-	
Sg. Kg. Masjid	Land filling of lowlying area	-	-	-
Sg. Kg. Seronok (Sg. Bayan Lepas)	Improvement of Sg. Kg. Seronok downstream reaches	about 500 m	Upstream 5.0 m (W) x 1.5 m (D) S=1/500	Concrete Channel
			Downstream 2.5 m (W) x 1.5 m (D) S=1/110	Earth Channel

TABLE I-11 FINANCIAL COST OF GEORGETOWN DRAINAGE MASTER PLAN

NORTH NO.	(10^3 M$) Direct costs	(10^3 M$) Evacuation costs	(10^3 M$) Administration costs	(10^3 M$) Enginering services costs	(10^3 M$) Contingency	(10^3 M$) TOTAL
1	0	0	0	0	0	0
2	0	0	0	0	0	0
3	262	0	13	13	58	346
4	253	0	13	13	56	334
5	391	0	20	20	86	516
6	708	78	35	35	171	1,029
7	307	175	15	15	102	615
8	352	194	18	18	116	698
9	1,373	466	69	69	395	2,371
10	293	0	15	15	65	387
11	289	0	14	14	64	381
12	2,188	0	109	109	481	2,889
13	283	0	14	14	62	374
14	659	0	33	33	145	870
15	239	0	12	12	53	316
16	2,710	0	136	136	596	3,577
17	185	0	9	9	41	244
18	673	0	34	34	148	889
19	199	0	10	10	44	263
TOTAL	11,365	912,270	568	568	2,683	16,098

SOUTH NO.	(10^3 M$) Direct costs	(10^3 M$) Evacuation costs	(10^3 M$) Administration costs	(10^3 M$) Enginering services costs	(10^3 M$) Contingency	(10^3 M$) TOTAL
1	0	0	0	0	0	0
2	157	0	8	8	34	207
3	0	0	0	0	0	0
4	0	0	0	0	0	0
5	269	0	13	13	59	355
6	427	0	21	21	94	564
7	474	0	24	24	104	626
8	542	0	27	27	119	715
9	623	0	31	31	137	822
10	9,687	0	484	484	2,131	12,787
11	0	0	0	0	0	0
12	419	0	21	21	92	553
13	290	0	14	14	64	383
14	0	0	0	0	0	0
15	996	0	50	50	219	1,315
16	442	0	22	22	97	584
17	0	0	0	0	0	0
18	17,040	630	852	852	3,875	23,249
19	0	0	0	0	0	0
20	0	0	0	0	0	0
21	0	0	0	0	0	0
22	1,860	0	93	93	409	2,455
24	936	399	47	47	286	1,715
TOTAL	34,162	1,029	1,708	1,708	7,721	46,329

TOTAL AMOUNT (M$) 62,427

TABLE I-12 BILL OF QUANTITY FOR GEORGETOWN DRAINAGE MASTER PLAN

DRAINAGE

NORTH	(m) STRETCH	(m^3) EXCAVATION	(m^3) CONCRETE	(m^3) BACKFILL	(sq.m) PAVING	(nos.) WOODEN PILE
1	-	-	-	-	-	-
2	-	-	-	-	-	-
3	225	468	216	585	900	4,676
4	210	464	208	546	840	4,364
5	345	1,525	518	449	1,380	7,169
6	680	3,774	1,163	1,020	2,720	14,130
7	270	770	316	405	1,080	5,611
8	315	1,134	416	473	1,260	6,546
9	815	4,727	4,059	4,115	3,260	16,936
10	260	624	281	390	1,040	5,403
11	240	792	302	360	960	4,987
12	2,445	21,500	3,600	7,900	9,800	38,550
13	250	544	259	375	1,000	5,195
14	380	4,484	1,630	1,881	1,520	7,896
15	180	540	216	270	720	3,740
16	1,635	33,681	7,400	12,246	6,540	33,975
17	145	315	44	218	580	3,013
18	290	1,044	2,088	435	1,160	6,026
19	125	694	152	188	500	2,598
TOTAL	8,810	77,079	22,866	31,854	35,260	170,815

SOUTH	(m) STRETCH	(m^3) EXCAVATION	(m^3) CONCRETE	(m^3) BACKFILL	(sq.m) PAVING	(nos.) WOODEN PILE
1	-	-	-	-	-	-
2	76	188	83	114	304	1,579
3	-	-	-	-	-	-
4	-	-	-	-	-	-
5	215	688	271	344	860	4,468
6	390	1,593	579	741	1,560	8,104
7	432	2,052	687	821	1,728	8,977
8	523	2,354	785	785	2,092	10,868
9	575	3,364	1,018	863	2,300	11,949
10	1,660	32,000	5,700	13,500	11,700	34,500
11	-	-	-	-	-	-
12	240	600	960	360	960	4,987
13	143	358	572	215	572	2,972
14	-					-
15	642	2,215	2,761	963	2,568	13,341
16	150	1,350	1,275	225	600	3,117
17	-	-	-	-	-	-
18	4,530	105,000	31,000	50,000	34,800	80,180
19	-	-	-	-	-	-
20	-	-	-	-	-	-
21	-	-	-	-	-	-
22	1,050	6,300	5,880	1,575	4,200	21,819
24	460	1,656	2,944	828	1,840	9,559
TOTAL	11,086	159,717	54,514	71,332	66,084	216,419

TABLE I-13 BREAKDOWN OF CONSTRUCTION COST FOR GEORGETOWN DRAINAGE MASTER PLAN

DRAINAGE

NORTH	(10^3 M$) EXCAVATION	(10^3 M$) CONCRETE	(10^3 M$) BACKFILL	(10^3 M$) PAVING	(10^3 M$) WOODEN PILE	(10^3 M$) MISCELLANEOUS	(10^3 M$) TOTAL
1	-	-	-	-	-	-	-
2	-	-	-	-	-	-	-
3	4	43	2	90	23	100	262
4	4	42	2	84	22	100	253
5	12	104	1	138	36	100	391
6	30	233	3	272	71	100	708
7	6	63	1	108	28	100	307
8	9	83	1	126	33	100	352
9	38	812	12	326	85	100	1,373
10	5	56	1	104	27	100	293
11	6	60	1	96	25	100	289
12	172	720	24	980	193	100	2,188
13	4	52	1	100	26	100	283
14	36	326	6	152	39	100	659
15	4	43	1	72	19	100	239
16	269	1,480	37	654	170	100	2,710
17	3	9	1	58	15	100	185
18	8	418	1	116	30	100	673
19	6	30	1	50	13	100	199
TOTAL	617	4,573	96	3,526	854	1,700	11,365

SOUTH	(10^3 M$) EXCAVATION	(10^3 M$) CONCRETE	(10^3 M$) BACKFILL	(10^3 M$) PAVING	(10^3 M$) WOODEN PILE	(10^3 M$) MISCELLANEOUS	(10^3 M$) TOTAL
1	-	-	-	-	-	-	-
2	2	17	0	30	8	100	157
3	-	-	-	-	-	-	-
4	-	-	-	-	-	-	-
5	6	54	1	86	22	100	269
6	13	116	2	156	41	100	427
7	16	137	2	173	45	100	474
8	19	157	2	209	54	100	542
9	27	204	3	230	60	100	623
10	256	1,140	41	1,170	173	100	2,879
11	-	-	-	-	-	-	-
12	5	192	1	96	25	100	419
13	3	114	1	57	15	100	290
14	-	-	-	-	-	-	-
15	18	552	3	257	67	100	996
16	11	255	1	60	16	100	442
17	-	-	-	-	-	-	-
18	840	6,200	150	3,480	401	300	11,371
19	-	-	-	-	-	-	-
20	-	-	-	-	-	-	-
21	-	-	-	-	-	-	-
22	50	1,176	5	420	109	100	1,860
24	13	589	2	184	48	100	936
TOTAL	1,278	10,903	214	6,608	1,082	1,600	21,685

TOTAL AMOUNT 1,136

Figures

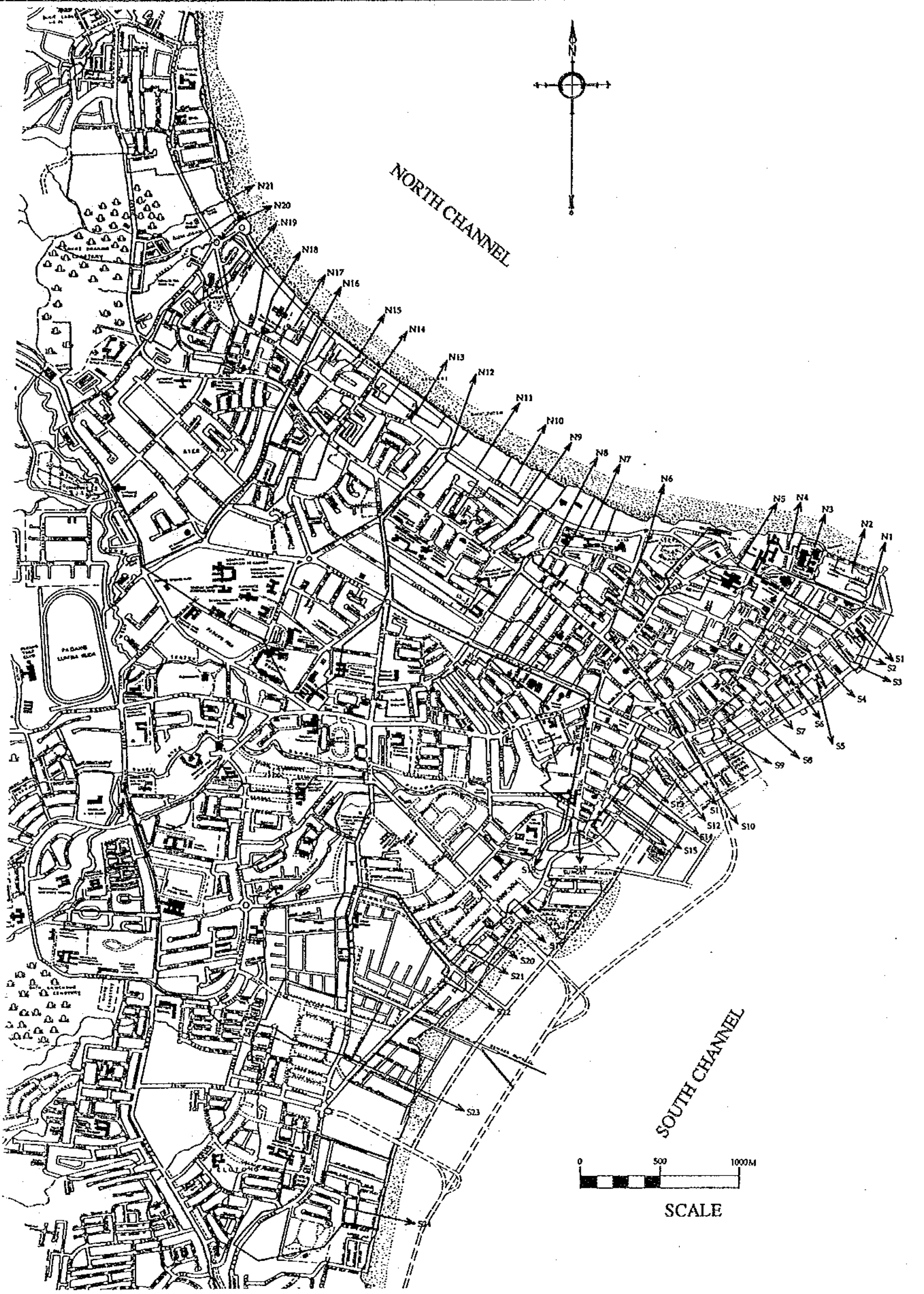

FIG. I-1	EXISTING OUTFALLS ALONG NORTH AND SOUTH CHANNELS

THE STUDY ON FLOOD MITIGATION AND DRAINAGE IN PENANG ISLAND

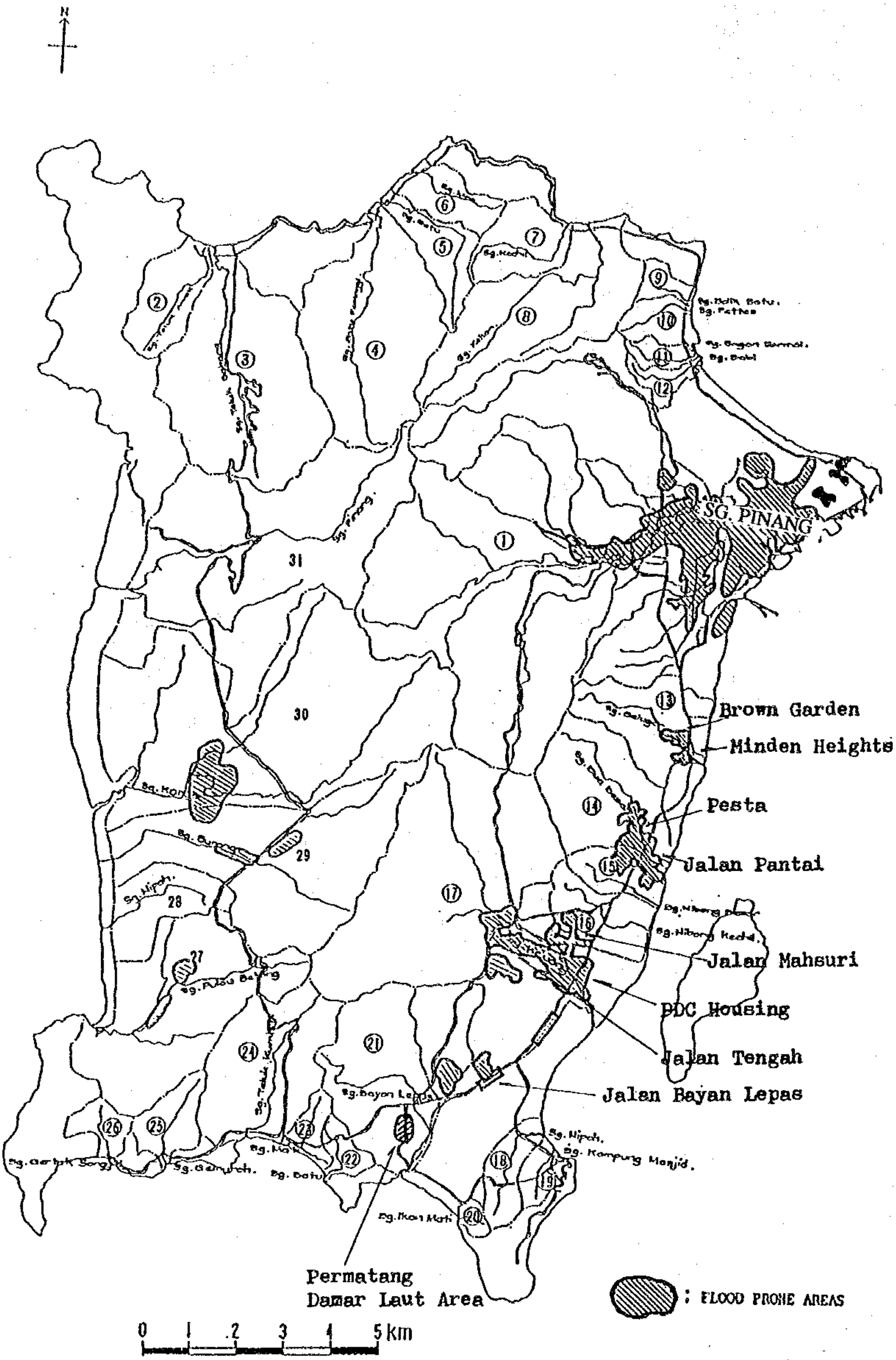

FIG. I-2	COMMONLY FLOODED AREAS IN PENANG ISLAND

THE STUDY ON FLOOD MITIGATION AND DRAINAGE IN PENANG ISLAND

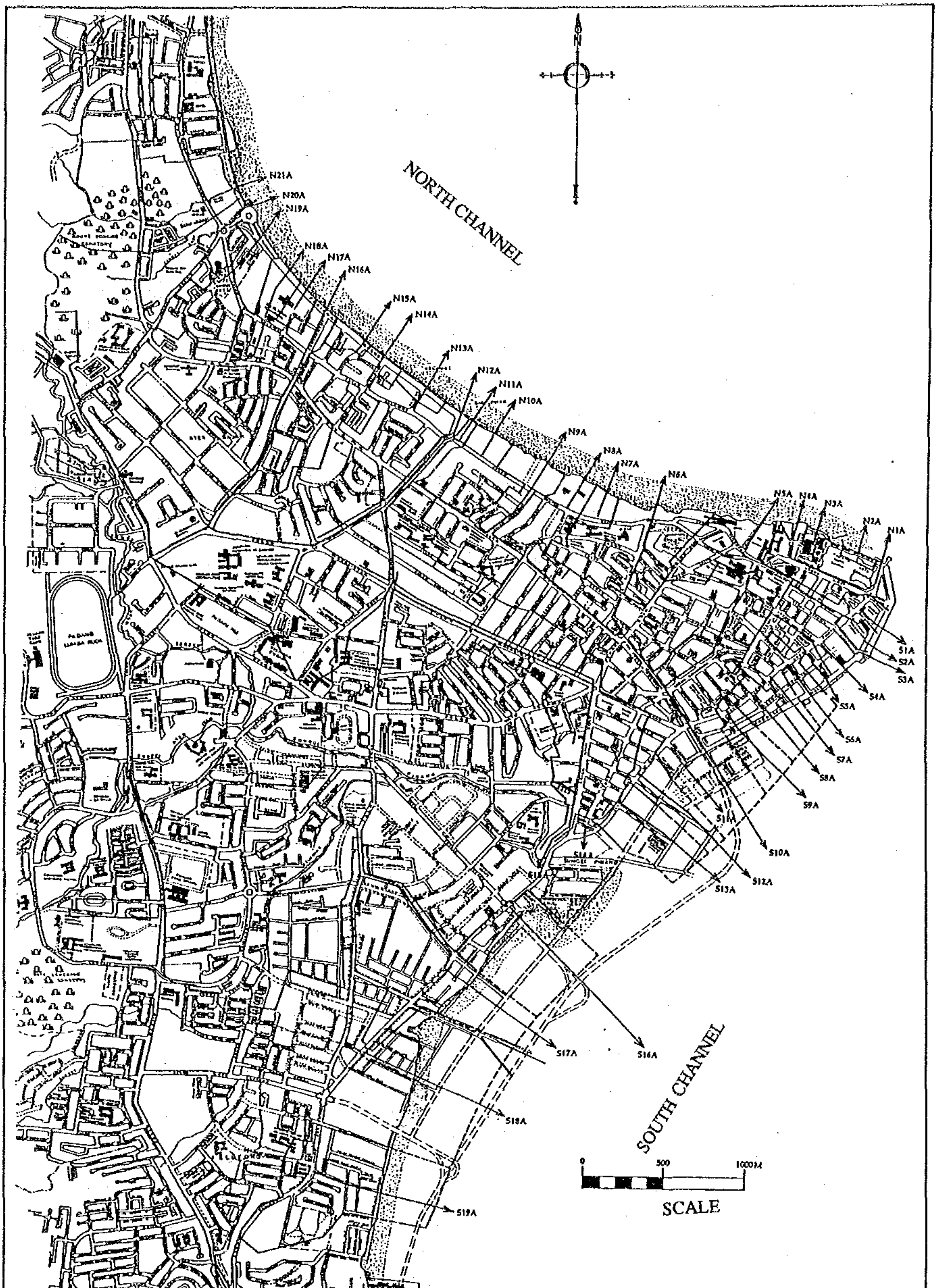

FIG. I-3	PROPOSED DRAINAGE OUTFALLS BY MPPP

THE STUDY ON FLOOD MITIGATION AND DRAINAGE IN PENANG ISLAND

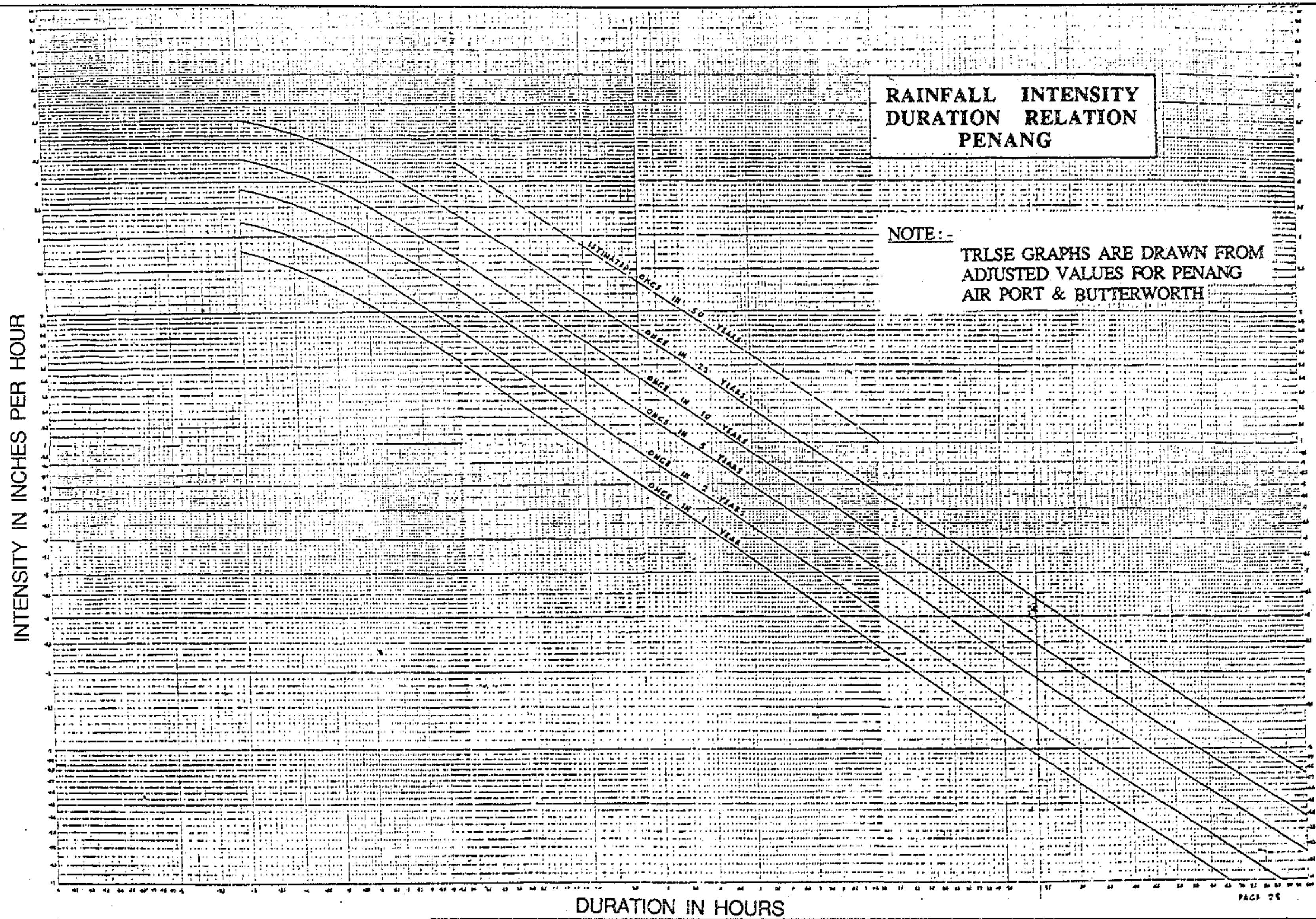

FIG. I-4	RAINFALL INTENSITY CURVE
THE STUDY ON FLOOD MITIGATION AND DRAINAGE IN PENANG ISLAND	

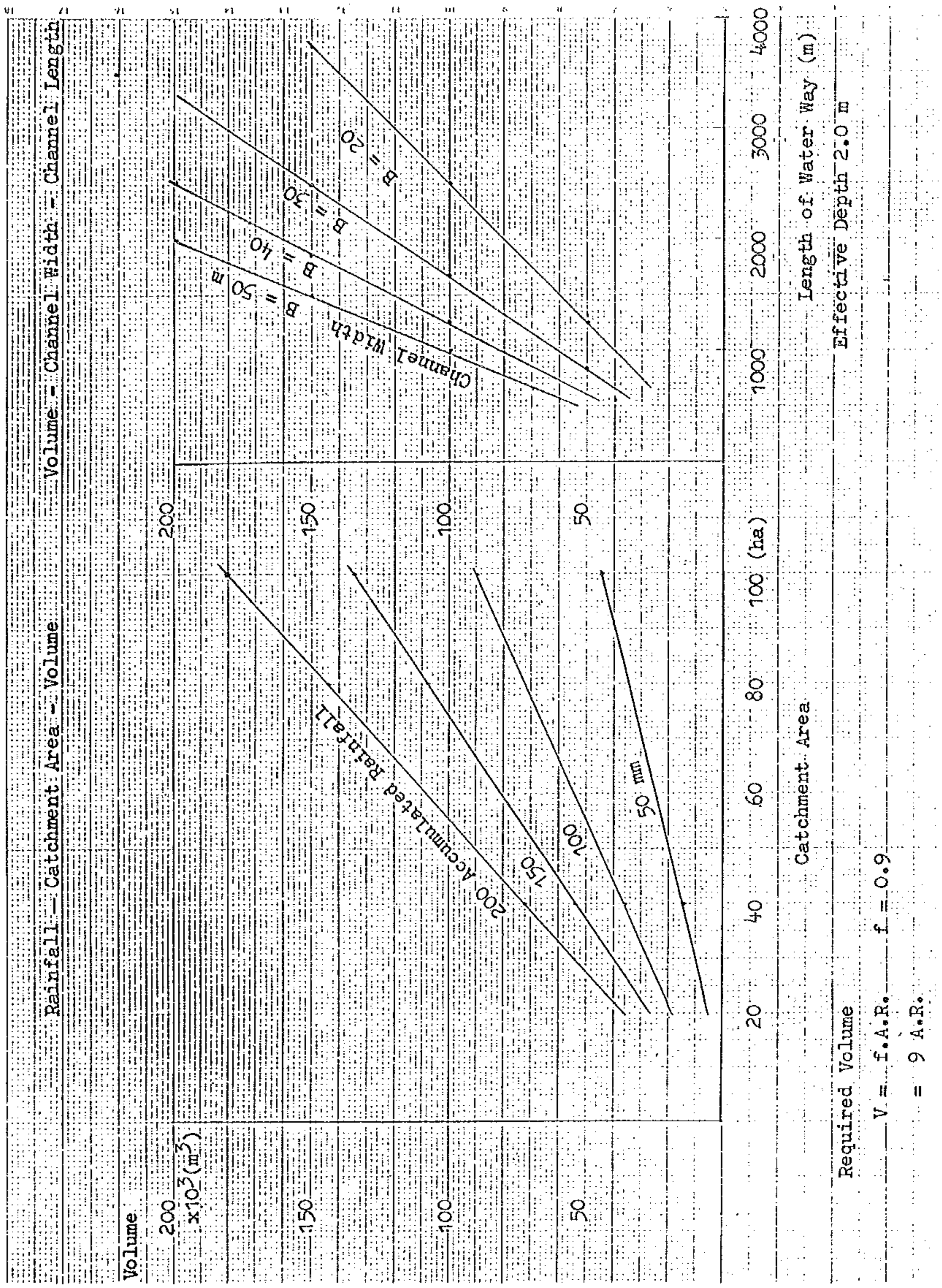

FIG. I-5	RELATIONSHIP AMONG THE NECESSARY VOLUME, CATCHMENT AREA AND RAINFALL

THE STUDY ON FLOOD MITIGATION AND DRAINAGE IN PENANG ISLAND

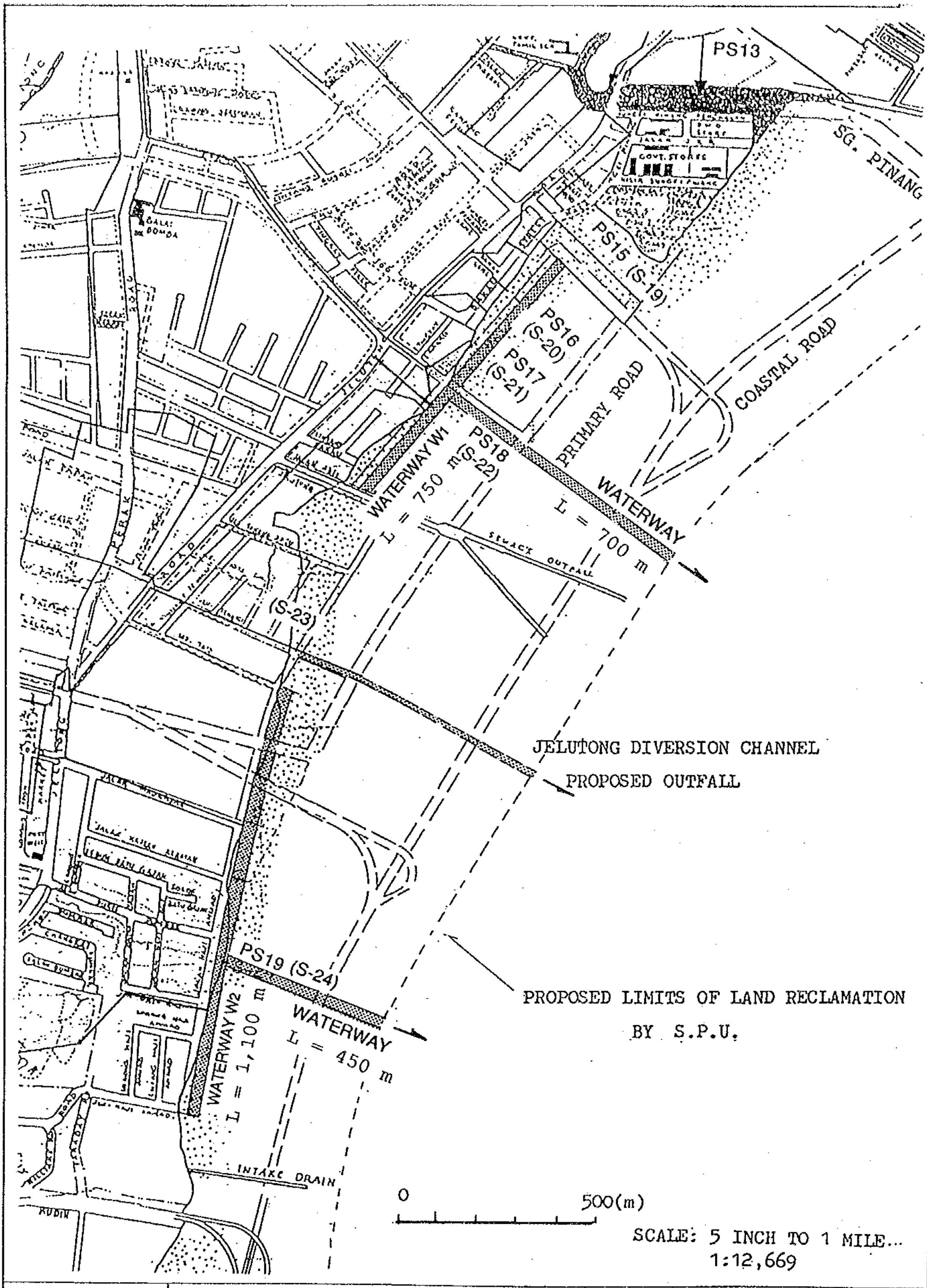

Fig. I-6	PROPOSED WATERWAY IN FUTURE RECLAMATION AREA IN SOUTH CHANNEL

THE STUDY ON FLOOD MITIGATION AND DRAINAGE IN PENANG ISLAND

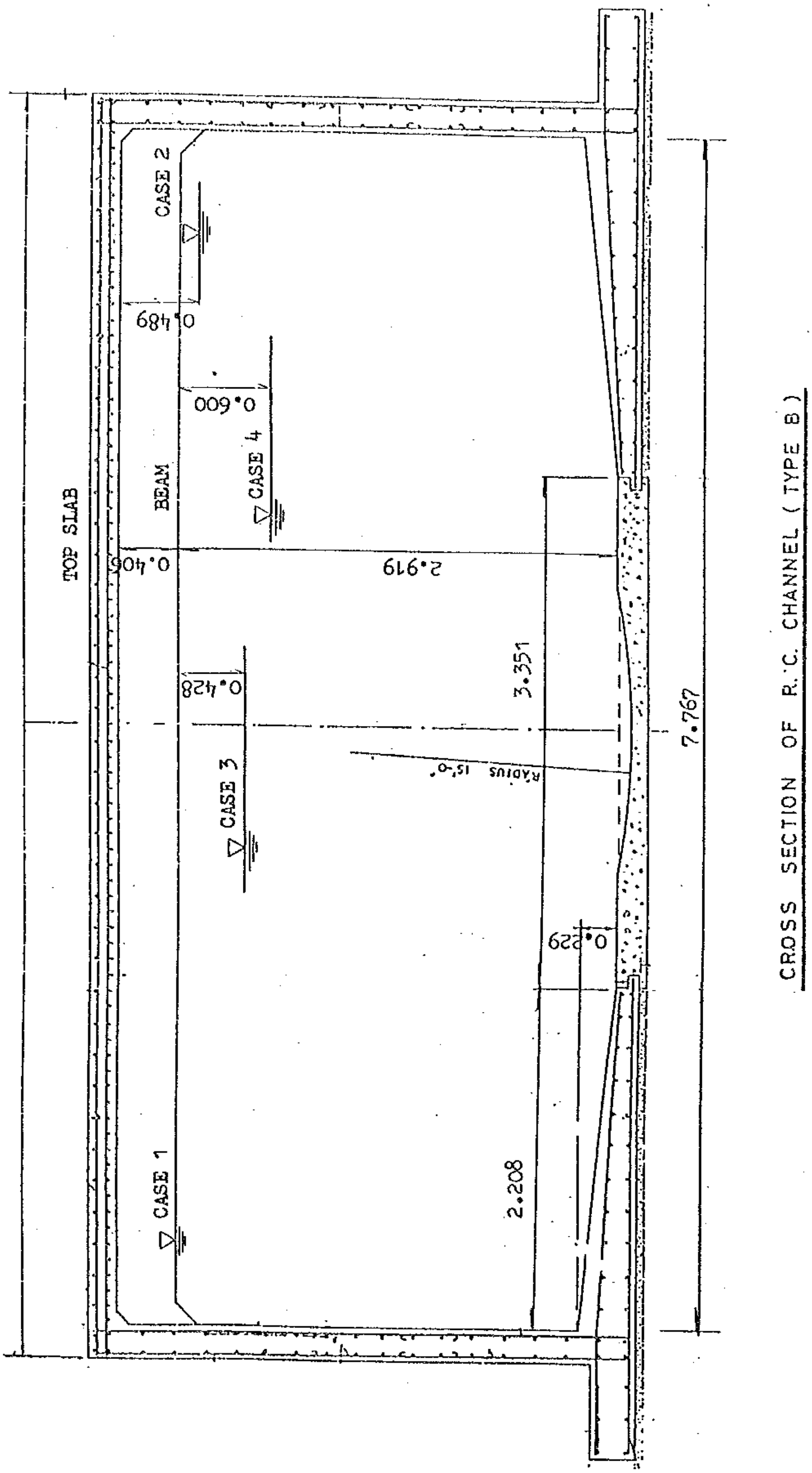

FIG. I-7	TYPICAL CROSS SECTION OF THE EXISTING JELUTONG DIVERSION CHANNEL

THE STUDY ON FLOOD MITIGATION AND DRAINAGE IN PENANG ISLAND

JICA

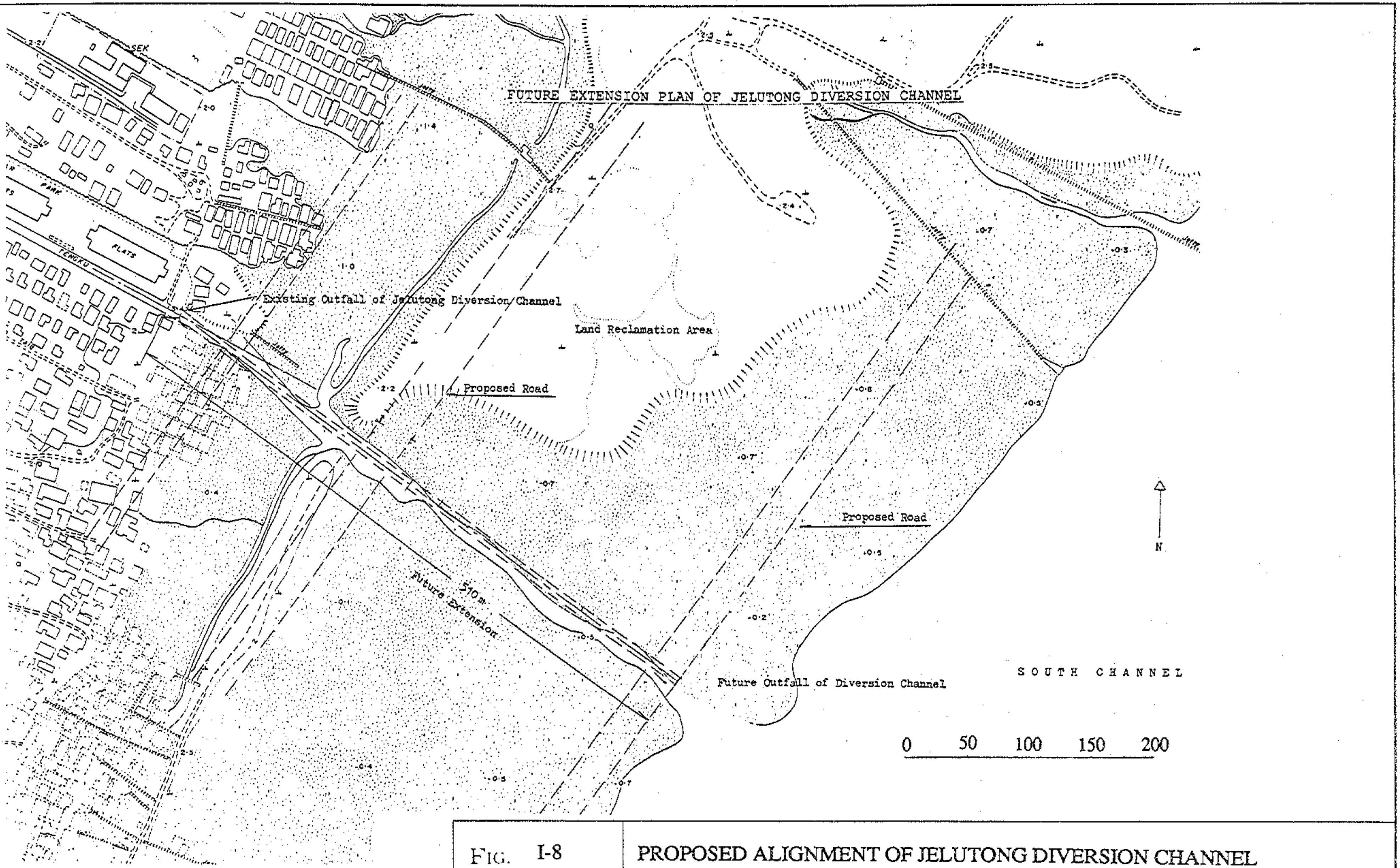

FIG. I-8 PROPOSED ALIGNMENT OF JELUTONG DIVERSION CHANNEL

THE STUDY ON FLOOD MITIGATION AND DRAINAGE IN PENANG ISLAND