$$Q = C.L.H$$

where, Q : design flood discharge (m^3/s) = 77.0 m^3/s
C : coefficient of discharge = 2.1
L : length of crest (m)
H : overflow depth (m)

$$L = \frac{Q}{C.H} = \frac{77.0}{2.1} \times H^{-1} = 36.7 \times H^{-1}$$

Overflow depth (m)	Length of Crest (m)
0.3	122.3
0.4	91.8
0.5	73.4
0.6	61.2
0.8	45.9
1.0	36.7

Considering the topography of the site and the scale of discharge, the appropriate overflow depth was established to be 0.6 m, then the length of crest to be 61.2 = 62 m.

2.1.7 Water Balance Study

The storage volume and the irrigable area were computed as a result of water balance study for the period of 1973 - 1982.

Cases	Use of Return Flow	Storage Volume (m^3)	Irrigable Area (ha)
A - 1	Yes	18,500,000	5,400
A - 2	"	15,000,000	5,000
A - 3	"	10,000,000	4,200
A - 4	"	5,000,000	2,600
B - 1	No	18,500,000	4,200
B - 2	"	15,000,000	3,900
B - 3	"	10,000,000	3,200

The breakdown calculation related to each case is attached herewith.

CASE A-1

Paddy Field Area = 5400 Ha
Regulating Volum = 18500000 M3

Month		Regula. Volum (M3)	Pumping Volum (M3/S)	El Pozo Intake Volum (M3/S)	Aguacate Intake Volum (M3/S)	Pumping Eficiency (%)
1973	1	18500000	1.213	.425	.775	22.05
1973	2	18500000	.760	.252	.576	13.82
1973	3	18500000	2.531	.327	2.333	46.01
1973	4	13459800	5.500	3.475	3.912	100.00
1973	5	11686500	5.500	5.644	.361	100.00
1973	6	18500000	4.297	1.710	0.000	100.00
1973	7	18500000	2.738	2.170	.585	78.13
1973	8	17888100	5.500	4.569	1.271	49.79
1973	9	18500000	2.951	1.289	1.485	100.00
1973	10	18500000	.673	0.000	.743	53.65
1973	11	18500000	0.000	0.000	0.000	12.23
1973	12	18500000	0.000	0.000	0.000	0.00
1974	1	18500000	1.137	.037	1.161	0.00
1974	2	18500000	1.475	0.000	1.563	20.68
1974	3	18500000	2.616	.723	1.868	26.83
1974	4	14352200	5.500	2.177	4.888	47.57
1974	5	15001900	5.500	1.185	4.062	100.00
1974	6	15590100	5.500	5.383	0.000	100.00
1974	7	12102800	5.500	5.500	1.298	100.00
1974	8	15978800	5.500	2.422	1.661	100.00
1974	9	18500000	4.314	1.010	2.415	100.00
1974	10	18500000	1.380	.373	1.354	78.44
1974	11	18500000	0.000	0.000	0.000	25.10
1974	12	18500000	0.000	0.000	0.000	0.00
1975	1	18500000	.547	.530	0.000	0.00
1975	2	18500000	4.522	4.338	.110	9.94
1975	3	5358760	5.500	5.500	4.888	82.22
1975	4	0	5.500	5.500	3.627	100.00
1975	5	10534100	5.500	1.660	.049	100.00
1975	6	10979400	5.500	5.216	0.000	100.00
1975	7	7637420	5.500	5.157	1.545	100.00
1975	8	15961800	5.500	1.287	1.025	100.00
1975	9	18500000	3.689	.724	2.055	100.00
1975	10	18500000	0.000	0.000	0.000	67.07
1975	11	18500000	0.000	0.000	0.000	0.00
1975	12	18500000	0.000	0.000	0.000	0.00
1976	1	18500000	2.296	.212	2.037	0.00
1976	2	18500000	4.636	.804	3.949	41.75
1976	3	9023680	5.500	4.710	4.352	84.29
1976	4	2852200	5.500	3.127	4.888	100.00
1976	5	6522070	5.500	4.161	0.000	100.00
1976	6	11328200	5.500	2.995	.742	100.00
1976	7	7781270	5.500	4.360	2.461	100.00
1976	8	0	5.500	3.597	4.888	100.00
1976	9	582300	5.500	2.992	2.225	100.00
1976	10	13226300	5.500	.067	.897	100.00
1976	11	18500000	1.966	0.000	0.000	100.00
1976	12	18500000	0.000	0.000	0.000	35.74
1977	1	18500000	2.354	.549	1.750	0.00
1977	2	18500000	4.419	3.671	.671	42.80
1977	3	8748260	5.500	5.500	3.562	80.35
1977	4	15541800	5.500	1.046	2.146	100.00
1977	5	18500000	2.387	.459	.904	100.00
1977	6	18500000	4.041	3.958	0.000	43.40
1977	7	18500000	4.782	3.308	1.658	73.48
1977	8	18500000	4.379	1.678	2.933	86.94
1977	9	7241780	5.500	5.309	4.523	79.61
1977	10	18500000	5.099	.992	0.000	100.00
1977	11	18500000	0.000	0.000	0.000	92.71
1977	12	18500000	0.000	0.000	0.000	0.00
						0.00

Month		Regula. Volum (M3)	Pumping Volum (M3/S)	El Pozo Intake Volum (M3/S)	Aguacate Intake Volum (M3/S)	Pumping Eficiency (%)
1978	1	18500000	2.831	1.028	1.865	51.47
1978	2	18500000	5.142	1.130	4.004	93.48
1978	3	14837600	5.500	2.062	4.888	100.00
1978	4	18500000	3.506	.961	1.231	63.75
1978	5	18500000	.293	.466	0.000	5.33
1978	6	18500000	2.249	2.034	.224	40.90
1978	7	18500000	4.499	4.560	0.000	81.80
1978	8	18500000	5.132	4.554	.639	93.30
1978	9	18500000	4.712	1.164	3.685	85.68
1978	10	18500000	1.557	.695	.886	28.30
1978	11	18500000	0.000	0.000	0.000	0.00
1978	12	18500000	0.000	0.000	0.000	0.00
1979	1	18500000	.901	.928	0.000	16.38
1979	2	18500000	3.676	.795	2.842	66.84
1979	3	18500000	4.454	1.986	2.482	80.98
1979	4	18500000	3.673	1.555	2.284	66.78
1979	5	18500000	0.000	0.000	0.000	0.00
1979	6	18500000	0.000	0.000	0.000	0.00
1979	7	18500000	1.999	1.587	.471	36.35
1979	8	18500000	4.092	1.924	2.614	74.40
1979	9	18500000	2.169	1.213	1.055	39.43
1979	10	18500000	2.988	.953	2.224	54.33
1979	11	18500000	0.000	0.000	0.000	0.00
1979	12	18500000	0.000	0.000	0.000	0.00
1980	1	18500000	.469	0.000	.495	8.53
1980	2	12866700	5.500	4.397	3.363	100.00
1980	3	13799500	5.500	1.350	3.748	100.00
1980	4	10251700	5.500	4.580	2.287	100.00
1980	5	18500000	2.857	.123	0.000	51.94
1980	6	18500000	3.874	3.939	0.000	70.44
1980	7	18500000	2.620	1.922	.792	47.64
1980	8	18500000	1.129	1.311	.030	20.53
1980	9	18500000	4.456	1.602	2.846	81.02
1980	10	18500000	.597	.672	0.000	10.86
1980	11	18500000	.336	.292	0.000	6.12
1980	12	18500000	0.000	0.000	0.000	0.00
1981	1	18500000	.373	0.000	.517	6.77
1981	2	18500000	2.289	0.000	2.530	41.62
1981	3	17328200	5.500	1.106	4.888	100.00
1981	4	18500000	4.607	.130	4.098	83.76
1981	5	18500000	0.000	0.000	0.000	0.00
1981	6	18500000	.284	.278	0.000	5.17
1981	7	18500000	1.123	1.280	0.000	20.43
1981	8	18500000	0.000	0.000	0.000	0.00
1981	9	18500000	4.174	.559	3.610	75.89
1981	10	18500000	1.203	0.000	1.280	21.88
1981	11	18500000	.302	0.000	.424	5.50
1981	12	18500000	0.000	0.000	0.000	0.00
1982	1	18585900	0.000	0.000	0.000	0.00
1982	2	18500000	1.171	.183	1.105	21.29
1982	3	12344900	5.500	4.740	2.967	100.00
1982	4	8226620	5.500	4.228	2.760	100.00
1982	5	18500000	3.507	0.000	0.000	63.76
1982	6	18500000	0.000	.114	0.000	0.00
1982	7	18500000	.204	.248	0.000	3.70
1982	8	18500000	3.570	2.002	1.609	64.90
1982	9	18500000	4.765	1.279	3.545	86.63
1982	10	18500000	1.814	.564	1.252	32.98
1982	11	18500000	0.000	0.000	0.000	0.00
1982	12	18500000	0.000	0.000	0.000	0.00

Pumping Eficiency = 45.2993806527 %
Pumping Eficiency for El Pozo = 28.9637529138 %

CASE A-2

Paddy Field Area = 5000 Ha
Regulating Volum = 15000000 M3

Month	Regula. Volum (M3)	Pumping Volum (M3/S)	El Pozo Intake Volum (M3/S)	Aguacate Intake Volum (M3/S)	Pumping Eficiency (%)
1973 1	15000000	1.156	.425	.718	21.02
1973 2	15000000	.718	.252	.533	13.06
1973 3	15000000	2.359	.327	2.160	42.90
1973 4	10709400	5.500	3.475	3.622	100.00
1973 5	9007520	5.500	5.644	.334	100.00
1973 6	15000000	3.982	1.710	0.000	72.40
1973 7	15000000	2.696	2.170	.541	49.02
1973 8	14635900	5.500	4.569	1.177	100.00
1973 9	15000000	2.747	1.289	1.375	49.94
1973 10	15000000	.619	0.000	.688	11.25
1973 11	15000000	0.000	0.000	0.000	0.00
1973 12	15000000	0.000	0.000	0.000	0.00
1974 1	15000000	1.052	.037	1.075	19.13
1974 2	15000000	1.361	0.000	1.447	24.74
1974 3	15000000	2.479	.723	1.730	45.07
1974 4	11788800	5.500	2.177	4.526	100.00
1974 5	13241800	5.500	1.165	3.762	100.00
1974 6	13825400	5.500	5.383	0.000	100.00
1974 7	10592800	5.500	5.500	1.202	100.00
1974 8	14795300	5.500	2.422	1.538	100.00
1974 9	15000000	3.243	1.010	2.236	58.96
1974 10	15000000	1.283	.373	1.254	23.33
1974 11	15000000	0.000	0.000	0.000	0.00
1974 12	15000000	0.000	0.000	0.000	0.00
1975 1	15000000	.547	.530	0.000	9.95
1975 2	15000000	4.514	4.338	.102	82.07
1975 3	2826700	5.500	5.500	4.526	100.00
1975 4	0	5.500	5.500	3.358	100.00
1975 5	10538700	5.500	1.660	.045	100.00
1975 6	10983000	5.500	5.216	0.000	100.00
1975 7	7945360	5.500	5.157	1.430	100.00
1975 8	15000000	4.950	1.287	.949	90.01
1975 9	15000000	2.559	.724	1.903	46.52
1975 10	15000000	0.000	0.000	0.000	0.00
1975 11	15000000	0.000	0.000	0.000	0.00
1975 12	15000000	0.000	0.000	0.000	0.00
1976 1	15000000	2.146	.212	1.886	39.02
1976 2	15000000	4.345	.804	3.657	78.99
1976 3	6384690	5.500	4.710	4.030	100.00
1976 4	1147000	5.500	3.127	4.526	100.00
1976 5	4813550	5.500	4.161	0.000	100.00
1976 6	9757880	5.500	2.995	.687	100.00
1976 7	6696530	5.500	4.360	2.279	100.00
1976 8	0	5.500	3.597	4.526	100.00
1976 9	1008200	5.500	2.992	2.060	100.00
1976 10	13825200	5.500	.067	.831	100.00
1976 11	15000000	.385	0.000	0.000	7.00
1976 12	15000000	0.000	0.000	0.000	0.00
1977 1	15000000	2.225	.549	1.621	40.45
1977 2	15000000	4.370	3.671	.622	79.45
1977 3	5954210	5.500	5.500	3.298	100.00
1977 4	13152400	5.500	1.046	1.987	100.00
1977 5	15000000	1.907	.459	.837	34.67
1977 6	15000000	4.042	3.958	0.000	73.49
1977 7	15000000	4.661	3.308	1.535	84.75
1977 8	15000000	4.164	1.678	2.716	75.71
1977 9	4608090	5.500	5.309	4.188	100.00
1977 10	15000000	4.777	.992	0.000	86.86
1977 11	15000000	0.000	0.000	0.000	0.00
1977 12	15000000	0.000	0.000	0.000	0.00

Month	Regula. Volum (M3)	Pumping Volum (M3/S)	El Pozo Intake Volum (M3/S)	Aguacate Intake Volum (M3/S)	Pumping Eficiency (%)
1978 1	15000000	2.693	1.028	1.727	48.97
1978 2	15000000	4.846	1.130	3.707	88.10
1978 3	12303900	5.500	2.062	4.526	100.00
1978 4	15000000	3.044	.961	1.140	55.34
1978 5	15000000	.295	.466	0.000	5.37
1978 6	15000000	2.234	2.034	.207	40.62
1978 7	15000000	4.500	4.560	0.000	81.82
1978 8	15000000	5.086	4.554	.591	92.47
1978 9	15000000	4.441	1.164	3.412	80.75
1978 10	15000000	1.492	.695	.820	27.13
1978 11	15000000	0.000	0.000	0.000	0.00
1978 12	15000000	0.000	0.000	0.000	0.00
1979 1	15000000	.902	.928	0.000	16.39
1979 2	15000000	3.466	.795	2.632	63.02
1979 3	15000000	4.271	1.986	2.298	77.66
1979 4	15000000	3.506	1.555	2.115	63.74
1979 5	15000000	0.000	0.000	0.000	0.00
1979 6	15000000	0.000	0.000	0.000	0.00
1979 7	15000000	1.966	1.587	.437	35.74
1979 8	15000000	3.902	1.924	2.421	70.95
1979 9	15000000	2.092	1.213	.977	38.03
1979 10	15000000	2.825	.953	2.059	51.37
1979 11	15000000	0.000	0.000	0.000	0.00
1979 12	15000000	0.000	0.000	0.000	0.00
1980 1	15000000	.433	0.000	.458	7.88
1980 2	9989190	5.500	4.397	3.114	100.00
1980 3	11664400	5.500	1.350	3.470	100.00
1980 4	8553280	5.500	4.580	2.118	100.00
1980 5	15000000	2.187	.123	0.000	39.76
1980 6	15000000	3.876	3.939	0.000	70.47
1980 7	15000000	2.563	1.922	.734	46.60
1980 8	15000000	1.129	1.311	.028	20.53
1980 9	15000000	4.246	1.602	2.636	77.20
1980 10	15000000	.598	.672	0.000	10.88
1980 11	15000000	.337	.292	0.000	6.12
1980 12	15000000	0.000	0.000	0.000	0.00
1981 1	15000000	.336	0.000	.479	6.10
1981 2	15000000	2.104	0.000	2.342	38.25
1981 3	14794900	5.500	1.106	4.526	100.00
1981 4	15000000	3.932	.130	3.794	71.49
1981 5	15000000	0.000	0.000	0.000	0.00
1981 6	15000000	.285	.278	0.000	5.19
1981 7	15000000	1.125	1.280	0.000	20.46
1981 8	15000000	0.000	0.000	0.000	0.00
1981 9	15000000	3.907	.559	3.343	71.04
1981 10	15000000	1.110	0.000	1.185	20.18
1981 11	15000000	.272	0.000	.393	4.95
1981 12	15000000	0.000	0.000	0.000	0.00
1982 1	15083900	0.000	0.000	0.000	0.00
1982 2	15000000	1.091	.183	1.023	19.83
1982 3	9432870	5.500	4.740	2.747	100.00
1982 4	5843590	5.500	4.228	2.555	100.00
1982 5	15000000	3.093	0.000	0.000	56.23
1982 6	15000000	0.000	.114	0.000	0.00
1982 7	15000000	.205	.248	0.000	3.72
1982 8	15000000	3.452	2.002	1.490	62.76
1982 9	15000000	4.503	1.279	3.282	81.88
1982 10	15000000	1.722	.564	1.160	31.30
1982 11	15000000	0.000	0.000	0.000	0.00
1982 12	15000000	0.000	0.000	0.000	0.00

Pumping Eficiency = 43.462302331 %
Pumping Eficiency for El Pozo = 28.9637529138 %

CASE A-3

Paddy Field Area = 4200 Ha
Regulating Volum = 10000000 M3

Month		Regula. Volum (M3)	Pumping Volum (M3/S)	El Pozo Intake Volum (M3/S)	Aguacate Intake Volum (M3/S)	Pumping Eficiency (%)
1973	1	10000000	1.042	.425	.603	18.94
1973	2	10000000	.635	.252	.448	11.55
1973	3	9999990	2.017	.327	1.814	36.67
1973	4	7208440	5.500	3.475	3.043	100.00
1973	5	5649360	5.500	5.644	.281	100.00
1973	6	10000000	3.351	1.710	0.000	60.93
1973	7	9999990	2.612	2.170	.455	47.49
1973	8	10000000	5.451	4.569	.989	99.11
1973	9	10000000	2.389	1.289	1.155	43.43
1973	10	10000000	.511	0.000	.578	9.29
1973	11	10000000	0.000	0.000	0.000	0.00
1973	12	10000000	0.000	0.000	0.000	0.00
1974	1	10000000	.882	.037	.903	16.04
1974	2	10000000	1.132	0.000	1.216	20.57
1974	3	10000000	2.203	.723	1.453	40.06
1974	4	8661890	5.500	2.177	3.802	100.00
1974	5	9999990	4.857	1.185	3.160	88.31
1974	6	10000000	5.278	5.383	0.000	95.97
1974	7	7276820	5.500	5.500	1.009	100.00
1974	8	10000000	4.704	2.422	1.292	85.52
1974	9	10000000	2.809	1.010	1.878	51.07
1974	10	9999990	1.088	.373	1.053	19.78
1974	11	10000000	0.000	0.000	0.000	0.00
1974	12	10000000	0.000	0.000	0.000	0.00
1975	1	10000000	.548	.530	0.000	9.97
1975	2	10000000	4.498	4.338	.085	81.78
1975	3	0	5.500	5.500	3.802	100.00
1975	4	0	5.500	5.500	2.821	100.00
1975	5	10000000	5.296	1.660	.833	96.28
1975	6	9939990	5.329	5.216	0.000	96.90
1975	7	7571090	5.500	5.157	1.201	100.00
1975	8	10000000	3.073	1.287	.797	55.86
1975	9	9999990	2.257	.724	1.598	41.03
1975	10	10000000	0.000	0.000	0.000	0.00
1975	11	10000000	0.000	0.000	0.000	0.00
1975	12	10000000	0.000	0.000	0.000	0.00
1976	1	9999990	1.845	.212	1.584	33.54
1976	2	10000000	3.762	.804	3.072	68.40
1976	3	3106700	5.500	4.710	3.385	100.00
1976	4	0	5.500	3.127	3.802	100.00
1976	5	3659900	5.500	4.161	0.000	100.00
1976	6	8880710	5.500	2.995	.577	100.00
1976	7	6790450	5.500	4.360	1.914	100.00
1976	8	1812200	5.500	3.597	3.802	100.00
1976	9	3672100	5.500	2.992	1.730	100.00
1976	10	9999990	2.948	.067	.698	53.60
1976	11	10000000	0.000	0.000	0.000	0.00
1976	12	10000000	0.000	0.000	0.000	0.00
1977	1	9999990	1.966	.549	1.361	35.74
1977	2	10000000	4.271	3.671	.522	77.65
1977	3	2366100	5.500	5.500	2.771	100.00
1977	4	10000000	5.356	1.046	1.669	97.38
1977	5	10000000	1.086	.459	.703	19.75
1977	6	9999990	4.043	3.958	0.000	73.51
1977	7	10000000	4.420	3.308	1.289	80.36
1977	8	9999990	3.734	1.678	2.282	67.89
1977	9	1340700	5.500	5.309	3.518	100.00
1977	10	10000000	4.133	.992	0.000	75.14
1977	11	10000000	0.000	0.000	0.000	0.00
1977	12	10000000	0.000	0.000	0.000	0.00

Month	Regula. Volum (M3)	Pumping Volum (M3/S)	El Pozo (ntake Volum (M3/S)	Aguacate Intake Volum (M3/S)	Pumping Eficiency (%)
1978 1	10000000	2.419	1.028	1.450	43.98
1978 2	9999990	4.254	1.130	3.114	77.34
1978 3	9236510	5.500	2.062	3.802	100.00
1978 4	10000000	2.119	.961	.958	38.52
1978 5	10000000	.300	.466	0.000	5.45
1978 6	10000000	2.203	2.034	.174	40.05
1978 7	9999990	4.503	4.560	0.000	81.88
1978 8	10000000	4.994	4.554	.497	90.80
1978 9	10000000	3.899	1.164	2.866	70.88
1978 10	10000000	1.363	.695	.689	24.77
1978 11	10000000	0.000	0.000	0.000	0.00
1978 12	10000000	0.000	0.000	0.000	0.00
1979 1	10000000	.903	.928	0.000	16.42
1979 2	10000000	3.046	.795	2.211	55.38
1979 3	10000000	3.905	1.986	1.931	71.00
1979 4	10000000	3.171	1.555	1.777	57.66
1979 5	10000000	0.000	0.000	0.000	0.00
1979 6	10000000	0.000	0.000	0.000	0.00
1979 7	10000000	1.899	1.587	.367	34.52
1979 8	10000000	3.522	1.924	2.033	64.04
1979 9	10000000	1.938	1.213	.821	35.24
1979 10	10000000	2.500	.953	1.730	45.45
1979 11	10000000	0.000	0.000	0.000	0.00
1979 12	10000000	0.000	0.000	0.000	0.00
1980 1	10000000	.361	0.000	.385	6.57
1980 2	6234060	5.500	4.397	2.616	100.00
1980 3	9393930	5.500	1.350	2.915	100.00
1980 4	7156140	5.500	4.580	1.779	100.00
1980 5	10000000	.848	.123	0.000	15.42
1980 6	10000000	3.879	3.939	0.000	70.52
1980 7	10000000	2.449	1.922	.616	44.53
1980 8	9999990	1.129	1.311	.024	20.53
1980 9	10000000	3.826	1.602	2.214	69.56
1980 10	10000000	.601	.672	0.000	10.92
1980 11	10000000	.337	.292	0.000	6.13
1980 12	10000000	0.000	0.000	0.000	0.00
1981 1	10000000	.262	0.000	.402	4.76
1981 2	9999990	1.733	0.000	1.968	31.51
1981 3	10000000	4.855	1.106	3.802	88.27
1981 4	9999990	3.248	.130	3.187	59.06
1981 5	10000000	0.000	0.000	0.000	0.00
1981 6	10000000	.288	.278	0.000	5.23
1981 7	9999990	1.130	1.280	0.000	20.54
1981 8	10000000	0.000	0.000	0.000	0.00
1981 9	10000000	3.374	.559	2.808	61.35
1981 10	10000000	.922	0.000	.995	16.77
1981 11	10000000	.212	0.000	.330	3.85
1981 12	10000000	0.000	0.000	0.000	0.00
1982 1	10080000	0.000	0.000	0.000	0.00
1982 2	10000000	.931	.183	.860	16.93
1982 3	5608890	5.500	4.740	2.307	100.00
1982 4	3077600	5.500	4.228	2.146	100.00
1982 5	9999990	2.265	0.000	0.000	41.18
1982 6	10000000	0.000	.114	0.000	0.00
1982 7	10000000	.208	.248	0.000	3.77
1982 8	9999990	3.216	2.002	1.251	58.46
1982 9	10000000	3.981	1.279	2.757	72.37
1982 10	10000000	1.538	.564	.974	27.96
1982 11	10000000	0.000	0.000	0.000	0.00
1982 12	10000000	0.000	0.000	0.000	0.00

Pumping Eficiency = 40.0137254079 %
Pumping Eficiency for El Pozo = 28.9637529138 %

CASE A-4

Paddy Field Area = 2600 Ha
Regulating Volum = 5000000 M3

Month	Regula. Volum (M3)	Pumping Volum (M3/S)	El Pozo Intake Volum (M3/S)	Aguacate Intake Volum (M3/S)	Pumping Eficiency (%)
1973 1	5000000	.814	.425	.373	14.80
1973 2	5000000	.469	.252	.277	8.52
1973 3	5000010	1.332	.327	1.123	24.22
1973 4	5000000	5.420	3.475	1.884	98.55
1973 5	3726500	5.500	5.644	.174	100.00
1973 6	4999990	2.169	1.710	0.000	39.44
1973 7	5000010	2.443	2.170	.281	44.43
1973 8	5000000	5.081	4.569	.612	92.38
1973 9	4999990	1.954	1.289	.715	35.52
1973 10	5000000	.295	0.000	.358	5.37
1973 11	5000000	0.000	0.000	0.000	0.00
1973 12	5000000	0.000	0.000	0.000	0.00
1974 1	5000000	.542	.037	.559	9.85
1974 2	5000000	.673	0.000	.753	12.24
1974 3	5000000	1.652	.723	.899	30.04
1974 4	5000010	4.571	2.177	2.353	83.11
1974 5	5000000	3.158	1.185	1.956	57.42
1974 6	5000010	5.285	5.383	0.000	96.10
1974 7	3295700	5.500	5.500	.625	100.00
1974 8	5000000	3.836	2.422	.800	69.74
1974 9	5000010	2.099	1.010	1.163	38.16
1974 10	5000000	.698	.373	.652	12.69
1974 11	5000000	0.000	0.000	0.000	0.00
1974 12	5000000	0.000	0.000	0.000	0.00
1975 1	5000000	.550	.530	0.000	10.00
1975 2	4999990	4.466	4.338	.053	81.20
1975 3	0	5.500	5.500	2.353	100.00
1975 4	0	5.500	5.500	1.746	100.00
1975 5	5000000	3.422	1.660	.023	62.22
1975 6	5000000	5.331	5.216	0.000	96.92
1975 7	3788500	5.500	5.157	.744	100.00
1975 8	4999990	2.316	1.287	.493	42.11
1975 9	5000010	1.653	.724	.989	30.05
1975 10	5000000	0.000	0.000	0.000	0.00
1975 11	5000000	0.000	0.000	0.000	0.00
1975 12	5000000	0.000	0.000	0.000	0.00
1976 1	5000010	1.242	.212	.981	22.58
1976 2	5000000	2.597	.804	1.902	47.22
1976 3	1550700	5.500	4.710	2.096	100.00
1976 4	1916300	5.500	3.127	2.353	100.00
1976 5	4999990	5.290	4.161	0.000	96.18
1976 6	5000010	3.272	2.995	.357	59.50
1976 7	4851970	5.500	4.360	1.185	100.00
1976 8	3739700	5.500	3.597	2.353	100.00
1976 9	4999990	4.611	2.992	1.071	83.84
1976 10	5000000	.327	.067	.432	5.95
1976 11	5000000	0.000	0.000	0.000	0.00
1976 12	5000000	0.000	0.000	0.000	0.00
1977 1	5000000	1.448	.549	.843	26.32
1977 2	4999990	4.073	3.671	.323	74.05
1977 3	189800	5.500	5.500	1.715	100.00
1977 4	5000000	3.642	1.046	1.033	66.22
1977 5	5000000	.825	.459	.435	14.99
1977 6	5000000	4.045	3.958	0.000	73.55
1977 7	4999990	3.938	3.308	.798	71.59
1977 8	5000000	2.874	1.678	1.412	52.26
1977 9	0	5.500	5.309	2.178	100.00
1977 10	5000010	2.772	.992	0.000	50.40
1977 11	5000000	0.000	0.000	0.000	0.00
1977 12	5000000	0.000	0.000	0.000	0.00

Month	Regula. Volum (M3)	Pumping Volum (M3/S)	El Pozo Intake Volum (M3/S)	Aguacate Intake Volum (M3/S)	Pumping Eficiency (%)
1978 1	5000000	1.870	1.028	.898	34.00
1978 2	5000010	3.070	1.130	1.928	55.81
1978 3	5000010	4.342	2.062	2.353	78.94
1978 4	5000000	1.465	.961	.593	26.65
1978 5	5000000	.308	.466	0.000	5.60
1978 6	5000000	2.141	2.034	.108	38.93
1978 7	5000000	4.509	4.560	0.000	81.98
1978 8	5000000	4.810	4.554	.307	87.45
1978 9	5000000	2.813	1.164	1.774	51.15
1978 10	5000010	1.104	.695	.426	20.07
1978 11	5000000	0.000	0.000	0.000	0.00
1978 12	5000000	0.000	0.000	0.000	0.00
1979 1	5000000	.906	.928	0.000	16.47
1979 2	4999990	2.205	.795	1.368	40.10
1979 3	5000000	3.173	1.986	1.195	57.69
1979 4	5000010	2.502	1.555	1.100	45.50
1979 5	5000000	0.000	0.000	0.000	0.00
1979 6	5000000	0.000	0.000	0.000	0.00
1979 7	5000010	1.765	1.587	.227	32.09
1979 8	5000010	2.762	1.924	1.259	50.23
1979 9	5000010	1.632	1.213	.508	29.67
1979 10	4999990	1.848	.953	1.071	33.61
1979 11	5000000	0.000	0.000	0.000	0.00
1979 12	5000000	0.000	0.000	0.000	0.00
1980 1	5000000	.217	0.000	.238	3.95
1980 2	3723800	5.500	4.397	1.619	100.00
1980 3	5000000	3.688	1.350	1.804	67.06
1980 4	4508860	5.500	4.580	1.101	100.00
1980 5	5000000	0.000	.123	0.000	0.00
1980 6	5000000	3.885	3.939	0.000	70.63
1980 7	4999990	2.221	1.922	.382	40.38
1980 8	4999990	1.129	1.311	.015	20.53
1980 9	5000010	2.986	1.602	1.370	54.29
1980 10	5000000	.605	.672	0.000	11.01
1980 11	5000000	.339	.292	0.000	6.16
1980 12	5000000	0.000	0.000	0.000	0.00
1981 1	5000000	.114	0.000	.249	2.08
1981 2	5000000	.992	0.000	1.218	18.04
1981 3	5000010	3.411	1.106	2.353	62.02
1981 4	4999990	2.040	.130	1.973	37.09
1981 5	5000000	0.000	0.000	0.000	0.00
1981 6	5000000	.292	.278	0.000	5.30
1981 7	4999990	1.138	1.280	0.000	20.68
1981 8	5000000	0.000	0.000	0.000	0.00
1981 9	4999990	2.308	.559	1.738	41.96
1981 10	5000000	.548	0.000	.616	9.97
1981 11	5000000	.091	0.000	.204	1.66
1981 12	5000000	0.000	0.000	0.000	0.00
1982 1	5072300	0.000	0.000	0.000	0.00
1982 2	5000000	.611	.183	.532	11.11
1982 3	2960900	5.500	4.740	1.428	100.00
1982 4	2545600	5.500	4.228	1.329	100.00
1982 5	5000000	.609	0.000	0.000	11.07
1982 6	5000000	0.000	.114	0.000	0.00
1982 7	5000000	.213	.248	0.000	3.87
1982 8	4999990	2.744	2.002	.775	49.88
1982 9	5000000	2.935	1.279	1.707	53.37
1982 10	5000000	1.169	.564	.603	21.26
1982 11	5000000	0.000	0.000	0.000	0.00
1982 12	5000000	0.000	0.000	0.000	0.00

Pumping Eficiency = 33.5653516667 %
Pumping Eficiency for El Pozo = 28.9637529138 %

CASE B-1

Paddy Field Area = 4200 Ha
Regulating Volum = 18500000 M3

Month	Regula. Volum (M3)	Pumping Volum (M3/S)	El Pozo Intake Volum (M3/S)	Aguacate Intake Volum (M3/S)	Pumping Eficiency (%)
1973 1	18500000	1.853	.425	1.414	33.70
1973 2	18500000	.885	.252	.698	16.09
1973 3	18500000	2.477	.327	2.275	45.04
1973 4	15932800	5.500	3.475	2.956	100.00
1973 5	15125900	5.500	5.644	0.000	100.00
1973 6	18500000	2.974	1.710	0.000	54.08
1973 7	18500000	2.475	2.170	.318	45.00
1973 8	18178500	5.500	4.569	1.158	100.00
1973 9	18500000	2.881	1.289	1.523	52.38
1973 10	18500000	.827	0.000	.894	15.03
1973 11	18500000	0.000	0.000	0.000	0.00
1973 12	18500000	0.000	0.000	0.000	0.00
1974 1	18500000	1.789	.037	1.811	32.53
1974 2	18500000	1.627	0.000	1.711	29.57
1974 3	18500000	2.548	.723	1.798	46.33
1974 4	16205200	5.500	2.177	4.171	100.00
1974 5	18500000	5.009	1.185	2.955	91.08
1974 6	18500000	5.278	5.383	0.000	95.97
1974 7	15670100	5.500	5.500	1.049	100.00
1974 8	18500000	5.010	2.422	1.558	91.08
1974 9	18500000	3.408	1.010	2.478	61.97
1974 10	18500000	1.556	.373	1.521	28.28
1974 11	18500000	0.000	0.000	0.000	0.00
1974 12	18500000	0.000	0.000	0.000	0.00
1975 1	18500000	.548	.530	0.000	9.97
1975 2	18500000	4.632	4.338	.220	84.23
1975 3	4597000	5.500	5.500	5.170	100.00
1975 4	0	5.500	5.500	2.663	100.00
1975 5	10649100	5.500	1.660	0.000	100.00
1975 6	11091400	5.500	5.216	0.000	100.00
1975 7	8391740	5.500	5.157	1.302	100.00
1975 8	17033100	5.500	1.287	.905	100.00
1975 9	18500000	3.333	.724	2.108	60.59
1975 10	18500000	0.000	0.000	0.000	0.00
1975 11	18500000	0.000	0.000	0.000	0.00
1975 12	18500000	0.000	0.000	0.000	0.00
1976 1	18500000	2.969	.212	2.709	53.98
1976 2	18500000	4.849	.804	4.159	88.17
1976 3	9031170	5.500	4.710	4.347	100.00
1976 4	2113900	5.500	3.127	5.170	100.00
1976 5	5773820	5.500	4.161	0.000	100.00
1976 6	11221700	5.500	2.995	.489	100.00
1976 7	8251270	5.500	4.360	2.243	100.00
1976 8	0	5.500	3.597	5.170	100.00
1976 9	428900	5.500	2.992	2.282	100.00
1976 10	12643400	5.500	.067	1.052	100.00
1976 11	18500000	2.194	0.000	0.000	39.88
1976 12	18500000	0.000	0.000	0.000	0.00
1977 1	18500000	3.019	.549	2.415	54.89
1977 2	18500000	4.545	3.671	.796	82.63
1977 3	8815660	5.500	5.500	3.536	100.00
1977 4	18183300	5.500	1.046	1.144	100.00
1977 5	18500000	.501	.459	0.000	9.11
1977 6	18500000	4.043	3.958	0.000	73.51
1977 7	18500000	4.549	3.308	1.419	82.71
1977 8	18500000	4.316	1.678	2.863	78.47
1977 9	6931560	5.500	5.309	4.640	100.00
1977 10	18500000	5.219	.992	0.000	94.89
1977 11	18500000	0.000	0.000	0.000	0.00
1977 12	18500000	0.000	0.000	0.000	0.00

Month	Regula. Volum (M3)	Pumping Volum (M3/S)	El Pozo Intake Volum (M3/S)	Aguacate Intake Volum (M3/S)	Pumping Eficiency (%)
1978 1	18500000	3.501	1.028	2.532	63.65
1978 2	18500000	5.354	1.130	4.215	97.35
1978 3	14070900	5.500	2.062	5.170	100.00
1978 4	18500000	2.781	.961	.206	50.56
1978 5	18500000	.300	.466	0.000	5.45
1978 6	18500000	2.029	2.034	0.000	36.89
1978 7	18500000	4.503	4.560	0.000	81.88
1978 8	18500000	5.006	4.554	.509	91.02
1978 9	18500000	4.813	1.164	3.781	87.51
1978 10	18500000	1.714	.695	1.040	31.16
1978 11	18500000	0.000	0.000	0.000	0.00
1978 12	18500000	0.000	0.000	0.000	0.00
1979 1	18500000	1.426	.928	.523	25.92
1979 2	18500000	3.858	.795	3.023	70.15
1979 3	18500000	4.403	1.986	2.428	80.05
1979 4	18500000	2.681	1.555	1.286	48.74
1979 5	18500000	0.000	0.000	0.000	0.00
1979 6	18500000	0.000	0.000	0.000	0.00
1979 7	18500000	1.734	1.587	.201	31.52
1979 8	18500000	4.025	1.924	2.536	73.18
1979 9	18500000	2.200	1.213	1.083	40.00
1979 10	18500000	3.183	.953	2.413	57.87
1979 11	18500000	0.000	0.000	0.000	0.00
1979 12	18500000	0.000	0.000	0.000	0.00
1980 1	18500000	1.103	0.000	1.127	20.06
1980 2	12374600	5.500	4.397	3.557	100.00
1980 3	13360700	5.500	1.350	3.727	100.00
1980 4	12392500	5.500	4.580	1.289	100.00
1980 5	18500000	2.067	.123	0.000	37.58
1980 6	18500000	3.879	3.939	0.000	70.52
1980 7	18500000	2.364	1.922	.531	42.97
1980 8	18500000	1.106	1.311	0.000	20.11
1980 9	18500000	4.532	1.602	2.920	82.41
1980 10	18500000	.601	.672	0.000	10.92
1980 11	18500000	.337	.292	0.000	6.13
1980 12	18500000	0.000	0.000	0.000	0.00
1981 1	18500000	1.009	0.000	1.150	18.35
1981 2	18500000	2.468	0.000	2.703	44.87
1981 3	16958300	5.500	1.106	5.023	100.00
1981 4	18500000	3.803	.130	3.146	69.14
1981 5	18500000	0.000	0.000	0.000	0.00
1981 6	18500000	.288	.278	0.000	5.23
1981 7	18500000	1.130	1.280	0.000	20.54
1981 8	18500000	0.000	0.000	0.000	0.00
1981 9	18500000	4.270	.559	3.704	77.64
1981 10	18500000	1.372	0.000	1.444	24.94
1981 11	18500000	.902	0.000	1.020	16.39
1981 12	18500000	0.000	0.000	0.000	0.00
1982 1	18500000	0.000	0.000	0.000	0.00
1982 2	18500000	1.312	.183	1.241	23.86
1982 3	12454200	5.500	4.740	2.925	100.00
1982 4	10888500	5.500	4.228	1.774	100.00
1982 5	18500000	2.522	0.000	0.000	45.85
1982 6	18500000	0.000	.114	0.000	0.00
1982 7	18500000	.208	.248	0.000	3.77
1982 8	18500000	3.469	2.082	1.504	63.06
1982 9	18500000	4.860	1.279	3.637	88.37
1982 10	18500000	1.980	.564	1.416	36.00
1982 11	18500000	0.000	0.000	0.000	0.00
1982 12	18500000	0.000	0.000	0.000	0.00

Pumping Eficiency = 44.7996318531 %
Pumping Eficiency for El Pozo = 28.9637529138 %

CASE B-2

Paddy Field Area = 3900 Ha
Regulating Volum = 15000000 M3

Month		Regula. Volum (M3)	Pumping Volum (M3/S)	El Pozo Intake Volum (M3/S)	Aguacate Intake Volum (M3/S)	Pumping Eficiency (%)
1973	1	15000000	1.753	.425	1.313	31.87
1973	2	15000000	.836	.252	.648	15.20
1973	3	15000000	2.316	.327	2.112	42.11
1973	4	12979000	5.500	3.475	2.745	100.00
1973	5	12172000	5.500	5.644	0.000	100.00
1973	6	15000000	2.765	1.710	0.000	50.27
1973	7	15000000	2.453	2.170	.295	44.60
1973	8	14896800	5.500	4.569	1.075	100.00
1973	9	15000000	2.689	1.289	1.414	48.89
1973	10	15000000	.764	0.000	.830	13.89
1973	11	15000000	0.000	0.000	0.000	0.00
1973	12	15000000	0.000	0.000	0.000	0.00
1974	1	15000000	1.661	.037	1.681	30.19
1974	2	15000000	1.505	0.000	1.589	27.37
1974	3	15000000	2.420	.723	1.669	44.01
1974	4	13475900	5.500	2.177	3.873	100.00
1974	5	15000000	4.511	1.185	2.744	82.02
1974	6	15000000	5.280	5.383	0.000	95.99
1974	7	12368700	5.500	5.500	.974	100.00
1974	8	15000000	4.825	2.422	1.447	87.73
1974	9	15000000	3.232	1.010	2.301	58.77
1974	10	15000000	1.449	.373	1.412	26.35
1974	11	15000000	0.000	0.000	0.000	0.00
1974	12	15000000	0.000	0.000	0.000	0.00
1975	1	15000000	.548	.530	0.000	9.97
1975	2	15000000	4.617	4.338	.204	83.94
1975	3	2084800	5.500	5.500	4.801	100.00
1975	4	0	5.500	5.500	2.473	100.00
1975	5	10645300	5.500	1.660	0.000	100.00
1975	6	11086900	5.500	5.216	0.000	100.00
1975	7	8634830	5.500	5.157	1.209	100.00
1975	8	15000000	4.586	1.287	.841	83.38
1975	9	15000000	2.617	.724	1.958	47.58
1975	10	15000000	0.000	0.000	0.000	0.00
1975	11	15000000	0.000	0.000	0.000	0.00
1975	12	15000000	0.000	0.000	0.000	0.00
1976	1	15000000	2.776	.212	2.515	50.47
1976	2	15000000	4.553	.804	3.862	82.79
1976	3	6360860	5.500	4.710	4.036	100.00
1976	4	397400	5.500	3.127	4.801	100.00
1976	5	4054800	5.500	4.161	0.000	100.00
1976	6	9590160	5.500	2.995	.455	100.00
1976	7	7046750	5.500	4.360	2.083	100.00
1976	8	0	5.500	3.597	4.801	100.00
1976	9	850500	5.500	2.992	2.119	100.00
1976	10	13262500	5.500	.067	.977	100.00
1976	11	15000000	.605	0.000	0.000	11.00
1976	12	15000000	0.000	0.000	0.000	0.00
1977	1	15000000	2.847	.549	2.242	51.76
1977	2	15000000	4.488	3.671	.739	81.60
1977	3	5991540	5.500	5.500	3.284	100.00
1977	4	15000000	5.282	1.046	1.063	96.03
1977	5	15000000	.384	.459	0.000	6.98
1977	6	15000000	4.043	3.958	0.000	73.52
1977	7	15000000	4.449	3.308	1.317	80.90
1977	8	15000000	4.113	1.678	2.659	74.78
1977	9	4289080	5.500	5.309	4.308	100.00
1977	10	15000000	4.900	.992	0.000	89.09
1977	11	15000000	0.000	0.000	0.000	0.00
1977	12	15000000	0.000	0.000	0.000	0.00

Month	Regula. Volum (M3)	Pumping Volum (M3/S)	El Pozo Intake Volum (M3/S)	Aguacate Intake Volum (M3/S)	Pumping Eficiency (%)
1978 1	15000000	3.321	1.028	2.351	60.37
1978 2	15000000	5.054	1.130	3.914	91.88
1978 3	11557500	5.500	2.062	4.801	100.00
1978 4	15000000	2.387	.961	.191	43.40
1978 5	15000000	.301	.466	0.000	5.48
1978 6	15000000	2.030	2.034	0.000	36.91
1978 7	15000000	4.504	4.560	0.000	81.90
1978 8	15000000	4.971	4.554	.472	90.37
1978 9	15000000	4.544	1.164	3.511	82.62
1978 10	15000000	1.640	.695	.966	29.82
1978 11	15000000	0.000	0.000	0.000	0.00
1978 12	15000000	0.000	0.000	0.000	0.00
1979 1	15000000	1.389	.928	.485	25.25
1979 2	15000000	3.643	.795	2.807	66.23
1979 3	15000000	4.230	1.986	2.255	76.91
1979 4	15000000	2.590	1.555	1.194	47.10
1979 5	15000000	0.000	0.000	0.000	0.00
1979 6	15000000	0.000	0.000	0.000	0.00
1979 7	15000000	1.720	1.587	.187	31.28
1979 8	15000000	3.846	1.924	2.355	69.93
1979 9	15000000	2.124	1.213	1.005	38.62
1979 10	15000000	3.012	.953	2.241	54.77
1979 11	15000000	0.000	0.000	0.000	0.00
1979 12	15000000	0.000	0.000	0.000	0.00
1980 1	15000000	1.023	0.000	1.046	18.60
1980 2	9509970	5.500	4.397	3.303	100.00
1980 3	11208100	5.500	1.350	3.460	100.00
1980 4	10476700	5.500	4.580	1.197	100.00
1980 5	15000000	1.478	.123	0.000	26.87
1980 6	15000000	3.880	3.939	0.000	70.54
1980 7	15000000	2.327	1.922	.493	42.31
1980 8	15000000	1.108	1.311	0.000	20.14
1980 9	15000000	4.324	1.602	2.712	78.63
1980 10	15000000	.602	.672	0.000	10.94
1980 11	15000000	.338	.292	0.000	6.14
1980 12	15000000	0.000	0.000	0.000	0.00
1981 1	15000000	.928	0.000	1.068	16.88
1981 2	15000000	2.277	0.000	2.510	41.39
1981 3	14416900	5.500	1.106	4.664	100.00
1981 4	15000000	3.209	.130	2.922	58.35
1981 5	15000000	0.000	0.000	0.000	0.00
1981 6	15000000	.288	.278	0.000	5.24
1981 7	15000000	1.131	1.280	0.000	20.56
1981 8	15000000	0.000	0.000	0.000	0.00
1981 9	15000000	4.006	.559	3.439	72.84
1981 10	15000000	1.269	0.000	1.341	23.08
1981 11	15000000	.830	0.000	.947	15.09
1981 12	15000000	0.000	0.000	0.000	0.00
1982 1	15078600	0.000	0.000	0.000	0.00
1982 2	15000000	1.225	.183	1.152	22.28
1982 3	9513430	5.500	4.740	2.716	100.00
1982 4	8275470	5.500	4.228	1.647	100.00
1982 5	15000000	2.193	0.000	0.000	39.87
1982 6	15000000	0.000	.114	0.000	0.00
1982 7	15000000	.209	.248	0.000	3.79
1982 8	15000000	3.362	2.002	1.397	61.13
1982 9	15000000	4.601	1.279	3.377	83.66
1982 10	15000000	1.879	.564	1.315	34.17
1982 11	15000000	0.000	0.000	0.000	0.00
1982 12	15000000	0.000	0.000	0.000	0.00

Pumping Eficiency = 43.0501687296 %
Pumping Eficiency for El Pozo = 28.9637529138 %

CASE B-3

Paddy Field Area = 3200 Ha
Regulating Volum = 10000000 M3

Month	Regula. Volum (M3)	Pumping Volum (M3/S)	El Pozo Intake Volum (M3/S)	Aguacate Intake Volum (M3/S)	Pumping Eficiency (%)
1973 1	10000000	1.518	.425	1.077	27.60
1973 2	10000000	.722	.252	.532	13.12
1973 3	10000000	1.940	.327	1.733	35.27
1973 4	9253240	5.500	3.475	2.252	100.00
1973 5	8445780	5.500	5.644	0.000	100.00
1973 6	10000000	2.276	1.710	0.000	41.38
1973 7	9999990	2.402	2.170	.242	43.68
1973 8	10000000	5.348	4.569	.882	97.24
1973 9	10000000	2.397	1.289	1.160	43.59
1973 10	10000000	.617	0.000	.681	11.22
1973 11	10000000	0.000	0.000	0.000	0.00
1973 12	10000000	0.000	0.000	0.000	0.00
1974 1	10000000	1.361	.037	1.380	24.74
1974 2	10000000	1.222	0.000	1.304	22.22
1974 3	10000000	2.122	.723	1.370	38.58
1974 4	10000000	5.394	2.177	3.178	98.08
1974 5	10000000	3.452	1.185	2.251	62.75
1974 6	10000000	5.283	5.383	0.000	96.05
1974 7	7832270	5.500	5.500	.799	100.00
1974 8	9999990	4.394	2.422	1.187	79.90
1974 9	10000000	2.822	1.010	1.888	51.31
1974 10	9999990	1.201	.373	1.159	21.83
1974 11	10000000	0.000	0.000	0.000	0.00
1974 12	10000000	0.000	0.000	0.000	0.00
1975 1	10000000	.549	.530	0.000	9.99
1975 2	10000000	4.581	4.338	.167	83.28
1975 3	0	5.500	5.500	3.939	100.00
1975 4	0	5.500	5.500	2.029	100.00
1975 5	10000000	5.262	1.660	0.000	95.68
1975 6	10000000	5.330	5.216	0.000	96.91
1975 7	8125700	5.500	5.157	.992	100.00
1975 8	10000000	2.759	1.287	.690	50.17
1975 9	10000000	2.268	.724	1.606	41.23
1975 10	10000000	0.000	0.000	0.000	0.00
1975 11	10000000	0.000	0.000	0.000	0.00
1975 12	10000000	0.000	0.000	0.000	0.00
1976 1	10000000	2.325	.212	2.064	42.27
1976 2	9999990	3.862	.804	3.169	70.23
1976 3	3296900	5.500	4.710	3.312	100.00
1976 4	0	5.500	3.127	3.939	100.00
1976 5	3651600	5.500	4.161	0.000	100.00
1976 6	9391030	5.500	2.995	.373	100.00
1976 7	7844050	5.500	4.360	1.709	100.00
1976 8	2489200	5.500	3.597	3.939	100.00
1976 9	4323480	5.500	2.992	1.739	100.00
1976 10	10000000	2.813	.067	.801	51.15
1976 11	10000000	0.000	0.000	0.000	0.00
1976 12	10000000	0.000	0.000	0.000	0.00
1977 1	10000000	2.445	.549	1.840	44.45
1977 2	10000000	4.355	3.671	.606	79.19
1977 3	2568700	5.500	5.500	2.694	100.00
1977 4	10000000	4.488	1.046	.872	81.59
1977 5	10000000	.387	.459	0.000	7.03
1977 6	10000000	4.044	3.958	0.000	73.53
1977 7	9999990	4.217	3.308	1.081	76.67
1977 8	10000000	3.640	1.678	2.181	66.18
1977 9	1290000	5.500	5.309	3.535	100.00
1977 10	10000000	4.155	.992	0.000	75.55
1977 11	10000000	0.000	0.000	0.000	0.00
1977 12	10000000	0.000	0.000	0.000	0.00

Month	Regula. Volum (M3)	Pumping Volum (M3/S)	El Pozo Intake Volum (M3/S)	Aguacate Intake Volum (M3/S)	Pumping Eficiency (%)
1978 1	10000000	2.900	1.028	1.929	52.73
1978 2	9999990	4.352	1.130	3.211	79.13
1978 3	8859410	5.500	2.062	3.939	100.00
1978 4	10000000	1.467	.961	.157	26.68
1978 5	10000000	.305	.466	0.000	5.54
1978 6	10000000	2.032	2.034	0.000	36.94
1978 7	10000000	4.507	4.560	0.000	81.94
1978 8	10000000	4.888	4.554	.388	88.87
1978 9	10000000	3.917	1.164	2.880	71.22
1978 10	10000000	1.468	.695	.793	26.70
1978 11	10000000	0.000	0.000	0.000	0.00
1978 12	10000000	0.000	0.000	0.000	0.00
1979 1	9999990	1.303	.928	.398	23.69
1979 2	10000000	3.140	.795	2.303	57.08
1979 3	9999990	3.827	1.986	1.850	69.58
1979 4	10000000	2.379	1.555	.980	43.26
1979 5	10000000	0.000	0.000	0.000	0.00
1979 6	10000000	0.000	0.000	0.000	0.00
1979 7	10000000	1.689	1.587	.154	30.71
1979 8	9999990	3.430	1.924	1.932	62.37
1979 9	10000000	1.946	1.213	.825	35.39
1979 10	10000000	2.613	.953	1.838	47.51
1979 11	10000000	0.000	0.000	0.000	0.00
1979 12	10000000	0.000	0.000	0.000	0.00
1980 1	10000000	.837	0.000	.858	15.21
1980 2	5992480	5.500	4.397	2.710	100.00
1980 3	9352000	5.500	1.350	2.839	100.00
1980 4	9173180	5.500	4.580	.982	100.00
1980 5	10000000	.103	.123	0.000	1.87
1980 6	10000000	3.882	3.939	0.000	70.59
1980 7	10000000	2.241	1.922	.404	40.75
1980 8	10000000	1.112	1.311	0.000	20.21
1980 9	10000000	3.839	1.602	2.225	69.80
1980 10	10000000	.604	.672	0.000	10.97
1980 11	10000000	.338	.292	0.000	6.15
1980 12	10000000	0.000	0.000	0.000	0.00
1981 1	10000000	.739	0.000	.876	13.44
1981 2	10000000	1.830	0.000	2.059	33.27
1981 3	10000000	4.883	1.106	3.827	88.77
1981 4	10000000	2.462	.130	2.397	44.77
1981 5	10000000	0.000	0.000	0.000	0.00
1981 6	10000000	.290	.278	0.000	5.28
1981 7	10000000	1.135	1.280	0.000	20.63
1981 8	10000000	0.000	0.000	0.000	0.00
1981 9	10000000	3.390	.559	2.822	61.64
1981 10	10000000	1.031	0.000	1.100	18.74
1981 11	10000000	.662	0.000	.777	12.04
1981 12	10000000	0.000	0.000	0.000	0.00
1982 1	10075200	0.000	0.000	0.000	0.00
1982 2	10000000	1.022	.183	.946	18.58
1982 3	5818210	5.500	4.740	2.229	100.00
1982 4	5345050	5.500	4.228	1.352	100.00
1982 5	10000000	1.426	0.000	0.000	25.92
1982 6	10000000	0.000	.114	0.000	0.00
1982 7	10000000	.211	.248	0.000	3.83
1982 8	9999990	3.113	2.002	1.146	56.61
1982 9	10000000	3.997	1.279	2.771	72.68
1982 10	10000000	1.645	.564	1.079	29.90
1982 11	10000000	0.000	0.000	0.000	0.00
1982 12	10000000	0.000	0.000	0.000	0.00

Pumping Eficiency = 39.3090623193 %
Pumping Eficiency for El Pozo = 28.9637529138 %

2.2 Consideration on the Location of Headworks

2.2.1 General

The following three sites were proposed as a location for the installation of headworks, and field survey was carried out on each site (refer to Fig. L.2.2).

Proposal H-1: Villa Riva (Upper Stream)
Proposal H-2: Chiringo (Medium Stream)
Proposal H-3: Arenoso (Lower Stream)

It is disclosed as a result of the topographic and water level surveys carried out in the course of the field works that the supply of irrigation water by gravity is viable locating the headworks at any of the said proposals. Consequently, the Proposal H-1 has been excluded from further study, for the driving channel to be included in that proposal becomes so long that it constitutes a definite disadvantage to compare with other two ones. Then, the selection of the location has been made between Proposals of H-2 and H-3. In case of the Proposal H-2, considering that the installation site is to be located in the upper stream of the Pumping Station of the El Pozo Project, the design intake requirement was computted to take that (5.5 m^3/s) for the El Pozo Project into account.

2.2.2 Design Intake Water Level

The design intake water levels, which was calculated considering the water level at the diversion works of El Aguacate (EL = 7.20 m) and the conveyance loss in the driving channel, are as presented below:

Proposal H-2: EL 8.30 m
Proposal H-3: EL 7.60 m

The level for the Proposal H-2 was set up higher than that for the Proposal H-3, because the conveyance loss during siphon works for the former was larger than that for the latter. Detailed design criteria are presented in Tables L.2.1 and L.2.2.

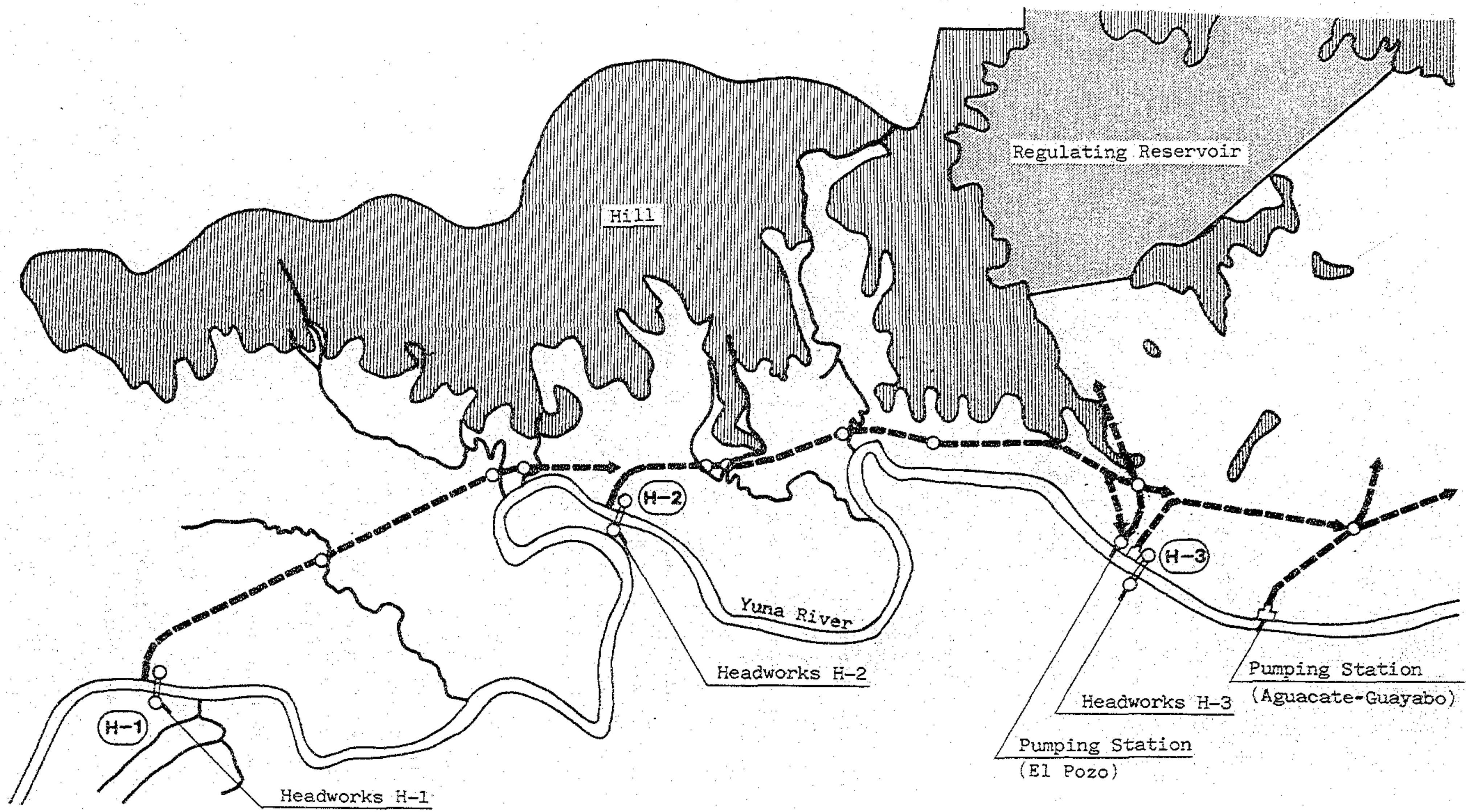


FIG.L.2.2 LOCATION OF HEADWORKS

TABLE L.2.1 DESIGN CRITERIA FOR THE PROPOSAL H-2

Components	Length (L)	Discharge (Q)	Head Loss (h)	Energy Head Elevation	Water Surface Elevation
	m	m^3/s	m	m	m
El Aguacate Diversion Works	-	-	-	7.235	7.200
Driving Channel B-2	1280.0	5.90	0.213	7.448	7.413
Siphon 4	20.0	"	0.053	7.501	7.466
El Pozo Diversion Works	0.0	"	0.050	7.551	7.507
Driving Channel B-1	2570.0	11.40	0.367	7.918	7.874
Siphon 3	40.0	"	0.079	7.997	7.953
Siphon 2	30.0	"	0.076	8.073	8.029
Siphon 1	30.0	"	0.076	8.149	8.105
Intake Facilities	30.0	"	0.150	-	8,255
	4000.0	-	-	-	≒ 8.300

Note: 1] water level lowering

TABLE L.2.2 DESIGN CRITERIA FOR THE PROPOSAL H-3

Components	Length (L)	Discharge (Q)	Head Loss (h)	Energy Head Elevation	Water Surface Elevation
	m	m^3/s	m	m	m
El Aguacate Diversion Works				7.235	7.200
Driving Channel	1278.0	5.90	0.213	7.448	7.413
Intake Facilities	22.0		0.150 1]		7.563
	1300.0				≒ 7.600

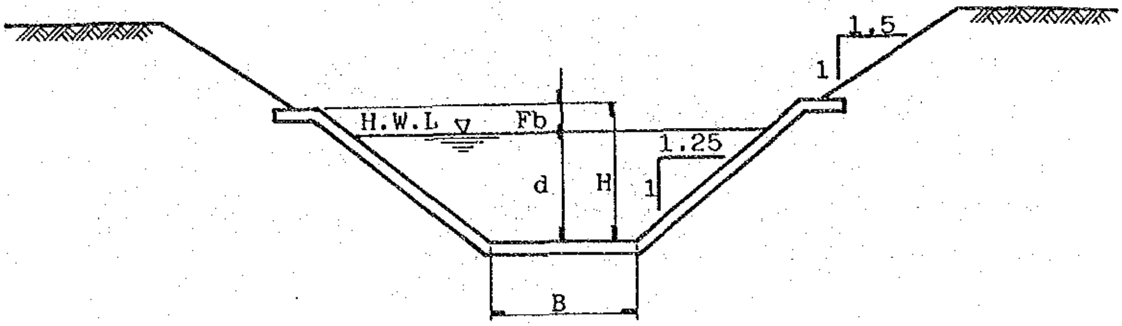
Note: 1] water level lowering

2.2.3 Driving Canal

A driving canal to connect the headworks with the diversion works at El Aguacate has been proposed as illustrated below. The section and design criteria corresponding to each Proposal are as presented below:

(1) Proposal H-2

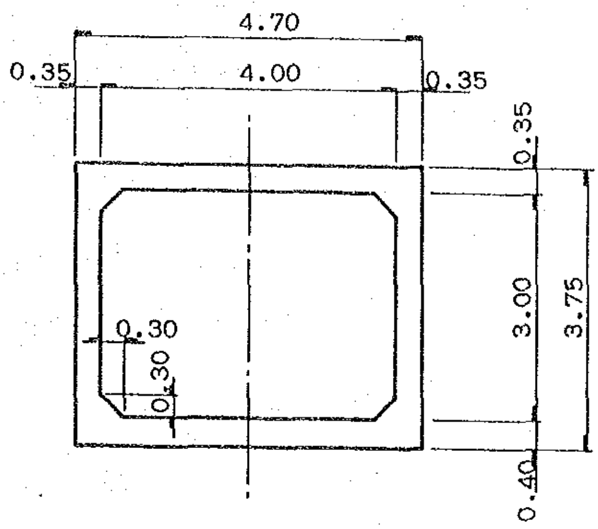
1) Driving Channel



Components		Unit	Driving Canal		
			B-1	B-2	B-3
Discharge	(Q)	m^3/s	11.4	5.9	5.5
Gradient of canal	(I)		1/7000	1/6000	1/6000
Width of invert	(B)	m	2.400	2.000	2.000
Water depth	(d)	m	2.323	1.715	1.657
Flow area	(A)	m^2	12.321	7.107	6.746
Wetted perimeter	(P)	m	9.837	7.491	7.305
Hydraulic radius	(R)	m	1.252	0.949	0.923
Velocity of flow	(V)	m/s	0.926	0.831	0.816
Velocity head	(hv)	m	0.044	0.035	0.034
Height of lining	(H)	m	2.700	2.000	2.000
Freeboard	(Fb)	m	0.377	0.285	0.343

2) Siphons 1 - 3

Siphons for the crossing of the tributaries in the course of the Driving channel B-1 has been proposd as follows:



Design criteria at full water level are as summarized below.

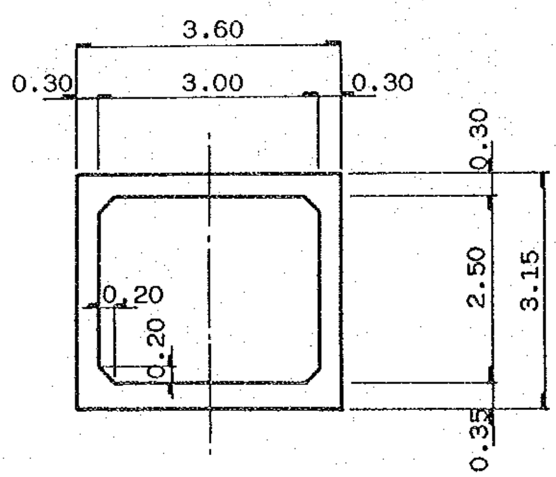
Discharge	(Q)	:	11.400 m^3/s
Flow area	(A)	:	12.000 m^2
Wetted perimeter	(P)	:	14.000 m
Hydraulic radius	(R)	:	0.857 m
	($R^{2/3}$)	:	0.9023
Velocity of flow	(V)	:	0.950 m/s
Velocity head	(hv)	:	0.046 m
Hydraulic gradient	(I)	:	$(\frac{n.V}{R2/3})^2$

$$= (\frac{0.015 \times 0.950}{0.9023})^2 = 0.000249$$

Item		Unit	Siphon 1	Siphon 2	Siphon 3
Length of siphon	(L)	m	30.0	30.0	40.0
Friction loss (hf)	(I.L)	m	0.007	0.007	0.010
Inlet loss (hi)	(0.5 hv)	m	0.023	0.023	0.023
Outlet loss (ho)	(1.0 hv)	m	0.046	0.046	0.046
Total loss	(h)	m	0.076	0.076	0.079

3) Siphon 4

The siphon 4 to cross the main irrigation canal for the El Pozo Project has been proposed as illustrated below:



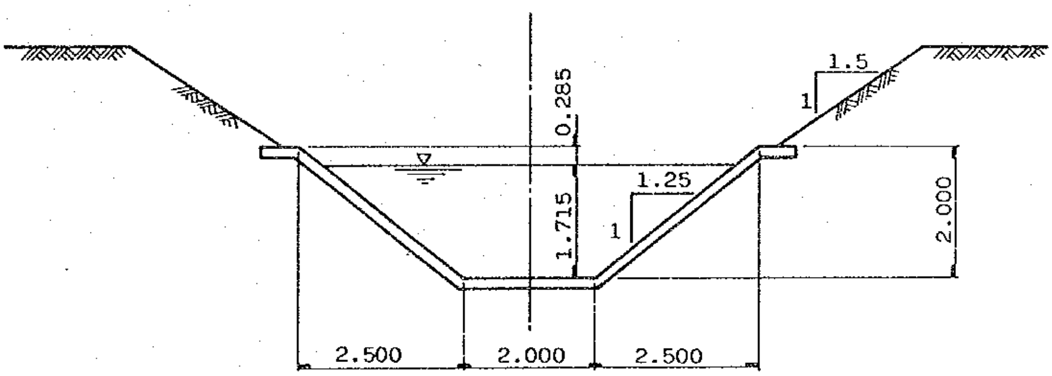
Design criteria at the full water level are as summarized below:

Discharge	(Q)	:	5.900 m^3/s
Flow area	(A)	:	7.500 m^2
Wetted perimeter	(P)	:	11.000 m
Hydraulic radius	(R)	:	0.682 m
	($R^{2/3}$)	:	0.7747
Velocity of flow	(V)	:	0.787 m/s

Velocity head	(hv)	:	0.032 m
Hydraulic gradient	(I)	:	$(\frac{n.V}{R^{2/3}})^2$
		=	$(\frac{0.015 \times 0.787}{0.7747})^2 = 0.000232$
Length of siphon	(L)	:	20.000 m
Friction loss	(hf)	:	I.L = 0.005 m
Inlet loss	(hi)	:	0.5 hv = 0.016 m
Outlet loss	(ho)	:	1.0 hv = 0.032 m
Total loss	(h)	:	0.053 m

(2) Proposal H-3

No siphon has been proposed in the Proposal H-3. The section and design criteria of the channel are as featured below:



Discharge	(Q)	:	5.900 m^3/s	Wetted perimeter	(P)	:	7.491 m
Gradient of canal	(I)	:	1/6000	Hydraulic radius	(R)	:	0.949 m
Width of invert	(B)	:	2.000 m	Velocity of flow	(V)	:	0.831 m/s
Water depth	(d)	:	1.715 m	Velocity head	(hv)	:	0.035 m
Flow area	(A)	:	7.107 m^2	Height of lining	(H)	:	2.000 m
				Freeboard	(F.b)	:	0.285 m

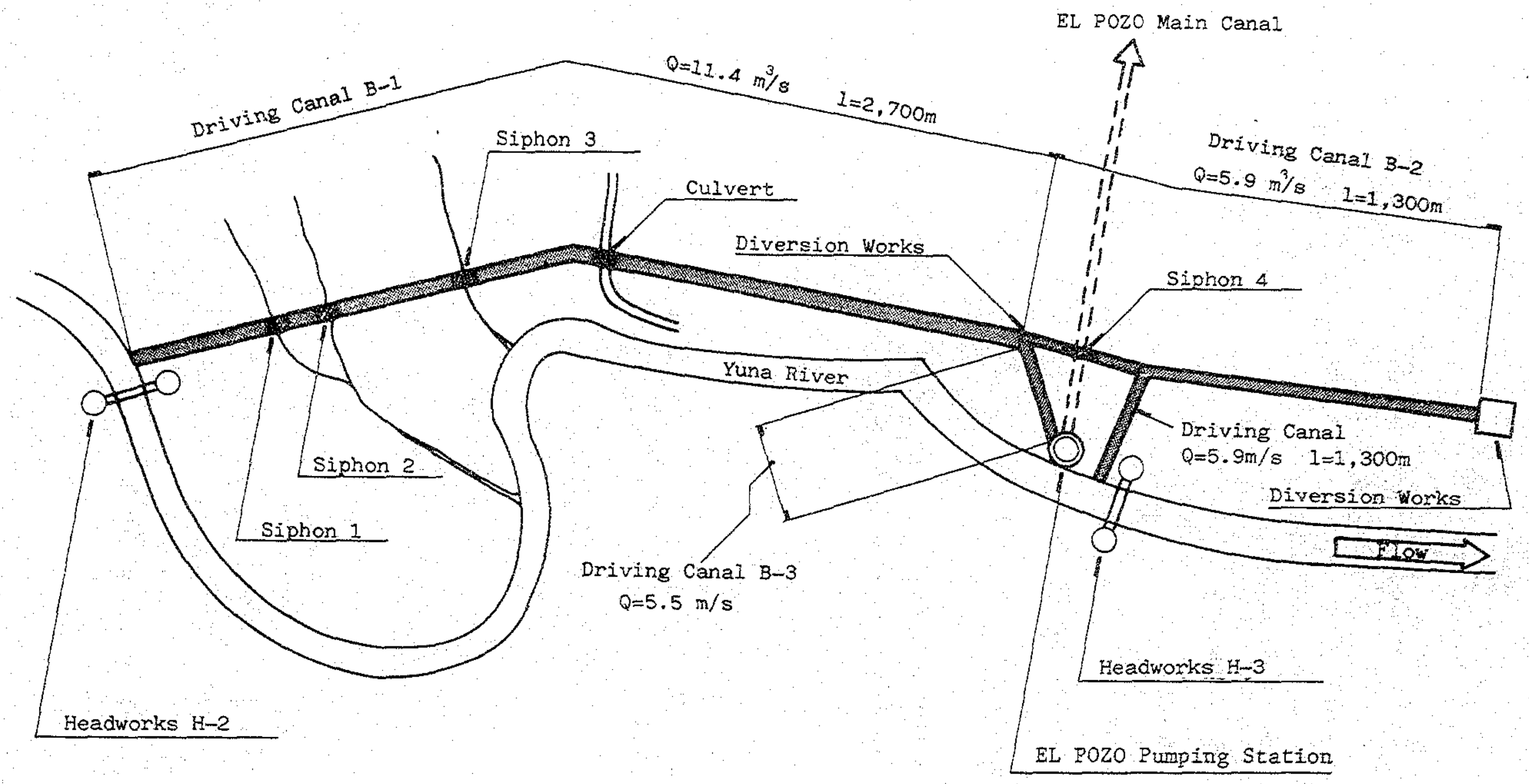


FIG. L.2.3 OUTLINE OF DRIVING CANAL

2.2.4 Design Flood Discharge

In the course of the Yuna River, an overall improvement project has not been formulated up to date, so the design flood discharge has been established to be equivalent to the actual flow capacity. The actual flow capacity can be computed by the Manning Formula.

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

$$Q = A \cdot V$$

where, V : Velocity of flow (m/s)

n : Coefficient of roughness 0.035

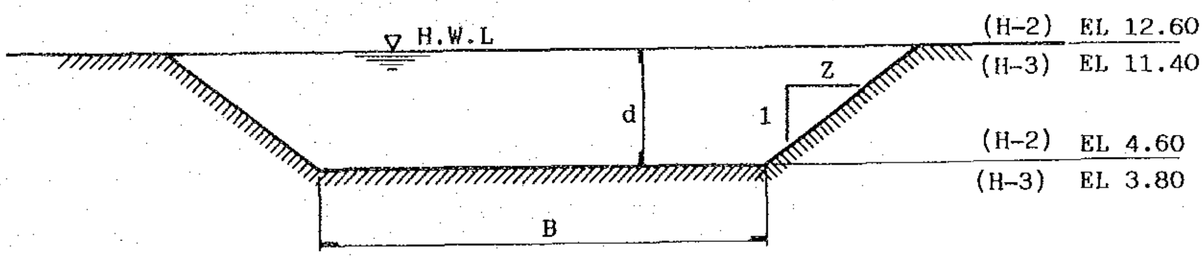
R : Hydraulic radius (m)

I : River slope

A : Cross-sectional area of flow (m^2)

Q : Discharge (m^3/s)

Components		Unit	Proposal H-2	Proposal H-3
River base width	(B)	m	48.000	45.000
Slope gradient	(Z)		1.5	1.5
Water depth	(d)	m	8.000	7.600
Cross-sectional area of flow	(A)	m^2	480.000	428.640
Wetted perimeter	(P)	m	76.844	72.402
Hydraulic radius	(R)	m	6.246	5.920
River slope	(I)		1/3300	1/3300
Velocity of flow	(V)	m/s	1.687	1.628
Discharge	(Q)	m^3/s	809.72	697.69



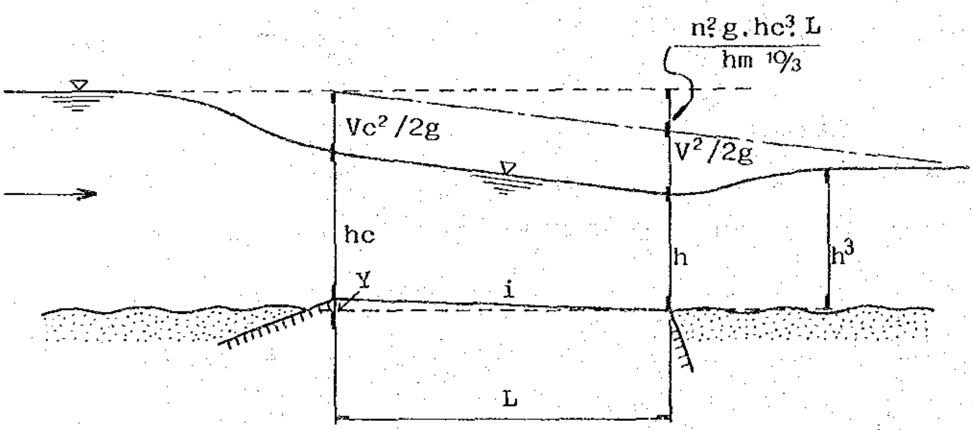
According to the said computation, the design flood discharge and the design flood water level have been established as summarized below:

Proposals	Design Flood Discharge (m^3/s)	Design Flood Water Level (m)
H-2	810.0	12.60
H-3	700.0	11.40

2.2.5 Scouring Sluice

In case of sluggish rivers, the longitudinal slope becomes gentle and the size of materials piled on the river bed results in small. Under these circumstances, materials are easily removed and the slope of scouring sluice is designed to be gentle. In this connection, if the extent of flow through the scouring sluice is designed under sub-critical flow condition, it takes more time to score sluice and permits vain discharge.

Consequently, the extent of flow has been established under jet flow condition. The drop to meet the jet flow condition can be gotton in the following manner.



LONGITUDINAL SECTION OF SCOURING SLUICE

$$Y = h - 1.5hc + \frac{V^2}{2g} + \frac{n^2 . g . hc^3 . L}{hm^{10/3}}$$

where,
- Y : drop
- h : water depth at the end of lower flow of scouring sluice
- V : velocity of flow at the end of lower flow of scouring sluice
- g : gravitational acceleration = 9.8 m/sec^2
- n : roughness coefficient = 0.017
- hc : critical depth
  $hc = (qc^2/g)^{1/3}$
  $qc = V.h$
- L : length of scouring sluice
- hm : average depth within scouring sluice
  $hm = (hc + h)/2$

In the said formula, an appropriate Froude number to corresponde to the water depth at the end of lower flow of a scouring sluice will be 1.75. If given this value, the following formula is presented.

$$h \fallingdotseq 0.5\ h3$$

where, h3 : water depth of lower river flow

$$V = 1.75 \sqrt{gh}$$

Then, if given values for h3, n and L, the drop can be computed. Generally, the discharge in the course of scouring sluice is considered to be less than the average discharge during irrigation period (Qm). The width of a scouring sluice (Bm) is calculated as follows:

$$Bm \leqq Qm/qc$$

(1) Calculation of Water Depth

Given the discharge at the time of scouring sluice to be $Qm = 50\ m^3/s$ for both Proposals H-2 & H-3, the water depth (h3) at the lower flow is determined by using the Manning Formula as summarized in the Table below.

		Unit	Poposal H-1	Proposal H-2
Water depth	(h3)	m	1.560	1.620
River base width	(B)	m	48.000	45.000
Slope gradient	(Z)		1.5	1.5
Cross-sectional area of flow	(A)	m^2	78.530	76.837
Wetted perimeter	(P)	m	53.625	50.841
Hydraulic radius	(R)	m	1.464	1.511
Roughness coefficient	(n)		0.035	0.035
River slope	(I)		1/3300	1/3300
Velocity of flow	(V)	m/s	0.641	0.655
Discharge	(Q)	m^3/s	50.3	50.3

(2) Calculation of Slope and Width of Scouring Sluice

Symbol	Formula	Unit	Proposal H-1	Proposal H-2
h3		m	1.560	1.620
h	$h = 0.5\ h3$	m	0.780	0.810
V	$V = 1.75\ \sqrt{gh}$	m/s	4.838	4.931
qc	$qc = V.h$	$m^3/s/m$	3.774	3.994
hc	$hc = (qc^2/g)^{1/3}$	m	1.132	1.176
hm	$hm = (hc + h)/2$	m	0.956	0.993
n			0.017	0.017
l		m	47.00	43.00
Y		m	0.500	0.490
i			1/94.0	1/87.7
Bm	$Bm = Qm/qc$	m	13.2	12.5

(cont'd)

Symbol	Formula	Unit	Proposal H-1	Proposal H-2
Y		m	0.500	0.490
i			1/94.0	1/87.7
Bm	Bm = Qm/qc	m	13.2	12.5
Length			47.00 m	43.00 m
Width			12.50 m	12.50 m
Longitudinal slope			1/90	1/85

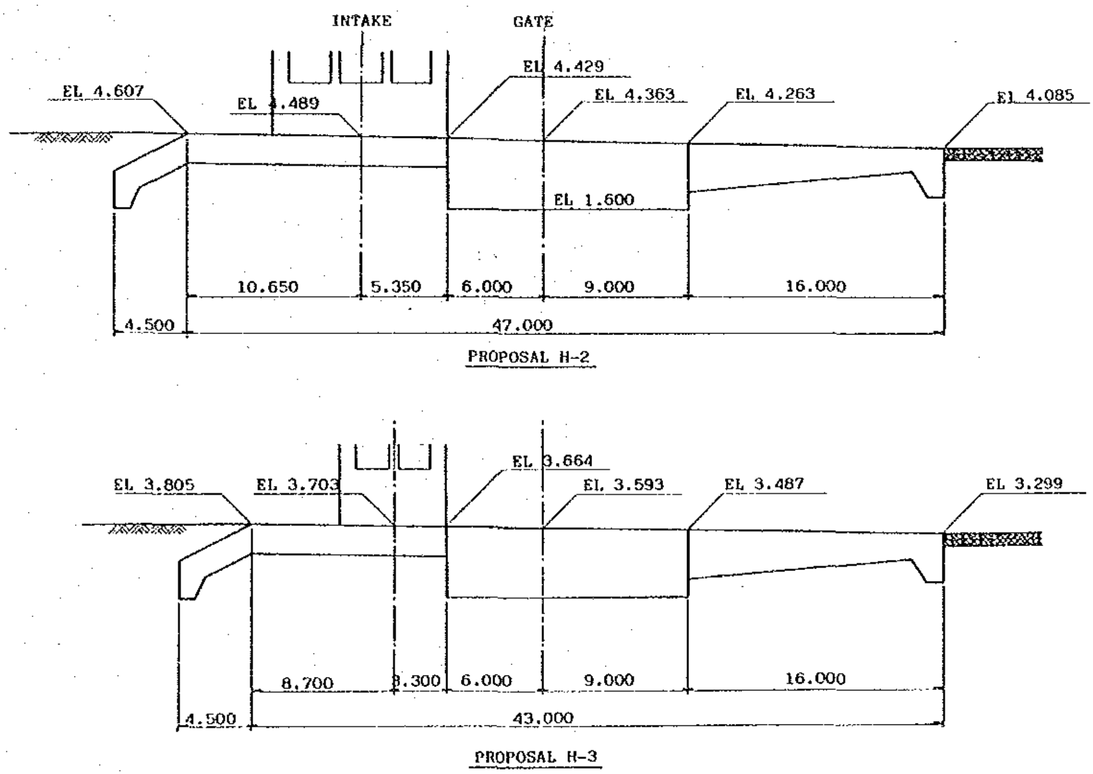
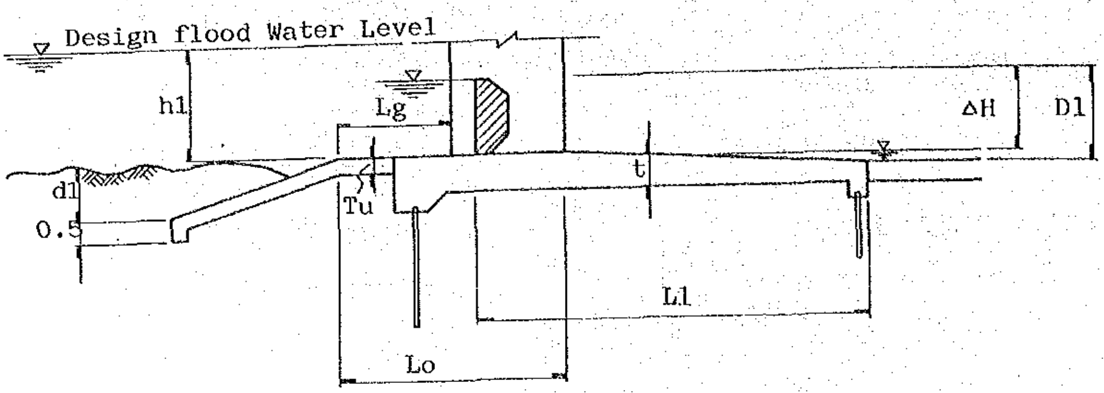


FIG. L.2.4 LONGITUDINAL SECTION OF SCOURING SLUICE

2.2.6 Flood Sluice



LONGITUDINAL SECTION OF FLOOD SLUICE

(1) Length of Rear Apron

The length of a rear apron is determined using the Bligh Formula,

. $L1 = 0.9\ C\ \sqrt{D1}$

where, L1 : length of rear apron (m)

D1 : difference between the water surface of rear apron and the rest of gate

Proposal H-2 : D1 = 8.3 - 4.6 = 3.7 m

Proposal H-3 : D1 = 7.6 - 3.8 = 3.8 m

C : C for the Bligh Formula

Proposal H-2 : C = 15 (fine sand)

Proposal H-3 : C = 12 (coarse sand)

1) Proposal H-2

$L1 = 0.9 \times 15 \times \sqrt{3.7} \fallingdotseq 25.9 = 25.0$ m

2) Proposal H-3

$$L1 = 0.9 \times 12 \times \sqrt{3.8} \doteqdot 21.1 = 25.0 \text{ m}$$

(2) Thickness of Rear Apron

The thickness of rear apron can be obtained by,

$$t \geqq \frac{4}{3} \cdot \frac{\Delta H - Hf}{\gamma - 1}$$

where, t : thickness of rear apron (m)

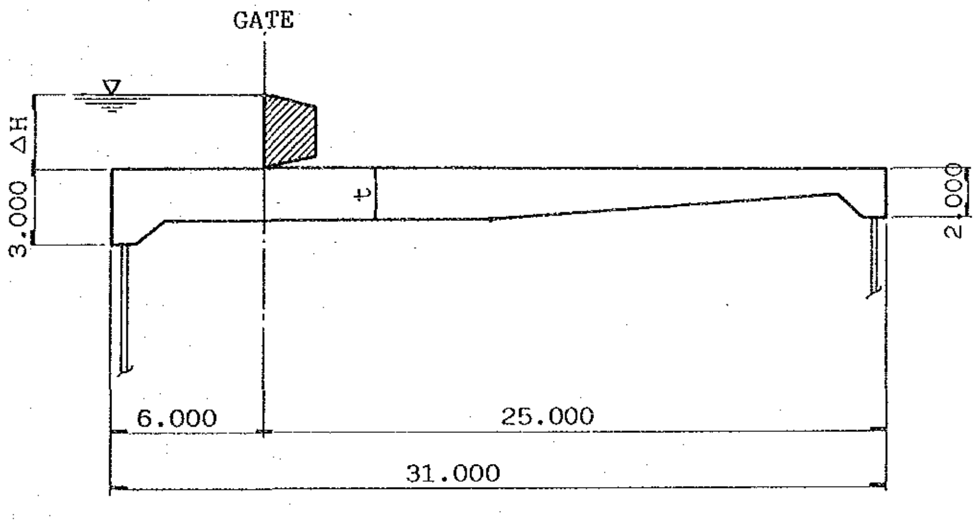
ΔH : difference in water surface elevations

Proposal H-2 : $\Delta H = 3.7$ m

Proposal H-3 : $\Delta H = 3.8$ m

Hf : head loss due to friction

γ : specific gravity of apron = 2.50



And, the head loss due to friction was computed,

$$Hf = \frac{\Delta H}{S} \cdot S'$$

where, Hf : head loss due to friction (m)

ΔH : difference in water surface elevations (m)

S : total creep length (m)

S' : creep length to the specific point (m)

Then, the creep lengths obtained by using the Bligh Formula are as follows:

1) Proposal H-2

$$S = 3.0 + 6.0 \times 2 + 3.0 \times 2 + 2.0 + 31.0 = 54.0 \text{ m}$$

$$S' = 3.0 + 6.0 \times 2 + 6.0 = 21.0 \text{ m}$$

$$Hf = \frac{3.7 \times 21.0}{54.0} = 1.44 \text{ m}$$

$$t \geqq \frac{4}{3} \times \frac{3.7 - 1.44}{2.5 - 1.0} = 2.01 \fallingdotseq 2.00 \text{ m}$$

2) Proposal H-3

$$S = 54.0 \text{ m}$$

$$S' = 21.0 \text{ m}$$

$$Hf = \frac{3.8 \times 21.0}{54.0} = 1.48 \text{ m}$$

$$t \geqq \frac{4}{3} \times \frac{3.8 - 1.48}{2.5 - 1.0} = 2.06 \fallingdotseq 2.00 \text{ m}$$

2.2.7 Intake

(1) Elevation of Intake Invert

The elevations of intake invert were calculated to comply with the following conditions:

1) that it should be higher by more than 1 m than that of scouring sluice invert and

2) that the difference between the elevations of intake invert and scouring sluice invert should be larger than one-sixth of flood water depth.

		Proposal H-2	Proposal H-3
A.	Elevation of Intake invert	6.700 m	6.000 m
B.	Elevation Scouring sluice invert	5.020 m	4.196 m
C.	A - B	1.680 m	1.804 m
D.	Flood water depth	8.000 m	7.600 m
E.	D/6	1.333 m	1.267 m

(2) Cross-section of Intake

The cross section of intake has been proposed, given that the intake water depth to be 1.60 m and the velocity of flow to be around 1.00 m/s.

Components	Proposal H-2	Proposal H-3
Intake water depth	1.600 m	1.600 m
Width of Intake	2.500 m	2.000 m
Number of Span	3	2
Discharge of Intake Water	11.4 m^3/s	5.9 m^3/s
Velocity	0.950 m/s	0.922 m/s

2.2.8 Construction Cost

The construction costs, built up in each Proposals, are as follows:

Proposal H-2 : RD$22,320,000
Proposal H-3 : RD$13,997,000

A summary of breakdown is shown in the table below.

(Unit: RD$)

Item	H-2	H-3
1. Headworks	12,562,000	13,257,000
2. Driving Canal	5,237,000	740,000
3. Siphon	400,000	0
4. Diversion Works	497,000	0
5. Culvert	48,000	0
6. Bridge	800,000	0
7. Construction Road	294,000	0
8. Emergency Floodway	2,446,000	0
Total	22,320,000	13,997,000

2.2.9 Summary

A comparative study to determine the predominance between the two Proposals of H-2 and H-3 with respect to the location of headworks has been made and, as a result, the Proposal H-3 has been selected on which further study is to be carried out. Technical justifications for the selection of the Proposal H-3 are as presented hereinbelow.

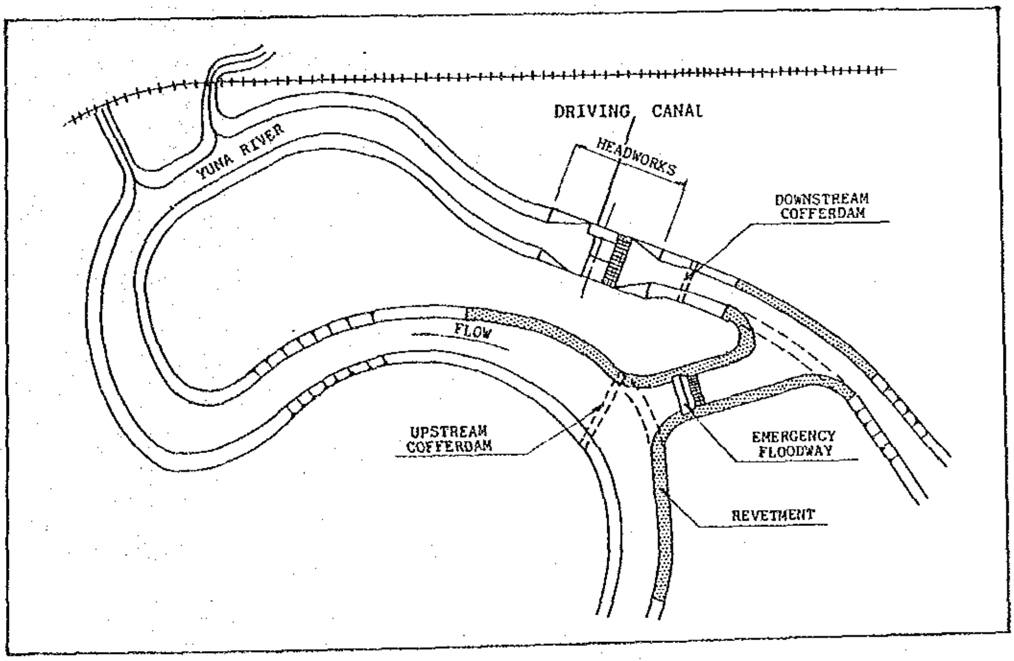
(1) Less Construction Cost is Required

(2) Convenience and Safety for Operation and Maintenance Stages

If the headworks is to be installed at the site of the Proposal H-3, which is located in the neighborhood of the El Pozo Pumping Station, an integrated organization to combine the headworks of the Aguacate-Guayabo Project and the Pumping Station of the El Pozo Project will be envisaged. In the case of the Proposal H-2, an independent organization apart from the El Pozo project will be needed to take the distance of two installations into account. Furthermore, considering the proposed course of the driving canal is compelled to be passed through the flooding catchment area of the Yuna River, appropriate measures to eliminate inflowing mud to the canal should be taken for the Proposal H-2, and there remains a fear that the operation and maintenance road will be damaged by the flooding.

(3) Stability of the River Course in Both Upper and Lower Streams

In the case of the Proposal H-2, the proposed location of the headworks and 1.3 km upper stream point are so close due to the curve of the river course that there remains fear for being changed the river flow by further erosion of the curved area in the future. If the headworks will be constructed in the Proposal H-2, revetment or water control measures should be considered in view of maintaining the actual river flow.



(4) Beneficial to the El Pozo Project

The Proposal H-2 has an influence upon the El Pozo Project because a driving channel crosses with the main irrigation canal of the El Pozo.

In such case, construction works should be carried out when there supplies no water through the driving channel.

The Proposal H-3, on the contrary, has no influence on the El Pozo Project, its headworks being located lower stream of the related pumping station. The El Pozo Project will be benefited by the installation of the headworks on its down stream because: a) the operation cost will be reduced owing to the elevation of the lowest water intake level from EL 4.20 m to EL 7.60 m and b) the pumping potentiality will be raised to contribute to the extension of the benefited area in the future.

(5) Saving of Construction Period

If the headworks is to be located at H-3 site, no constraint will be presented in the context of the access road to the construction site, because those roads existing on the both banks of the Yuna River will be available as a access road. If the headworks should be constructed at H-2 site, it is required to broaden the existing road located 1.8 km apart from the Villa Riva-Arenoso Road or to construct the operation and maintenance road in advance of the commencement for the construction of the headworks so as to make use of it as a access road.

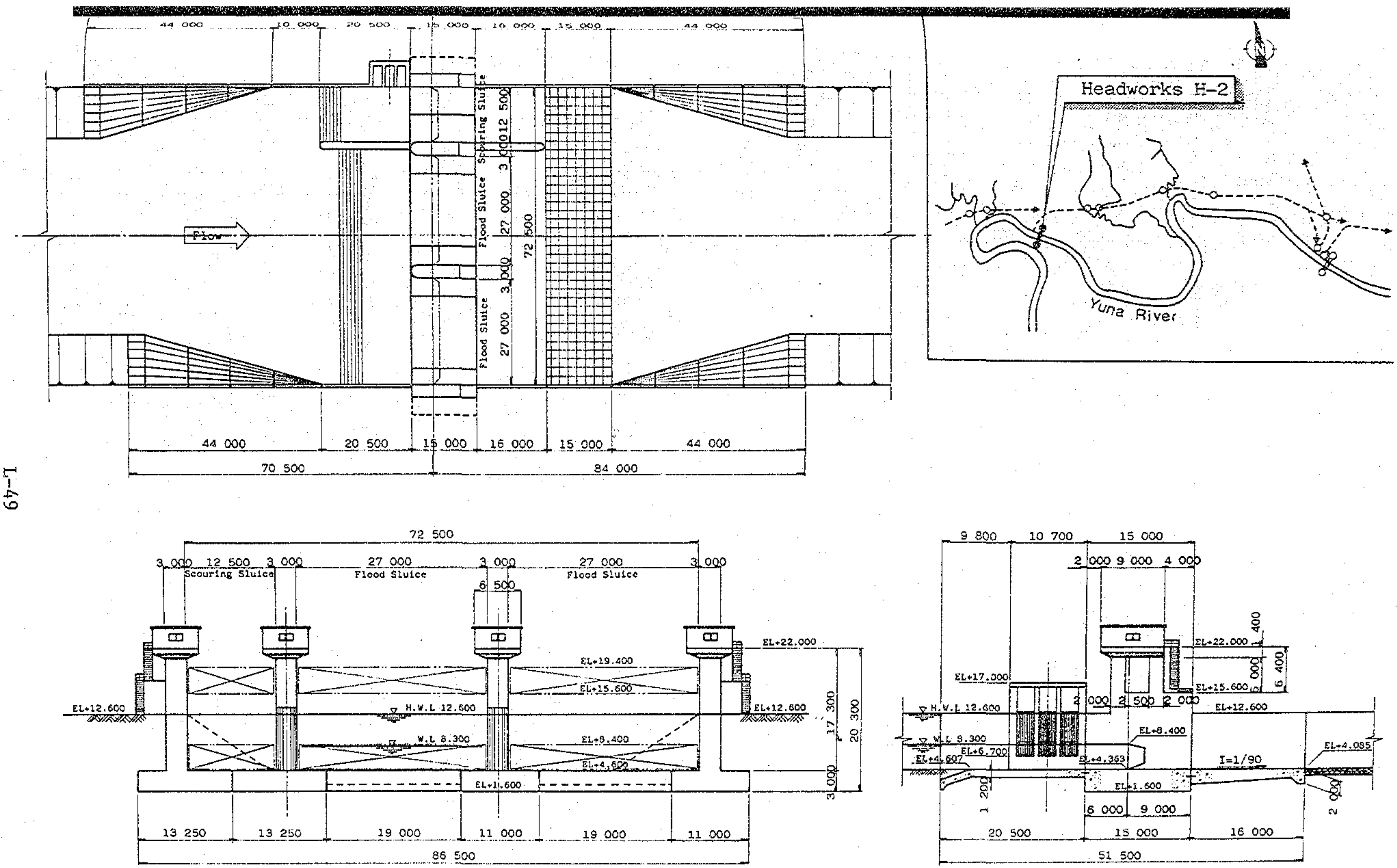


FIG. L.2.5 GENERAL PLAN OF PROPOSAL H-2

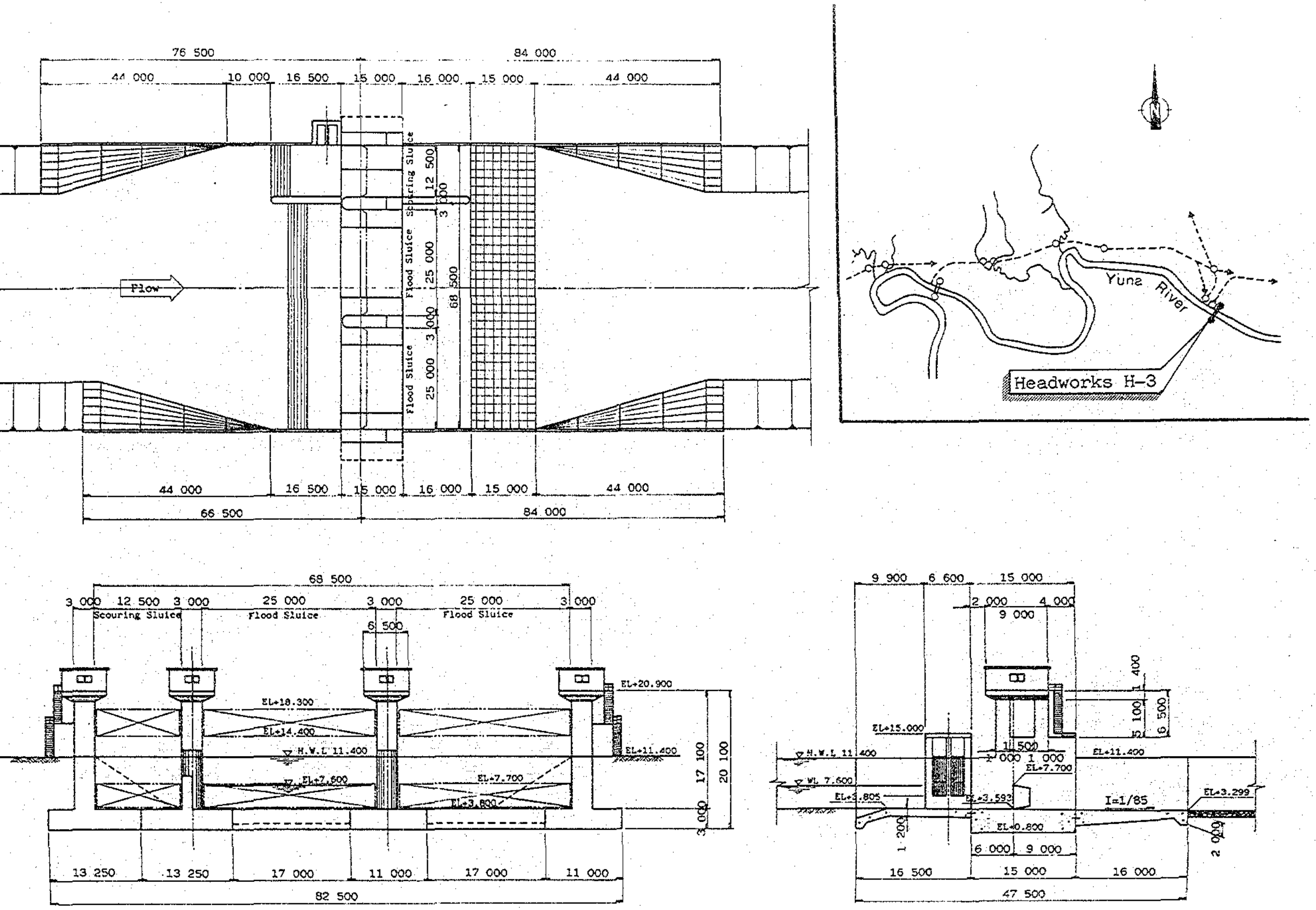


FIG. L.2.6 GENERAL PLAN OF PROPOSAL H-3

3. Headworks

3.1 General

Headworks constitutes the principal water intake facility to be installed in the Yuna River; the design intake requirement has been taken as 5.90 m^3/s and the location has been proposed at 200 m downstream the Pumping Station for the AGLIPO Project.

3.2 Type

For flood mitigation purpose, a movable weir has been proposed to be fixed to the headworks and whose section has been designed larger than present cross-sectional area of flow. Considering geological condition of sub-base, a floating type of weir is recommended.

3.3 Gates

Dimentions of Gates are as follows:

Scouring Sluice Gate	:	12.500 m (W) x 4.107 m (H) x 1 set
Flood Sluice Gate	:	25.000 m (W) x 3.900 m (H) x 2 sets
Intake Gate	:	2.000 m (W) x 2.000 m (H) x 2 sets

A switch of one-motor-one-drum type will be fixed so as to facilitate manual operation in case of emergency.

3.4 Design Intake Water Level

The design intake water level has been taken as W.L. 7.60 m (refer to 2.2.2).

3.5 Hydraulic Calculation of Intake

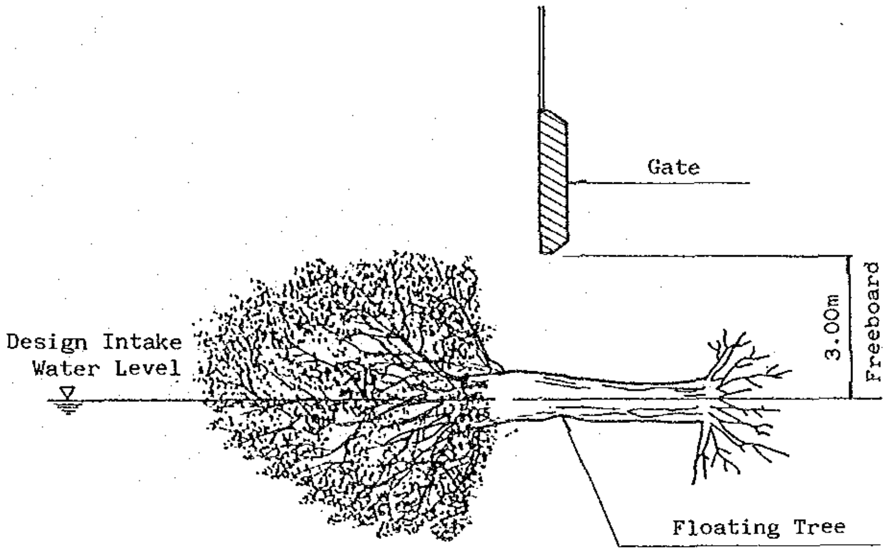
A hydraulic calculation of intake is presented in Table L.3.1. Given that the design intake level at headworks is W.L. 7.60 m, the design intake level at No. 1 diversion works becomes W.L. 7.28 m, which covers the required level at diversion works of W.L. 7.20 m.

TABLE L.3.1 HYDRAULIC CALCULATION OF INTAKE

Item	Distance	Discharge	Velocity	Head Loss	Energy Head Elevation	Velocity Head	Water Surface Elevation	Water Depth	Canal Invert Elevation	Remarks
	(m)	(m^3/s)	(m/s)	(m)	(m)	(m)	(m)	(m)	(m)	
Intake water Level			5.90			7.600		7.600		
Inlet loss	0.0	5.90	0.823	0.007	7.593	0.034	7.559	1.559	6.000	
Weir pillar loss	6.0	5.90	0.959	0.008	7.585	0.047	7.538	1.538	6.000	
Screen loss	0.0	5.90	0.989	0.044	7.541	0.050	7.491	1.491	6.000	
Box culvert	10.0	5.90	0.989	0.004	7.537	0.050	7.487	1.491	5.996	Friction loss
Rapid-expanding loss	0.0	5.90	0.600	0.008	7.529	0.018	7.511	1.515	5.996	
Transition loss	6.0	5.90	0.830	0.006	7.523	0.035	7.488	1.715	5.773	
Driving canal	1,278.0	5.90	0.830	0.213	7.310	0.035	7.275	1.715	5.560	Friction loss
Total	1,300.0			0.290						

3.6 Freeboard at a Time of Raising a Gate

Generally speaking, a freeboard between the raised gate and the flood level is reserved at more than 1.5 m. In case of the Yuna River, because no protection works have been implemented and the floating tree with a diameter of about 1.0 m. is reported, a freeboard has been designed at 3.0 m.



3.7 Countermeasures in Case of No Functioning of Gates

As mentioned before, for flood mitigation purpose, a floating type movable weir has been proposed. In this case, an adequated gate operation should be maintained even if small flooding, because the elevated level of gate is designed higher than one-half of water depth at flooding. Gate will not be function adequately in the following cases, namely:

(1) Electric Supply will be Shut Off

(2) In Capability of Operation

(3) Too much load will be born on switch device due to lack of appropriate maintenance and operation systems

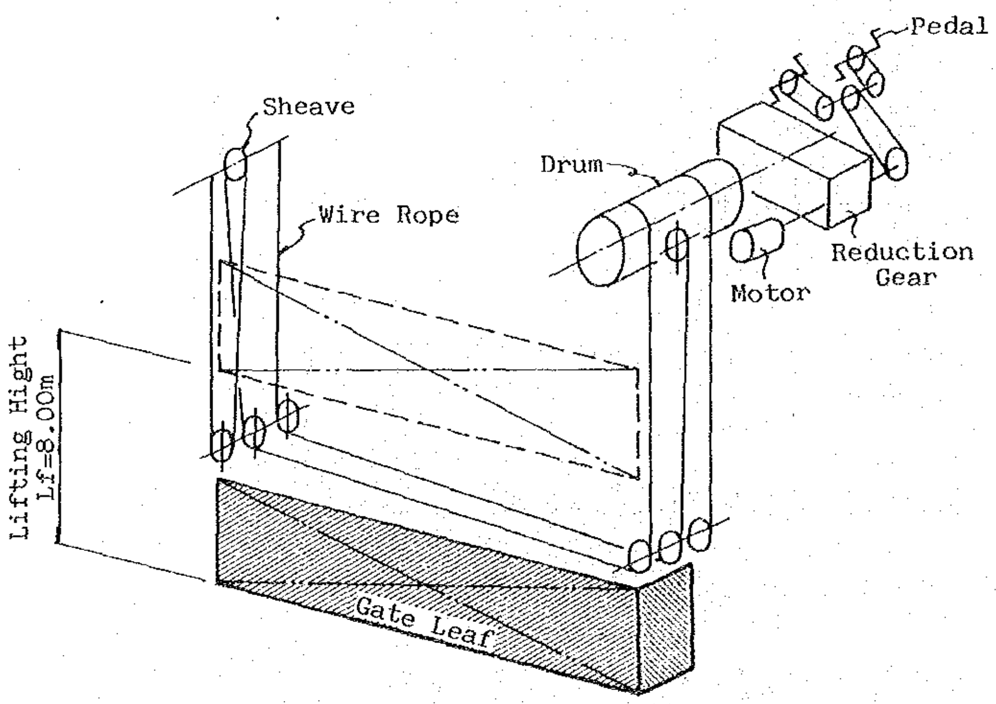
Countermeasure 1: Supplemental power supply source will be considered

Supplemental power supply source consisted of engine and generator will be deviced. Additional cost to be incurred related to this device is as estimated below:

Engine for Flood Sluice Gate:	RD$112,500
Engine for Scouring Sluice Gate:	RD$ 39,500
Sub-total	RD$152,000
Generator (60 KVA)	RD$ 62,500
Total	RD$214,500

Countermeasure 2: Manual operation system will be established

Though, a switch with 2 motors - 2 drums is generally fixed to a long span (15 m or more) wheel gate, that with 1 motor - 1 drum system has been proposed so as to facilitate a manual operation, Flood sluide gate will be raised within 24 hours with a use of pedal. Additional cost to be incurred related to this device will be RD$26,000.



Countermeasures 3: Consolidation of O/M system

In order to achieve the proper function of the gate, the adequated training system for the person to be responsible for the operation and maintenance of the installments should be established. For this purpose, it is advisable to station an O/M expert at the initial stage of the operation so as to make a technology transfer to the technicians of the O/M system.

Countermeasure 4: Installation of an emergency flood way

An emergency flood way will be considered as to inflow flooded water incase that a flood sluice gate should not open. The section of this flood way will be kept not smaller than actual flow area (430 m^2).

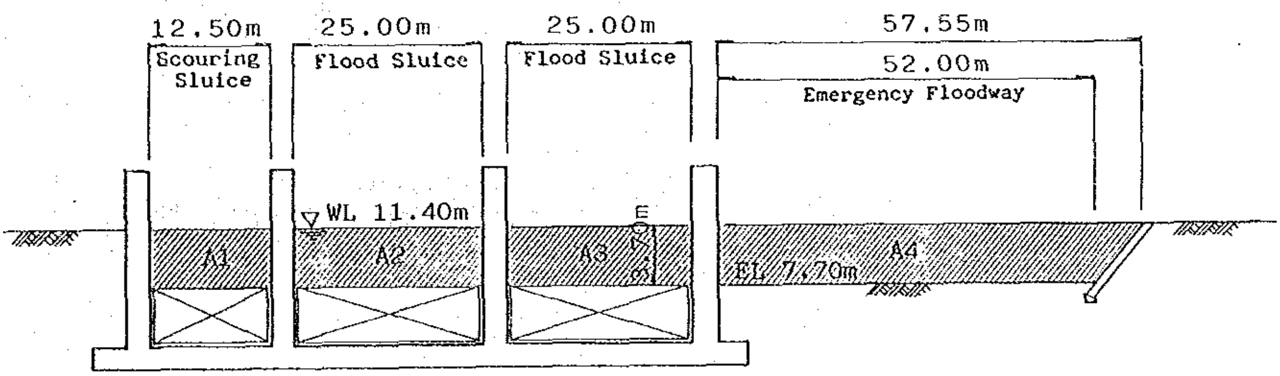


FIG. L.3.3 EMERGENCY FLOODWAY

Scouring Sluice	:	A1	= 12.50 x 3.70 =	46.25 m^2
Flood Sluice (1)	:	A2	= 25.00 x 3.70 =	92.50 m^2
" (2)	:	A3	= 25.00 x 3.70 =	92.50 m^2
Emergency Floodway:		A4	= $\frac{1}{2}$ (52.00 + 57.55) x 3.70 =	202.67 m^2
Total				433.92 m^2 > 430 m^2

4. Pumping Station

4.1 Location

In order to decide the location of the proposed pummping station, topographical survey and geological investigation were conducted in the selected sites. As results of the studies, left bank of the Yuna River was chosen as location of the pumping station based on the following reasons.

a) Flow of the Yuna River is stable and required quantities of intake water can be taken continuously.

b) As depth of bed rock is shallow, the foundation treatment will be easy and economical.

c) Accessibility to the site is quite easy due to rural road is passing beside the proposed site.

d) The site is in less populated area.

e) The site is in the most upstream of main irrigation canal and advantageous from hydraulic and economical point of view.

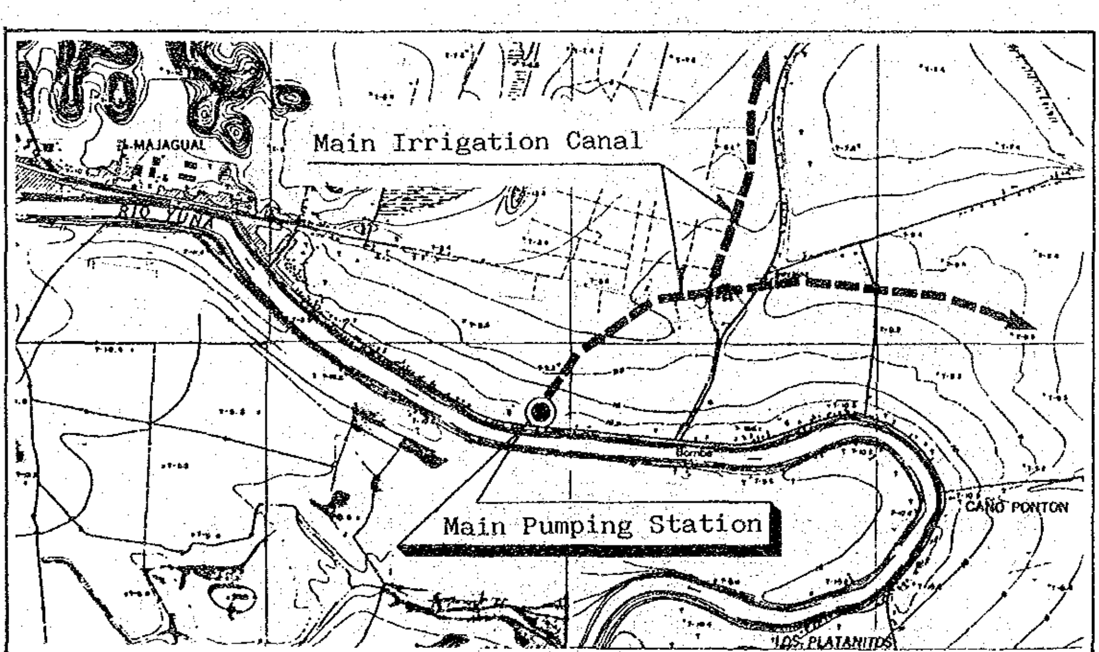


FIG. L.4.1 LOCATION OF PUMPING STATION

4.2 Design of Pumping Equipment

4.2.1 Units and Diameter

Taking into consideration comprehensively adjustability of pumping amount, dispersion of risk, scale of pump house and cost of equipment, unit and diameter of pumping equipment are decided at three units and ∅900 mm, respectively. No spare unit will be provided due to less risk with three units of pumps.

4.2.2 Total Head

(1) Suction Water Surface

The drought discharge of the Yuna River for 1:5 return period at the proposed site for the installation of a pumping station was estimated at $Q = 17.2\ m^3/s$. With this water discharge, the water surface will be as high as EL 4.00 subject to the following conditions:

Width of river bed:	34.00 m
Slope gradient:	1/1.5
River slope:	1/3,300
Coefficient of roughness:	0.035
Elevation of river bed:	EL 3.00 m

And, given water loss in the course of passing through water gate, screen, etc. to be 0.30 m, the suction water surface is thus determined at W.L. 3.70 m.

(2) Discharge Water Surface

The required water elevation at the No.1 diversion works (El Aguacate) which is connected with the main canal is EL 7.20 m. The discharge water surface of W.L. 8.0 m is obtained by adding head loss and allowance - 0.8 m to the said water elevation.

(3) Total Head Loss of Suction and Discharge

Friction head loss of suction and delivery pipes of pump and valves was designed at 0.6 m. Then, the total head has been obtained as follows:

$$H = 8.00 - 3.70 + 0.6 = 4.9 \text{ m}$$

4.2.3 Type of Pump Equipment

Considering water elevation of the Yuna River at the time of flooding the prime mover should be installed at higher place than the existing ground surface - E.L. 11.20 m. In such case, the elevation gap between the location of the prime mover and the suction water surface will be more than 8.6 m, which is too high lift for horizontal axis type of pumping equipment to cause cavitation. In light of this, a vertical axis type has been proposed as pumping equipment.

Pump equipment to meet the above-mentioned design criteria are either axial flow pump or mixed flow pump. Technical specifications of each type are compared as follows:

Item	Axial Flow	Mixed Flow
Pump weight	Light	Heavy
Pump efficiency	Low	High
Shaft power	Strong for narrow range of flow amount	Almost constant
Flow adjustment	Difficult	Easy
Cost	Low	High
Suction performance	Poor	Good

Based on the above comparison, mixed flow type of pumping equipment has been selected in terms of flow adjustability and high pump efficiency.

4.2.4 Structure of Pumping Station

With respect to the installation method of vertical axis, mixed flow pump single deck type and double deck type of pumping station may be considered. Comparison of respective type are as presented below:

	Single Deck	Double Deck
Advantage	a) Design of structures to be constructed is simple b) Loads against floor are weight of pump equipment, motor and water c) Easy installation	a) Installation of the motor is easy b) Maintenance of pump equipment can be made easily without dismantling motor, c) Maintenance will be easily due to lower location of motor d) Loads against floors are light
Disadvantage	a) Maintenance will be somewhat complicated due to higher location of motor b) Sensitive to vibration	a) Design of structures to be constructed is complicated

The above comparison draw a conclusion that double deck type is preferable as structure of pumping station.

4.2.5 Rotation Speed of Pump

Rotation (r.p.m.) of pump is calculated by the following equation.

$$N = \frac{Ns \times H^{3/4}}{Q}$$

where: Ns : Specific speed of pump
600 - 1,400 for mixed flow type of pump
take 1,400 in this case

Q : Discharge amount, 118 m^3/min.

H : Total head, 4.9 m

$$N = \frac{1{,}400 \times 4.9^{3/4}}{118} = 424 \text{ r.p.m.}$$

Thus, motor with 16 poles and the rotation speed of 445 r.p.m. is selected. (Including 1% slip)

4.2.6 Capacity of Motor

The required capacity of the motor is calculated using the following equation.

$$S = \frac{0.163 \times Q \times H}{\eta p} \times (1 + \alpha)$$

where:

Q	:	Required discharge amount	118 m^3/min.
H	:	Total head	4.9 m
ηp	:	Pump efficiency	80.0%
α	:	Allowance	20.0%

$$S = \frac{0.163 \times 118 \times 4.9}{0.8} \times (1 + 0.2) = 142 \text{ KW}$$

Thus, a motor with 16 poles and a capacity of 145 KW is proposed.

Major dimensions of the proposed pumping equipment are summarized as follows:

Type of pump	:	Vertical mixed flow pump
Discharge capacity	:	118 m^3/min
Total head	:	4.9 m
Rotation	:	445 r.p.m.

Motor output	:	145 KW
Installation	:	Double deck type
Units	:	3
Pump diameter	:	900 mm

4.3 Pumping Station and the Related Facilities

The scale of the pumping station is decided depending upon number, diameter and type of pumping equipment. Related facilities of the pumping station involve intake structure with gates, suction and delivery pool, screen, and switch-yard. Considering the discharge water amount and topographic feature of the site, the layout of the facilities is proposed as shown Fig. L.4.2.

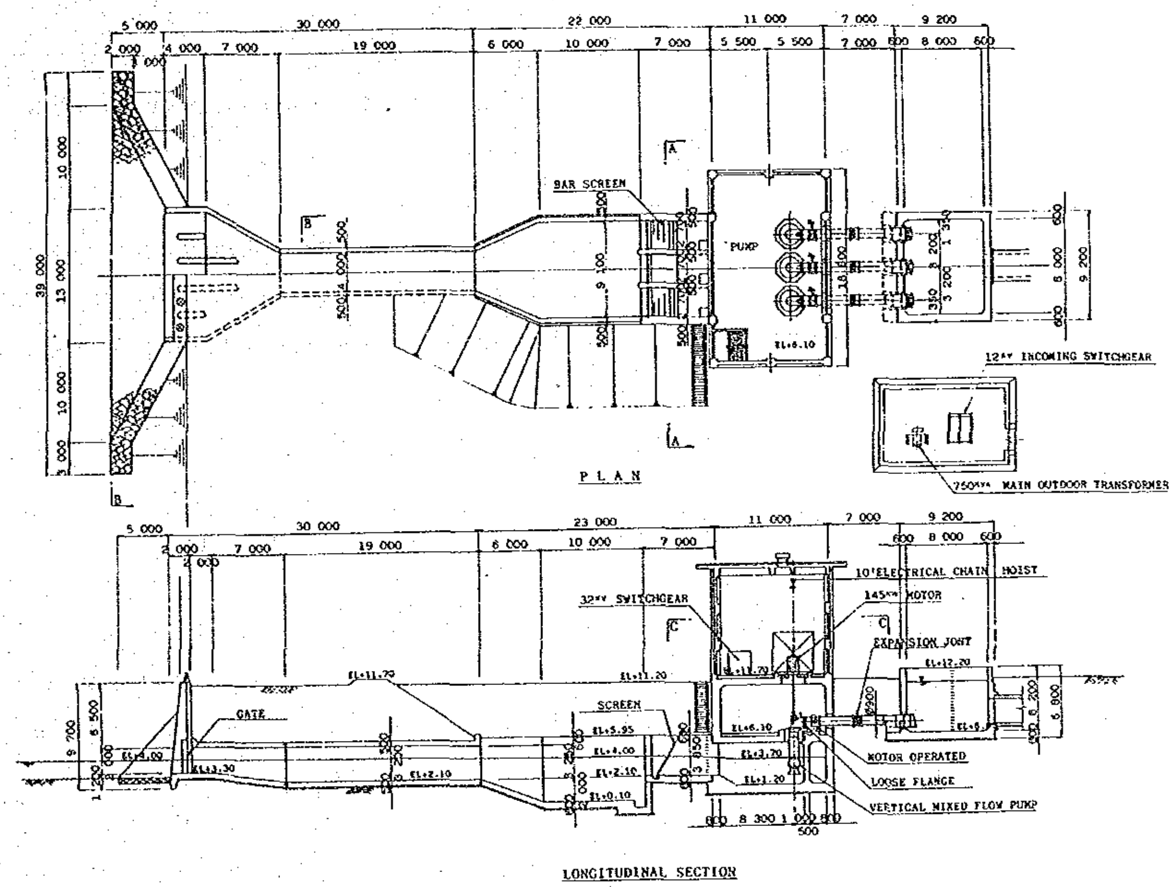


FIG. L.4.2 GENERAL PLAN OF MAIN PUMPING STATION

4.4 Constant Intake of Water

The location of the pumping station is proposed about 900 m downstream of the El Pozo pumping station. Although the river flow passing through the site is relatively stable, it is required to execute ground sill on river bed and/or to construct weir and other works so that river water may be obtained constantly. The execution of ground sill is recommended by the following reasons.

- Construction cost is cheaper than the installation of weir.

- Without dam-up of river discharge, there will be less fear against flooding.

- The water dpeth of at least 1.0 m will be secured in the event of drought water ($Q = 17.2\ m^3/s$) for 1:5 return period. Therefore, the river discharge can be pumped up in any season without dam-up by a weir.

- If a weir should be installed, movable one will be recommended in terms of mitigating sedimentation and flood control. Construction cost of such weir structure with movable gate is considerably high.

4.5 Operation and Maintenance of Pump

4.5.1 Operation

Each pump equipment will be operated by the control switches to which an operator gives command to start and to stop. When any trouble or disorder occur, the pump is automatically stopped and, at the same time, alarm is given to the operator. As to major trouble, overload in the motor and abnormal suction water level are considered. The adjustment of water discharge will not be made by the control of valve but by changing number of pump in operation. This adjustment will be made in principal at half a day interval and no shorter period.

Operation manager except operators can work for the project together with El Pozo Project.

Number of pumping equipment to be operated is decided by newly established O&M Office of this Project.

4.5.2 Maintenance

Pumps and motors should be checked and inspected periodically, according to the O&M manual prepared by the O&M office. The following tasks are required.

- Check and repainting of metal materials such as gates, screen and so on.

- Removal of sedimentation in suction and delivery pools.

- Cleaning of screen (removal of trash from screen) and others.

For better management, instant and accumulated value should be recorded by water level gauging instrument and current meters which are to be installed at appropriate places.

4.5.3 Anual Cost of Electricity

Annual operation hours of the pumping equipment are estimated at 5,143 hours and annual cost of electricity required for running pumping equipment is calculated at RD$543,641.

145 kw x 5,143 hr. x RD$0.243 x 3 units = RD$543,641

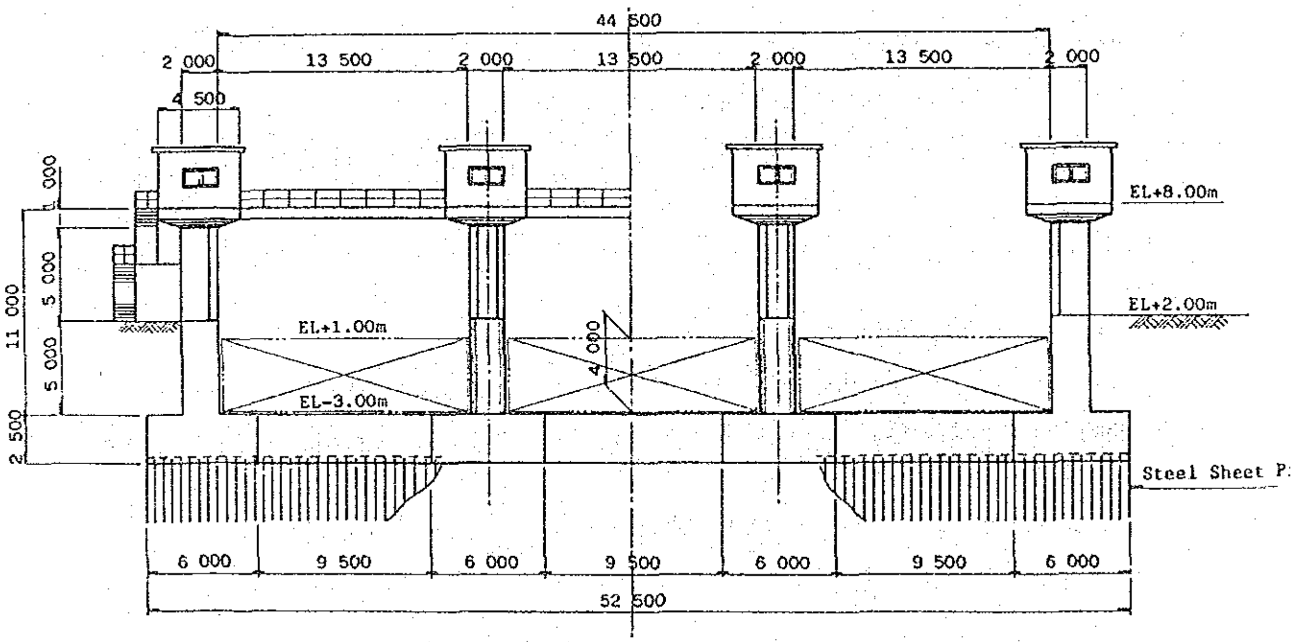
5. Tide Gate

5.1 General

In terms of salt wedge protection, the installation of a tide gate at an outlet of the Gaño Gran Estero has been proposed in Alternatives of A-I and A-II. For the convenience of construction works as well as for the course planning of main drain, the location of a tide gate has been set out at 1 km to the north-west from the present river mouth, with which cut-off course of the Caño Gran Estero will be connected.

5.2 Cross-section

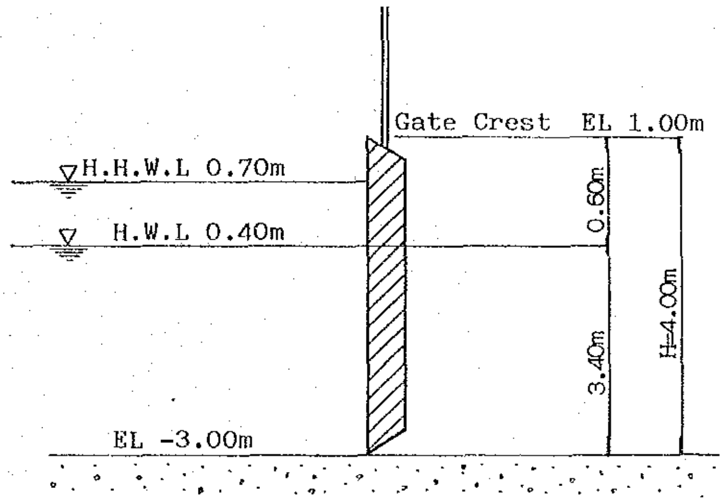
The cross section of a tide gate shall comply with that of the Caño Gran Estero. In consideration of continuous operation of a gate, three spans have been proposed.



SECTION OF TIDE GATE

5.3 Gate

Three gates with a width of 13.5 m and a height of 4.00 m will be fixed to a tide gate. In accordance with the proposed main irrigation canal, the evaluation of bed has been taken as EL -3.00 m, and that of crest as EL +1.00 m [1].



Note: 1] Maximum high tide level + free board =
0.70 m + 0.30 m = 1.00 m

6. Training Dike

6.1 General

In view of mitigating accumulation of sand at the outlet of the Caño Gran Estero, the construction of a training dike has been considered at newly opened river-mouth.

6.2 Wave Data

An Oceangraphic Atlas of the North Atlantic Ocean prepared by the U.S. Navy Oceanographic Office was consulted so as to obtain wave data of the Escocesa Bay along which a training dike will be constructed.

(1) Wave Height and Wave Direction

Wave height and wave direction classified by season are summarized in the table below.

(Unit: %)

Wave Direction	Wave Height (m)	Jan. - Mar.	Apr. - Jun.	Jul. - Sep.	Oct. - Dec.	Annual
East	1.0	22.6	30.0	33.3	23.0	27.2
	1.0 - 1.5	6.0	10.0	13.0	7.3	9.1
	1.5 - 2.0	1.0	2.3	3.3	2.4	2.3
	2.0 - 2.5	0.9	1.1	1.7	1.0	1.2
	2.5 - 3.0	0.8	0.6	1.0	0.3	0.7
	3.0 - 3.5	-	-	-	-	-
	Sub-Total	31.3	44.0	52.3	34.0	40.5
North-East	1.0	18.7	13.5	11.7	18.8	15.7
	1.0 - 1.5	6.0	4.8	6.0	8.5	6.3
	1.5 - 2.0	2.3	1.7	2.0	3.6	2.4
	2.0 - 2.5	1.2	0.7	-	1.9	1.0
	2.5 - 3.0	0.7	0.3	-	1.1	0.5
	3.0 - 3.5	0.4	-	-	0.4	0.2
	3.5 -	0.4	-	-	-	0.1
	Sub-Total	29.7	21.0	19.7	34.3	26.2
North	1.0	3.7	1.5	-	3.3	2.1
	1.0 - 1.5	1.0	0.8	-	1.4	0.8
	1.5 - 2.0	0.6	-	-	0.7	0.3
	2.0 - 2.5	0.4	-	-	-	0.1
	Sub-Total	5.7	2.3	-	5.4	3.3
Another Direction		33.3	32.7	28.0	26.3	30.1
Total		100.0	100.0	100.0	100.0	100.0

(2) Wave Height and Wave Period

Wave height and wave period classified by season are as presented below.

(Unit: %)

Season	Wave Height H (m)	Wave Period T (sec) 5 - 7	7 - 9	9 - 11	11 -	Total
Jan. - Mar.	1.2 - 1.8	17	7	0	2	26
	1.8 - 2.4	8	6	3	1	18
	2.4 - 3.0	2	6	3	1	12
	Total	27	19	6	4	56
Apr. - Jun.	1.2 - 1.8	14	3	0	1	18
	1.8 - 2.4	3	3	1	0	7
	2.4 - 3.0	1	1	1	0	3
	Total	18	7	2	1	28
Jul. - Sept.	1.2 - 1.8	14	2	2	1	19
	1.8 - 2.4	4	4	1	0	9
	2.4 - 3.0	2	2	1	0	5
	Total	20	8	4	1	33
Oct. - Dec.	1.8 - 2.4	4	3	2	1	10
	2.4 - 3.0	3	2	1	0	6
	Total	22	10	4	2	38
Annual	1.2 - 1.8	15.0	4.2	0.8	1.3	21.3
	1.8 - 2.4	4.8	4.0	1.7	0.5	11.0
	2.4 - 3.0	2.0	2.8	1.5	0.2	6.5
	Total	21.8	11.0	4.0	2.0	38.8

The average wave periods by wave height are as follows:

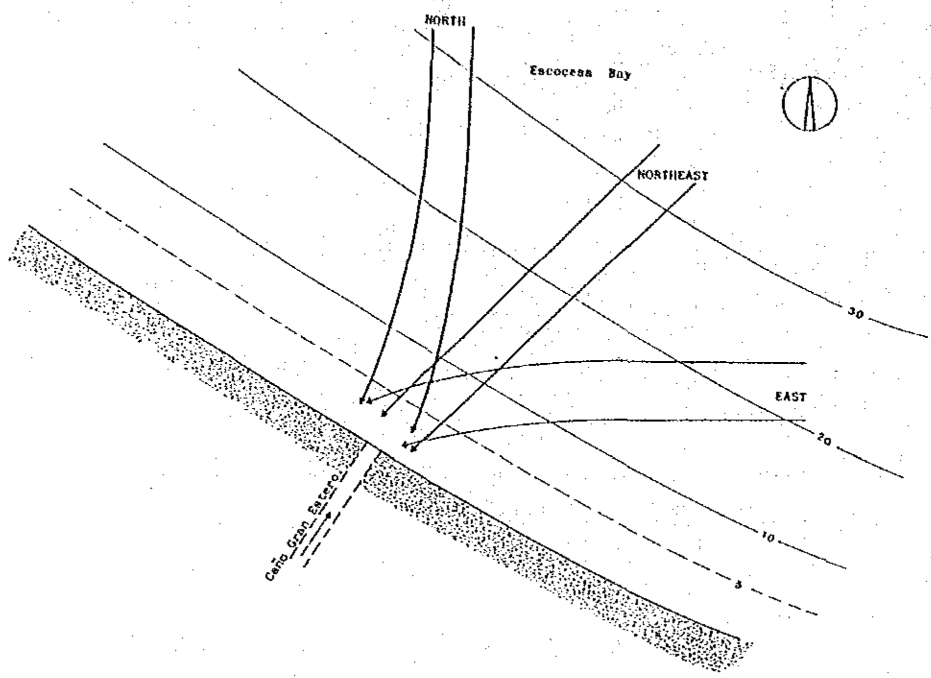
H = 1.2 - 1.8 m T = 6.8 sec

H = 1.8 - 2.4 m T = 7.6 sec

H = 2.4 - 3.0 m T = 7.9 sec

(3) Refraction Coefficient

According to the said oceanographic atlas, waves higher than 1.5 m occur at every 7-8 seconds. A refraction diagram with 8 seconds was prepared.



The refraction coefficient and the refraction angle indicated in the above diagram are as follows:

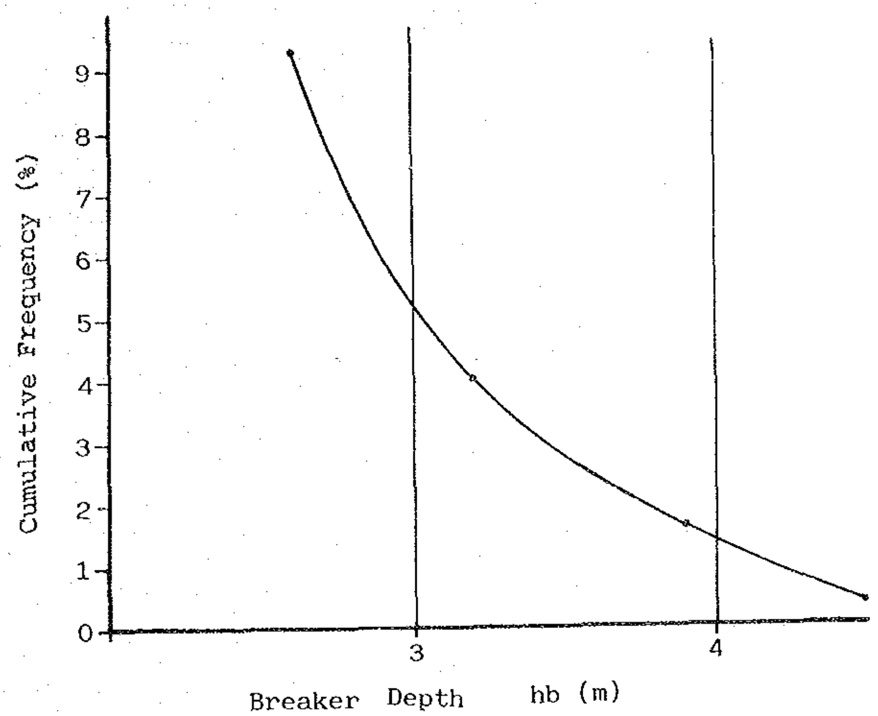
Deep water wave direction	East	North-east	North
Refraction Coefficient: Kr	1.00	1.04	0.79
Refraction Angle	ENE (75°)	NE (45°)	NNE (20°)

(4) Breaker Depth

The breaker depth by waves higher than 1.5 m and its frequency is summarized in the table below.

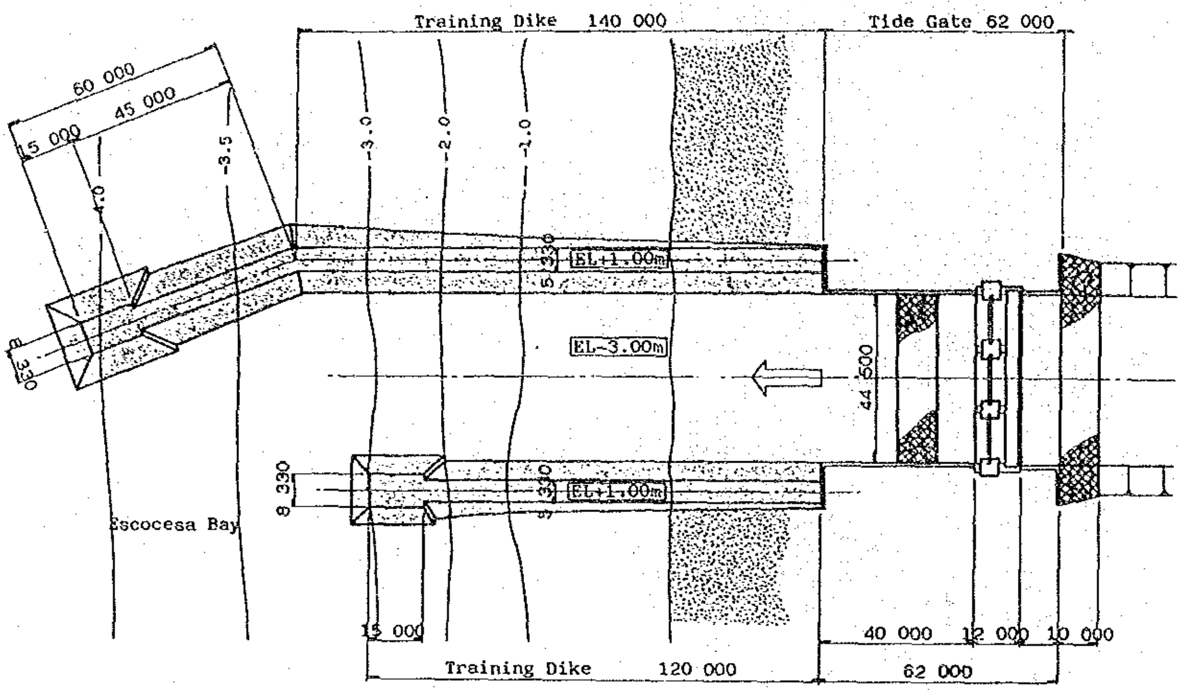
Wave Height Ho'(m)	Wave Direction ENE(75°) (%)	NE(45°) (%)	NNE(20°) (%)	Total (%)	Cumulative Frequency (%)	Ho'/Lo	hb (m)
1.5 - 2.0	2.3	2.9	0.1	5.3	9.3	0.016 -	2.6 -
2.0 - 2.5	1.2	1.2		2.4	4.0	0.022 -	3.2 -
2.5 - 3.0	0.7	0.5		1.2	1.6	0.027 -	3.9 -
3.0 -		0.4		0.4	0.4	0.033 -	4.5 -
Total	4.2	5.0	0.1	9.3			

Note: T mean = 7.7 sec, Lo = 92 m



6.3 Location of Head for Training Dike

Drift sand is frequently observed within breaker zone, therefore, the head of training dike should be located to the offshore side from the point in which more frequent breaker depth is observed. It is proposed that the head of a training dike be located where the breaker depth in the range of 3.0 - 4.0 m occures at the frequency of 1.5 - 1.0%.



6.4 Strucuture

From technical and economical viewpoint, concrete blocks designed for breakwater will be used as materials for training dike. The weight of a block will be around 6 tons.

ANNEX M: IMPLEMENTATION PROGRAM AND COST ESTIMATION

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ANNEX M: IMPLEMENTATION PROGRAM AND COST ESTIMATION

1. Project Implementation Program

1.1 Project Implementation

1.1.1 Executing Agency of the Project

Considering the administration system in the government organization of the Dominican Republic and evaluating the experiences in execution of similar type of projects, INDRHI will be justified to be an executing agency of the project because major components of the project are of irrigation and drainage facilities.

INDRHI has sufficient experiences and is competent in carrying out detailed design, construction and O/M of the irrigation and drainage facilities.

1.1.2 Construction Mode

The previous executing mode of similar projects to this in the Dominican Republic shows that most large scale of the projects were carried out in contract basis and small scale were in the force account basis. Therefore, it is considered that contract basis will be appropriate for construction of the project.

In this connection, at the implementation of the project, it is recommended the following modes be taken by INDRHI.

a) to employ an engineering consultants for carrying out detailed design including topographic survey and geological investigation, preparation of tender documents, tender evaluation and construction supervision.

b) to construct the project facilities by selected and qualified contractors and

c) to undertake O/M of the project facilities in collaboration with an organization of the concerned farmers.

1.1.3 Financing

The foreign currency portion of the Project will be financed by the international financing institute. While, the local currency portion will be appropriated by the Dominican Republic Government.

1.2 Construction Plan

1.2.1 Construction Volume

The construction volume of the Project is calculated as follows:

Item	A-I	A-II	B-I	B-II
1) Headworks	1	-	1	-
Excavation (m^3)	71,700	-	71,700	-
Struct. Concrete (m^3)	9,050	-	9,050	-
Gate (nos.)	3	-	3	-
Coffering (m)	540	-	540	-
2) Pumping Station	-	1	-	1
Excavation (m^3)	-	9,200	-	9,200
Struc. Concrete (m^3)	-	2,020	-	2,020
Coffering (m)	-	210	-	210
Pump (nos.)	-	3	-	3
Pump house (place)	-	1	-	1
3) Sub-Pumping Station	3	3	-	-
Pump (nos.)	5	5	-	-
Pump House (place)	3	3	-	-

(Cont'd)

Item	A-I	A-II	B-I	B-II
4) Irrigation Canal				
Length (m)	305,250	304,700	257,450	256,450
Excavation (m^3)	706,000	689,000	619,000	602,000
Concrete Lining (m^3)	22,000	21,600	21,500	21,000
Gate and Diversion (nos.)	125	125	105	105
5) Drainage Canal				
Length (m)	190,300	190,300	166,600	166,600
Excavation (m^3)	1,773,000	1,773,000	1,546,000	1,546,000
Dredging (m^3)	2,412,000	2,412,000	2,412,000	2,412,000
Gate (place)	2	2	1	1
6) Tide Gate	1	1	-	-
Excavation (m^3)	2,300	2,300	-	-
Struc. Concrete (m^3)	3,460	3,460	-	-
Gate (nos.)	3	3	-	-
Training Levee (m)	320	320	320	320
7) Road				
Length (m)	145,800	145,800	123,700	123,700
Embankment (m^3)	57,500	57,500	57,500	57,500
Bridge (place)	9	9	8	8
Crossing Structures (place)	32	32	29	29

1.2.2 Construction Schedule

(1) Construction Period

Construction period of all alternatives that are A-I, A-II, B-I and B-II is decided in four years span of time, taking into consideration total quantities of construction, similar scale of previous projects in the neighboring area and similar kind of construction works.

Construction seasons of each civil work are arranged so as to effect earlier the economic benefit of the project eliminating water shortage in the existing paddy field together with considering quantity and deployment of the proposed construction equipment and relationship among each civil work.

(2) Preparatory Works

Preparatory works will be conducted in first one and a half years, of which one year is for detailed design and preparation of tender documents and a half year for bidding and its evaluation. Topographic survey of a intake structure and other major structures, route survey of the irrigation and drainage canals and road networks and geological investigation of the said structures are included in the detailed design.

(3) Construction of Irrigation Facilities

Construction of irrigation facilities will be commenced in the earliest stage of the whole construction schedule and will be completed within two and a half years so that the project benefit can be obtained as early as possible after eliminating water shortage in the existing paddy field.

Excavated materials of good quality be utilized for embankment materials of main and lateral roads. While unsuitable excavated materials for embankment will be used as dressing soil for the neighboring passy field.

(4) Construction of Drainage Facilities

Construction of drainage facilities will be carried out in the latter half of the construction period of the project, that ius two and a half years following construction of the irrigation facilities. Most of the lateral drainage canals are located very closely to the construction sites of the lateral irrigatin canals. Therefore, construction of both canals will be undertaken simultaneously.

Rehabilitation of the existing drainage canals and excavation of some part of the river will be carried out by a pump dredger. To this end, a portable type of pump dredger with capacity of 400 horsepower will be selected. Excavated of training dike adjacent to the tide gate will also be carried out by the pumping dredger. Therefore, the river training must be completed before starting construction of the training dike.

Excavated materials will be utilized as dressing soil for the neighboring paddy field. Steel sheet piles used in construction of the intake structure will be converted after its completion to temporary facilities for construction of the tide gate in the Alternative A.

(5) Road Networks

Trunk road and in-farm road should be completed within one and a half years after commencmenet of construction, so that the said roads can be used as the construction and access roads. Lateral in-farm roads are constructed at same time as construction of lateral irrigation and drainage canals. Embankment materials will be selected among materials of good quality excavated from irrigation and drainage canals. The shortage of balance between embankment and excavation materials, the embankment materials will be supplied from a borrow area.

(6) Temporary Works

The access road, provision of borrow area, the contractor's camp office etc. will be made as temporary works by the contractor.

(7) Working Hours and Days

The construction works are planned to be carried out with net working hour of 7 hr/day and 25 working day/month except the earth work that will be carried out with 20 to 22 working days/month due to suspension by rainfall.

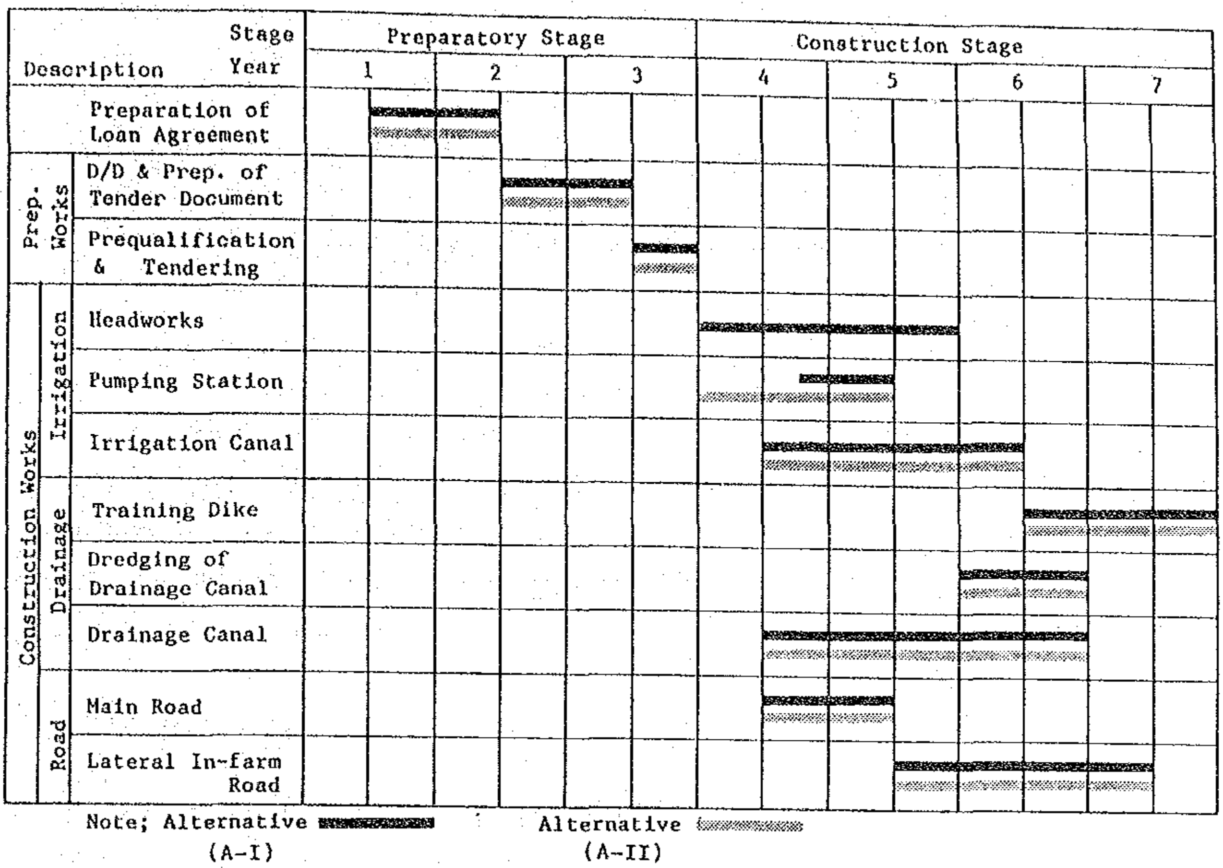
1.3 Implementation Schedule of the Project

The construction will be commenced from about three years after completion of the Feasibility Study taking into consideration the loan procedures, detailed design and tendering for contract.

The construction of headworks and irrigation facilities will be completed within about two and a half years. The in-farm road works will be commenced in parallel with the works of the lateral irrigation canals and the lateral drainage canals.

The implementation schedule for the Project is shown in Fig. M.1.1.

ALTERNATIVE A



ALTERNATIVE B

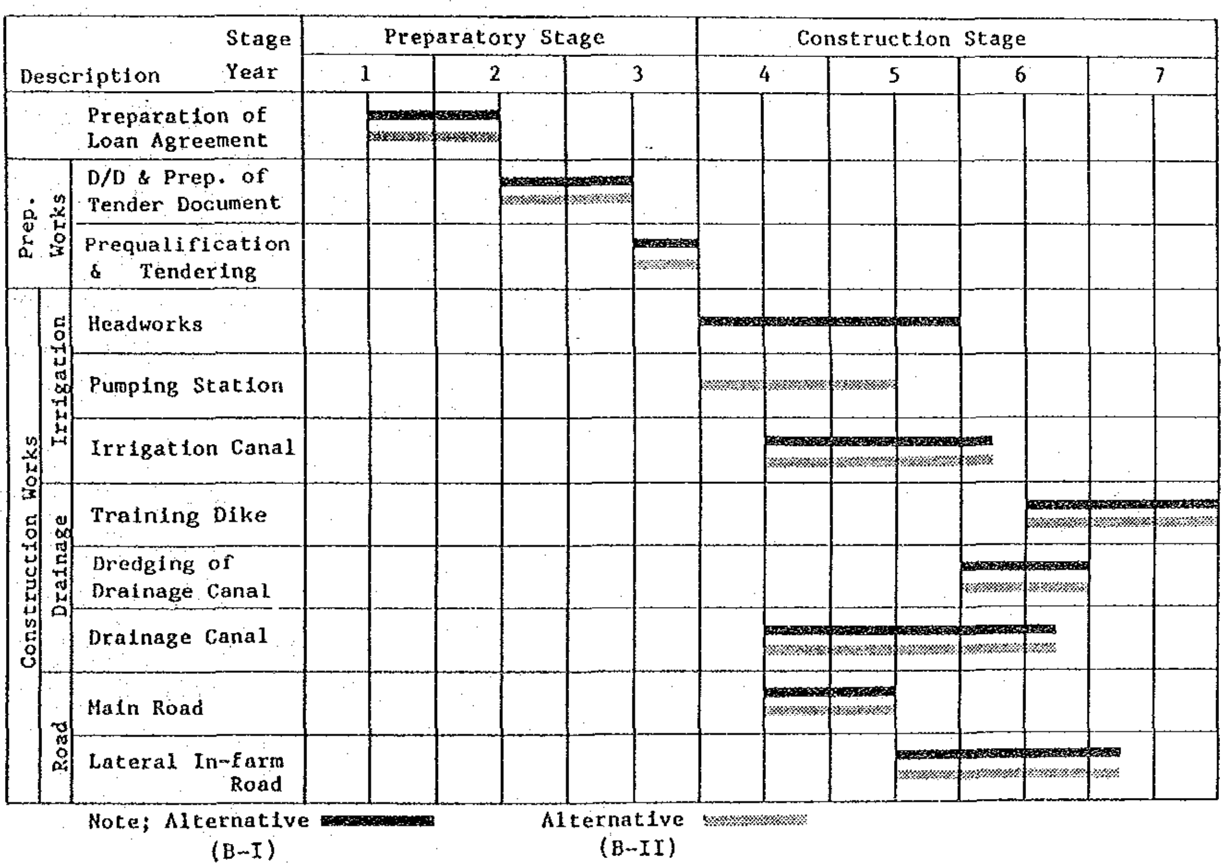


FIG. M.1.1 THE IMPLEMENTATION SCHEDULE FOR THE PROJECT