1,600 mg/l (1,600 to 2,600 mg/l as salt concentration). This high concentration of Chlorine Ion implies the tendency of salt accumulation in the well waters. Results of the water quality analyses on the irrigation well waters are summarized in Table 2.8.2 Relationships of conductivity and chloride ion, and chloride ion and total hardness are shown in Figures 2.8.2.(1) and 2.8.2.(2) respectively.

2.8.3. Wastewaters

In order to obtain the most representative data indicating the average organic and hydraulic loading pattarns of the domestic sewage from residences, two transhes were selected in Salem housing area. In the Salem area, approximately 100 to 270 persons in 16 to 48 families inhabit in an apartment building. The sewage from these apartment complexes is treated by the septic tank and discharged into ground through the transh. For the wastewater quality survey, two apartment buildings were selectd, each having approximately 16 families or 100 persons.

Tourism if one of the most important industries of El-Arish, and the number of tourists visiting the area accounts for about 18 % of the present population in the area. On account of this, the waste loadings discharged from the tourists are considered to be high in the whole waste loadings expected in the area. In view of the importance of the tourism, a wastewater quality survey had been undertaken at the sewage treatment plant of Egoth Oberoi Hotel which was the only sewage treatment plant in operation in the area as of August 1984. The sewage from the hotel is treated by the activated sludge process (Extended aeration) and some of design data are available at the hotel. For this reason, this hotel was selected for the wastewater quality survey.

The wastewater survey had been conducted simultaneously both at Salem housing complex for domestic sewage and at Hotel Oberoi for the hotel waste-water, over 24-hour sampling, from 9:00 a.m., 20th August to 9:00 a.m., 21st August, 1984. At Salem housing complex, the sampling was made for raw sewage before entering the septic tank and the effluent from the septic tank.

It was first intended to measure the sewage quantities simultaneously with the sewage quality survey, however, the apartment houses were not provided with water meters and that it was not possible to measure the exact amount of wastewater corresponding to the sampled water. In obtaining wastewater flow variation and quantities of domestic sewage, five families were selected from among the houses provided with water meters, with a total of about 100 family members. Water meters of these families were read and recorded at every hour from 6:00 a.m. to 12 p.m. on Ist September 1984, as tabulated in Tables 2.8.3.(1) through 2.8.3.(4), 2.8.4., and 2.8.5. These are also illustrated in Figures 2.8.2.(3) through 2.8.2.(13). The results of the survey may be summarized as follows:

- The variation of BOD and COD apparently follows the living pattarn of the families, gradually increasing at around 6:00 a.m. and the peak concentration occurring in the afternoon at around 3:00 p.m. The concentration then starts to decrease at around 7:00 p.m.

- The average BOD concentration was 133 mg/l, which is somewhat lower than those surveyed in other similar cities in Egypt. For example, BOD concentrations of Giza pumping station influent and Emergency Canal were 172 and 192 mg/l, respectively (Ref. No.32). The difference of the BOD and COD concentrations are probably due to the difference in the living pattarns between the cities. On account of the fact that El-Arish is rapidly developing and the improvement of living conditions which will surely increase the waste loads and it is reasonable to assume that in the near future these concentrations will reach the levels of Cairo.

- The city water contains Nitrate Nitrogen (NO_3-N) of 10 mg/l and also Kjeldahl Nitrogen, and the total Nitrogen becomes high. Some of the samples indicated a significantly high level of Nitrogen concentration, and based on this the total Nitrogen concentration is assumed to be 40 - 60 mg/l.

- The variation of T-P follows the variation pattarns of BOD and COD cr with the average concentration of 5 mg/l.

- The ratio of BOD : N : P was approximately 1 : 0.48 : 0.04. From nutrient balance viewpoint, N and P seem to be rather high side compared with those of BOD.

The results of the water quantities survey for 5 families with about 100 persons are shown in Table 2.8.4. and Figure 2.8.2.(12). As may be seen from the figure, the residents generally wake up at around 6:00 a.m. and retire to beds at around 12:00 p.m., and the water is mostly consumed during this time. During a day, three wastewater flow peaks occurred, two peaks during daylight hours at around 8:00 a.m. and 1:00 to 2:00 p.m., and the last peak at around 7:00 to 9:00 p.m. in night hours. The three peaks coincides the mealtimes, the highest peak occurring at lunch time as they generally eat the heaviest meal at lunch time at around 1:00 to 2:00 p.m., discharging about 2.3 times wastewater to the 18-hour average wastewater flows. It is also characterised that washing is generally done in the morning and as such more water is spent in the morning than in the evening.

Based on the results of the above wastewater qualities and quantities survey, the average daily per capita contribution of BOD, N and P are estimated as follows:

- BOD	8.3 g/cap/day
- N	3.1 g/cap/day
- P	0.38 g/cap/day

The per capita BOD contribution calculated on the basis of the survey results appears to be rather low as compared with the values surveyed in other similar cities; however, these surveys were undertaken in relatively short period, and therefore these values may need to be adjusted for the purpose of planning.

The septic tank effluent was relatively stabled conditions, having COD cr of 200 mg/l, total Nitrogen of 50 - 60 mg/l, and Total Phosphate of 5 - 7 mg/l. Since the anearobic digestion undergoes in the septic tank, the function of the tank often deteriorated due mainly to the flow out of the sloughed scum.

There have been observed certain differences in qualities between the hotel wastewater and the domestic sewage. The hotel wastewater has approximately twice as high BOD abd COD cr value as the domestic sewage, but at the same level of the Total Nitrogen of 40 - 60 mg/l and high Total Phosphate of 7 mg/l.

From the foregoing discussions and also the facts that the El-Arish area is rapidly developing, the living standards of the citizens will most probably reach the present level in Cairo in the immediate future, and for this reason organic loardings for the sewage treatment facilities can be based on the figure used in other similar cities in Egypt.

Table 2.8.1.(1) Quality of Drinking Water

Item / Well No. / Date	1 12/8/1984	2 12/8/1984	3 12/8/1984	6.Army 12/8/1984	7 12/8/1984	8 12/8/1984	9 12/8/1984	10 15/8/1984
Temperature (℃)	23.9	24.1	23.9	23.6	24.1	24.1	24.7	23.8
pH	6.86	7.19	7.21	7.09	7.18	7.09	7.09	7.27
Conductivity (μS/cm)	5000	5000	5020	3490	3770	3460	3670	3190
Turbidity (°)	5>	5>	5>	5>	5>	5>	5>	5>
Alkalinity as $CaCO_3$ (mg/ℓ)	167	157	129	130	136	118	147	143
Acidity as $CaCO_3$ (mg/ℓ)	19.9	17.7	14.9	15.5	17.1	11.4	16.1	15.3
Total Hardness as $CaCO_3$ (mg/ℓ)	1260	1240	1300	1030	1130	1020	1070	1020
Chloride Ion (mg/ℓ)	1190	1170	1270	684	737	675	588	516
Ammonia Nitrogen (NH_4-N) (mg/ℓ)	nd	nd	nd	nd	nd	nd	nd	nd
Nitrite Nitrogen (NO_2-N) (mg/ℓ)	0.025>	nd	nd	nd	nd	nd	nd	nd
Nitrate Nitrogen (NO_3-N) (mg/ℓ)	3.0	2.0	4.0	3.5	4.0	6.0	8.5	18.0
Coliform (colonies/100ml)	0	0	0	0	0	0	0	0
Total Bacteria (colonies/1ml)	18	33	15	2	0	67	67	110

nd; non detect

Table 2.8.1. (2) Quality of Drinking Water

Item \ Well No. / Date	12 12/8/1984	15 15/8/1984	16 15/8/1984	17 15/8/1984	18 15/8/1984	20 15/8/1984	21 15/8/1984	22 15/8/1984
Temperature (℃)	23.6	23.5	23.6	23.8	24.5	24.4	23.7	23.7
pH	7.20	7.32	7.37	7.08	7.68	7.27	7.47	7.54
Conductivity ($\mu S/cm$)	5160	5200	4210	2880	1610	2950	3360	2460
Turbidity (°)	5>	5>	5>	5>	5>	5>	5>	5>
Alkalinity as $CaCO_3$ (mg/ℓ)	127	109	108	126	100	154	115	108
Acidity as $CaCO_3$ (mg/ℓ)	15.8	9.3	8.9	11.4	5.5	11.9	9.1	6.6
Total Hardness as $CaCO_3$ (mg/ℓ)	1390	1220	1010	850	447	772	821	558
Chloride Ion (mg/ℓ)	1300	1290	911	438	230	560	764	443
Ammonia Nitrogen (NH_4-N) (mg/ℓ)	nd	nd	nd	nd	nd	nd	nd	nd
Nitrite Nitrogen (NO_2-N) (mg/ℓ)	nd	nd	nd	nd	nd	nd	nd	nd
Nitrate Nitrogen (NO_3-N) (mg/ℓ)	8.5	7.0	11.0	16.0	12.0	11.0	16.0	11.0
Coliform (colonies/100ml)	0	0	0	0	0	0	0	0
Total Bacteria (colonies/1ml)	49	18	5	28	144	1	226	105

nd; non detect

Table 2.8.1. (3) Quality of Drinking Water

Item / Well No. / Date	6. Army 5/7/1982	6. El Wady 5/7/1982	6. El Wady 3/8/1982	7. 30/8/1982	8. 5/7/1982	8. 30/8/1982	9. 30/8/1982	10. 23/2/1983
Physical Property								
Color (°)	colorless	colorless	colorless	colorless	colorless	colorless	colorless	colorless
Turbidity (°)	5>	5>	5>	5>	5>	5>	5>	5>
pH	8.0	8.0	8.0	8.0	8.0	7.9	8.1	7.3
Odor	odorless	odorless	odorless	odorless	odorless	odorless	odorless	odorless
Taste	salty	salty	salty	-	salty	-	-	acceptable
Conductivity ($\mu S/cm$)	2900	4500	2500	2800	2500	2300	3900	1050
Chemical Property								
Ammonia Nitrogen (mg/l)	0.06	0.04	0.14	0.14	0.02	0.14	0.16	1.0
Nitrite Nitrogen (mg/l)	0.001>	0.001>	0.04	0.3	0.001>	0.03	0.07	0.002
Nitrate Nitrogen (mg/l)	13.2	11.0	11.0	26.4	11.0	nd	11.0	nd
Total Dissolved Matter at 120℃ (mg/l)	2320	3348	1814	2022	2000	1744	3212	2400
Chloride Ion (Cl^-) (mg/l)	680	1180	620	570	560	530	1280	296
Sulfate Ion (SO_4^{2-}) (mg/l)	280	320	464	472	240	512	560	-
Calcium (Ca) (mg/l)	600	860	590	590	540	490	530	-
Magnecium (Mg) (mg/l)	410	640	410	500	380	430	490	-
Manganese (Mn) (mg/l)	nd	nd	nd	nd	nd	nd	nd	nd
Iron (Fe) (mg/l)	nd	nd	nd	nd	0.15	nd	nd	nd
Sodium Ion (Na^+) (mg/l)	380	650	300	660	180	260	300	-
Potassium Ion (k^+) (mg/l)	11.5	145	11.0	11.0	10.0	10.0	13.0	-
Total Hardness as $CaCO_3$ (mg/l)	1010	1500	1000	1090	920	920	1020	284
Alkalinity as $CaCO_3$ (mg/l)	140	160	140	150	132	140	150	72
Microscopic Inspection	(-)	(-)	(-)	(-)	(-)	(-)	(-)	(-)
Coliform (colonies/100ml)	0	0	0	5	0	5	9	2

Table 2.8.1. (4) Quality of Drinking Water

Item \ Well No. / Date	12.Salam 5/7/1982	15. 30/8/1982	15. 23/2/1983	16. 30/8/1982	16. 23/2/1983			
Physical Property								
Color (°)	colorless	colorless	colorless	colorless	colorless			
Turbidity (°)	5>	5>	5>	5>	5>			
pH	7.9	8.1	7.3	8.1	7.3			
Odor	odorless	odorless	odorless	odorless	odorless			
Taste	salty	salty	acceptable	-	salty			
Conductivity ($\mu S/cm$)	5000	4000	4200	3900	3300			
Chemical Property								
Ammonia Nitrogen (mg/l)	0.06	0.14	nd	0.16	nd			
Nitrite Nitrogen (mg/l)	0.001	0.006	nd	0.07	-			
Nitrate Nitrogen (mg/l)	11.0	16.0	nd	13.2	?			
Total Dissolved Matter at 120℃ (mg/l)	3436	3484	2460	3212	1900			
Chloride Ion (Cl^-) (mg/l)	1340	3474	1420	1280	986			
Sulfate Ion (SO_4^{2-}) (mg/l)	360	440	?	444	-			
Calcium (Ca) (mg/l)	720	500	-	520	-			
Magnecium (Mg) (mg/l)	730	750	-	710	-			
Manganese (Mn) (mg/l)	nd	nd	nd	nd	nd			
Iron (Fe) (mg/l)	nd	nd	nd	nd	nd			
Sodium Ion (Na^+) (mg/l)	710	600	-	560	-			
Potassium Ion (K^+) (mg/l)	9.0	14.0	-	11.0	-			
Total Hardness as $CaCO_3$ (mg/l)	1450	1250	1248	1230	920			
Alkalinity as $CaCO_3$ (mg/l)	132	110	120	110	160			
Microscopic Inspection	(-)	(-)	(+)	(-)	(-)			
Coliform (colonies/100ml)	0	0	0	2	0			

Table 2.8.1. (5) Quality of Drinking Water

Item / Well No. / Date		10. 20/10/1984	18. 20/10/1984	21. 20/10/1984	22. 20/10/1984
Physical Property					
Color	(°)	colorless	colorless	colorless	colorless
Turbidity	(°)	5>	5>	5>	5>
pH		7.4	7.4	7.4	7.4
Odor		odorless	odorless	odorless	odorless
Taste		salty	salty	salty	salty
Conductivity	(μS/cm)	1900	850	2100	1100
Chemical Property					
Ammonia Nitrogen	(mg/ℓ)	nd	nd	nd	nd
Nitrite Nitrogen	(mg/ℓ)	nd	nd	nd	nd
Nitrate Nitrogen	(mg/ℓ)	?	?	?	?
Total Dissolved Matter at 120℃	(mg/ℓ)	-	-	-	-
Chloride Ion (Cl^-)	(mg/ℓ)	220	130	240	120
Sulfate Ion (SO_4^{2-})	(mg/ℓ)	-	-	-	-
Calcium (Ca)	(mg/ℓ)	160	100	136	140
Magnecium (Mg)	(mg/ℓ)	48	36	67.2	31.2
Manganese (Mn)	(mg/ℓ)	nd	nd	nd	nd
Iron (Fe)	(mg/ℓ)	nd	nd	nd	nd
Sodium Ion (Na^+)	(mg/ℓ)	-	-	-	-
Potassium Ion (K^+)	(mg/ℓ)	-	-	-	-
Total Hardness as $CaCO_3$	(mg/ℓ)	600	400	620	480
Alkalinity as $CaCO_3$	(mg/ℓ)	250	150	130	130
Microscopic Inspection		(-)	(-)	(-)	(-)
Coliform	(colonies/100mℓ)	0	0	0	0

Table 2.8.2. Water Quality of Irrigation Wells

Item / Well No. / Date	8-4 1/9/1984	25 1/9/1984	Private 1/9/1984
Temperature (℃)	25.5	29.1	23.7
pH	7.29	8.17	7.51
Conductivity ($\mu S/cm$)	5140	6200	7900
Turbidity (°)	7	5>	5>
Alkalinity as $CaCO_3$ (mg/ℓ)	139	161	141
Acidity as $CaCO_3$ (mg/ℓ)	14.2	0	10.7
Total Hardness as $CaCO_3$ (mg/ℓ)	1080	1370	1580
Chloride Ion (mg/ℓ)	1410	1780	1940
Ammonia Nitrogen (NH_4-N) (mg/ℓ)	nd	nd	nd
Nitrite Nitrogen (NO_2-N) (mg/ℓ)	nd	nd	nd
Nitrate Nitrogen (NO_3-N) (mg/ℓ)	3.0	2.5	3.0
Coliform (colonies/100ml)	-	-	-
Total Bacteria (colonies/1ml)	-	-	-

nd; non detect

Table 2.8.3. (1) Raw Domestic Sewage Quality at Salem Housing Complex

Item \ Time		20/8/1984 11:00	15:00	19:00	21/8/1984 1:00	5:00	9:00
Temperature	(℃)	27.8	27.7	27.4	26.7	25.8	26.7
pH		7.63	7.68	7.92	8.10	7.91	8.11
Conductivity	($\mu S/cm$)	8300	6700	7600	7700	7400	7900
Alkalinity as $CaCO_3$	(mg/ℓ)	311	232	310	334	263	427
Suspended Solids	(mg/ℓ)	178	165	72.7	22.0	20.5	120
Biochemical Oxygen Demand (BOD)	(mg/ℓ)	162	199	62.7	39.9	20.5	168
Chemical Oxygen Demand ($COD_{Mn\,100C}$)	(mg/ℓ)	159	152	65.5	50.8	33.0	148
Chemical Oxygen Demand (COD_{Cr})	(mg/ℓ)	359	419	102	54.1	37.3	184
Kjeldahl Nitrogen (Kj-N)	(mg/ℓ)	19.6	28.0	87.3	86.2	14.0	78.4
Ammonia Nitrogen (NH_4-N)	(mg/ℓ)	14	12	25	28	9	64
Nitrite Nitrogen (NO_2-N)	(mg/ℓ)	nd	1.2	1.1	0.4	0.7	nd
Nitrate Nitrogen (NO_3-N)	(mg/ℓ)	10	11	12	15	12	11
Hexane Extracts	(mg/ℓ)	53.0	18.0	91.0	69.0	3.0	103
Chloride Ion (Cl^-)	(mg/ℓ)	1280	1220	1260	1280	1200	1260
Total Phosphate (T-P)	(mg/ℓ)	7.0	3.8	3.2	3.1	2.1	6.0

nd ; non detect

Table 2.8.3. (2) Quality of Septic Tank Effluent at Salem Housing Complex

Item \ Time		20/8/1984 9:00	11:00	13:00	15:00	17:00	19:00	21:00	23:00
Temperature	(℃)	27.7	28.2	29.4	28.0	28.2	25.8	27.6	27.4
pH		7.53	7.50	7.43	7.43	7.41	7.21	7.41	7.34
Conductivity	(μS/cm)	8900	8600	8800	6300	8600	8200	6300	8100
Alkalinity as $CaCO_3$	(mg/ℓ)	603	596	609	590	602	608	597	666
Suspended Solids	(mg/ℓ)	52.0	73.3	146	54.7	99.0	50.7	89.3	271
Biochemical Oxygen Demand (BOD)	(mg/ℓ)	123	182	181	116	122	201	112	225
Chemical Oxygen Demand ($COD_{Mn\,100C}$)	(mg/ℓ)	169	146	166	158	161	148	163	309
Chemical Oxygen Demand (COD_{Cr})	(mg/ℓ)	186	210	223	168	175	223	189	568
Kjeldahl Nitrogen (Kj-N)	(mg/ℓ)	56.7	65.5	57.0	58.8	56.0	52.0	64.4	55.4
Ammonia Nitrogen (NH_4-N)	(mg/ℓ)	50	54	54	56	52	50	58	54
Nitrite Nitrogen (NO_2-N)	(mg/ℓ)	nd	nd	nd	nd	nd	nd	nd	nd
Nitrate Nitrogen (NO_3-N)	(mg/ℓ)	0.1>	0.1>	0.1>	0.1>	0.1>	0.1>	0.1>	0.1>
Hexane Extracts	(mg/ℓ)	20.0	51.0	80.0	2.0	52.5	21.1	36.7	16.0
Chloride Ion (Cl^-)	(mg/ℓ)	1370	1410	1380	1400	1380	1340	1480	1440
Total Phosphate (T-P)	(mg/ℓ)	6.3	6.3	6.3	5.9	7.0	7.0	6.3	8.0

nd ; non detect

Table 2.8.3. (3) Quality of Septic Tank Effluent at Salem Housing Complex

Item \ Time		21/8/1984 1:00	5:00	7:00	9:00
Temperature	(℃)	27.5	26.1	26.1	27.2
pH		7.42	7.32	7.37	7.38
Conductivity	(μS/cm)	8200	8100	8200	8100
Alkalinity as $CaCO_3$	(mg/ℓ)	611	637	616	619
Suspended Solids	(mg/ℓ)	54.7	229	104	67.0
Biochemical Oxygen Demand (BOD)	(mg/ℓ)	113	202	200	98.0
Chemical Oxygen Demand (COD_{Mn100C})	(mg/ℓ)	156	320	252	155
Chemical Oxygen Demand (COD_{Cr})	(mg/ℓ)	189	366	270	168
Kjeldahl Nitrogen (Kj-N)	(mg/ℓ)	56.5	56.0	49.8	47.6
Ammonia Nitrogen (NH_4-N)	(mg/ℓ)	56	50	48	46
Nitrite Nitrogen (NO_2-N)	(mg/ℓ)	nd	nd	nd	nd
Nitrate Nitrogen (NO_3-N)	(mg/ℓ)	0.1>	0.1>	0.1>	0.1>
Hexane Extracts	(mg/ℓ)	18.0	4.0	6.7	10.0
Chloride Ion (Cl^-)	(mg/ℓ)	1360	1380	1410	1390
Total Phosphate (T-P)	(mg/ℓ)	5.9	5.2	6.7	5.2

nd ; non detect

Table 2.8.3.(4) Wastewater Quality at Hotel Oberoi

Item \ Time		20/8/1984 9:00	13:00	17:00	21:00	21/8/1984 1:00	9:00	20/8/1984 13:00 (Treated)
Temperature	(℃)	30.0	30.5	30.1	30.2	27.7	26.5	31.0
pH		7.56	6.03	8.01	7.09	7.13	8.00	7.07
Conductivity	(μS/cm)	3790	3690	3690	3280	2630	2580	3240
Alkalinity as $CaCO_3$	(mg/ℓ)	281	69.6	222	173	160	198	273
Suspended Solids	(mg/ℓ)	121	99.3	202	115	55.6	165	252
Biochemical Oxygen Demand (BOD)	(mg/ℓ)	375	142	360	325	200	225	116
Chemical Oxygen Demand (COD_{Mn100c})	(mg/ℓ)	237	81.8	209	229	127	378	99.7
Chemical Oxygen Demand (COD_{cr})	(mg/ℓ)	517	175	564	633	228	524	179
Kjeldahl Nitrogen (Kj-N)	(mg/ℓ)	39.7	33.6	34.7	39.8	39.7	60.5	39.2
Ammonia Nitrogen (NH_4-N)	(mg/ℓ)	26	11	12	12	22	29	23
Nitrite Nitrogen (NO_2-N)	(mg/ℓ)	nd	2.8	nd	nd	nd	nd	0.04
Nitrate Nitrogen (NO_3-N)	(mg/ℓ)	12	18	20	18	20	22	25
Hexane Extracts	(mg/ℓ)	282	145	124	10.0	97.0	142	15.7
Chloride Ion (Cl^-)	(mg/ℓ)	499	440	446	438	345	400	450
Total Phosphate (T-P)	(mg/ℓ)	8.6	8.4	7.5	4.8	2.9	11.0	4.5

nd ; non detect

Table 2.8.4. Water Consumption Variation at Salem Housing Complex

Time	① Mahmoud El Sherif	② Saloman El Gazal	③ Keder Yokoule	④ Abd Allah Nafei	⑤ Kanal Assad	Total (Whole Data) m³/hr	(Whole Data) m³/hr/100person	Total (without④) m³/hr	(without④) m³/hr/100person
6 — 7	0.0421	0.07	0.063	0.0096	0.001	0.1857	0.3641	0.1761	0.3913
7 — 8	0.0214	0.11	0.071	0.0358	0.002	0.2402	0.4710	0.2044	0.4524
8 — 9	0.0220	0.05	0.146	0.0689	0.012	0.2989	0.5861	0.2300	0.5111
9 — 10	0.0003	0.11	0.010	0.0025	0.006	0.1288	0.2525	0.1263	0.2807
10 — 11	0.0184	0.05	0.033	0.0123	0.010	0.1237	0.2425	0.1114	0.2476
11 — 12	0.0906	0.04	0.090	0.6571	0.001	0.8787	1.7229	0.2216	0.4924
12 — 13	0.0609	0.09	0.054	0.3579	0.003	0.5658	1.1094	0.2079	0.4620
13 — 14	0.0080	0.05	0.211	0.1480	0.095	0.5120	1.0039	0.3640	0.8089
14 — 15	0.0504	0.02	0.026	0.0289	0.000	0.1253	0.2457	0.0964	0.2142
15 — 16	0.0299	0.01	0.004	0.0343	0.001	0.0792	0.1553	0.0449	0.0998
16 — 17	0.0637	0.01	0.013	0.0156	0.002	0.1043	0.2045	0.0887	0.1971
17 — 18	0.0696	0.07	0.010	0.0702	0.003	0.2228	0.4369	0.1526	0.3391
18 — 19	0.1475	0.04	0.008	0.0657	0.000	0.2612	0.5122	0.1955	0.4344
19 — 20	0.0053	0.03	0.013	0.0032	0.006	0.0575	0.1127	0.0543	0.1207
20 — 21	0.0677	0.02	0.102	0.0124	0.009	0.2111	0.4139	0.1987	0.4416
21 — 22	0.0137	0.05	0.070	0.0132	0.006	0.1579	0.2998	0.1397	0.3104
22 — 23	0.0105	0.07	0.038	0.0764	0.003	0.1979	0.3880	0.1215	0.2700
23 — 24	0.0048	0.07	0.004	0.0524	0.004	0.1352	0.2651	0.0828	0.1840
24 — 6	0	0	0	0	0	0	0	0	0
Total(m³/day)	0.7268	0.96	0.966	1.6644	0.164	4.4812	8.7667	3.8168	6.2595
Adult	13	5	10	4	5	37		33	
Child	5	5	2	2	0	14		12	
Total(persons)	18	10	12	6	5	51		45	

Table 2.8.5. Organic Loadings of Domestic Sewage at Salem Housing Complex

Time	Water Consumption (m^3/100 person)		BOD		Total Nitrogen		Total Phosphate	
			Concentration (mg/ℓ)	Load (g/100 person)	Concentration (mg/ℓ)	Load (g/100 person)	Concentration (mg/ℓ)	Load (g/100 person)
6 — 7	0.3913	1.6373	168	275.1	89.4	146.4	6.0	9.82
7 — 8	0.4542							
8 — 9	0.5111							
9 — 10	0.2807							
10 — 11	0.2476	1.2020	162	194.7	29.6	35.6	7.0	8.41
11 — 12	0.4924							
12 — 13	0.4620							
13 — 14	0.8089	1.3200	199	262.7	29.0	38.3	3.8	5.02
14 — 15	0.2142							
15 — 16	0.0998							
16 — 17	0.1971							
17 — 18	0.3391	1.6462	62.7	103.2	99.3	163.5	3.2	5.27
18 — 19	0.4344							
19 — 20	0.1207							
20 — 21	0.4416							
21 — 22	0.3104							
22 — 23	0.2700	0.4540	39.9	18.1	101.2	45.9	3.1	1.41
23 — 24	0.1840							
24 — 6	0.6260	0.6260	20.5	12.8	16.0	10.0	2.1	1.31
Total	6.8855	6.8855		866.6		439.7		31.24

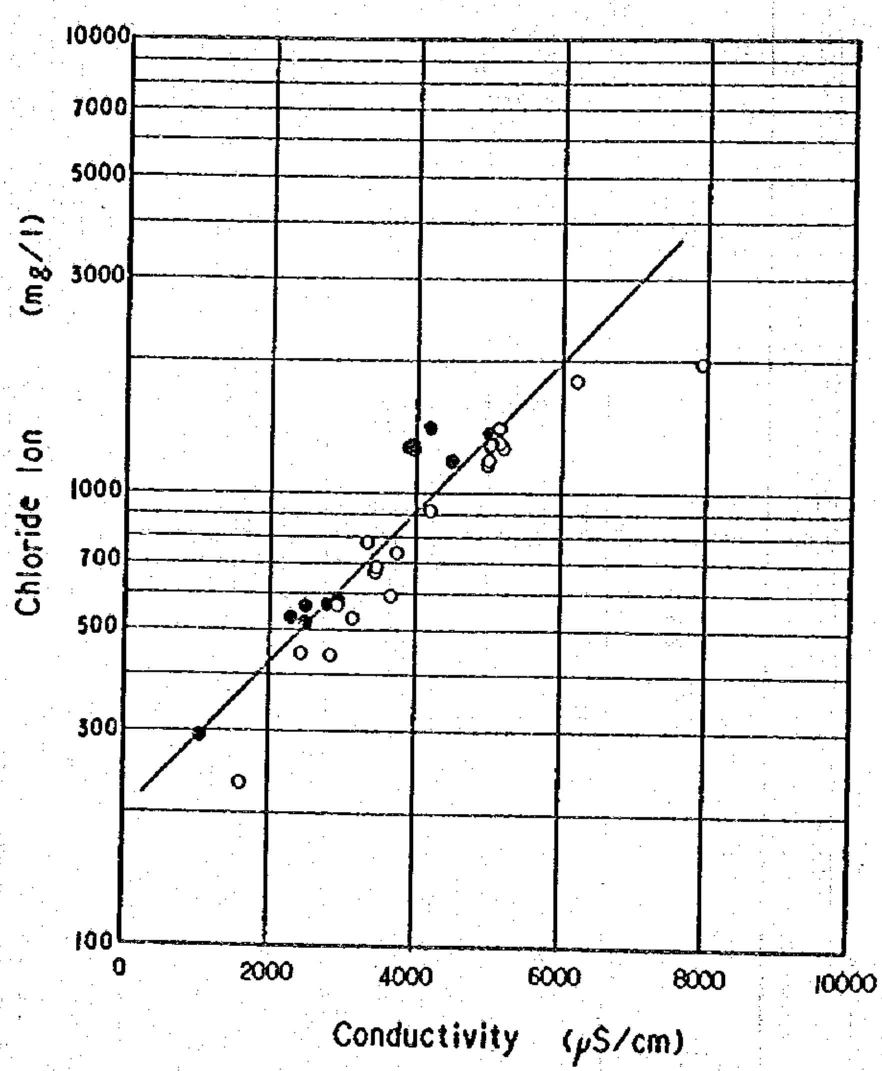


Figure 2.8.2.(1) Relationship of Conductivity and Chloride Ion

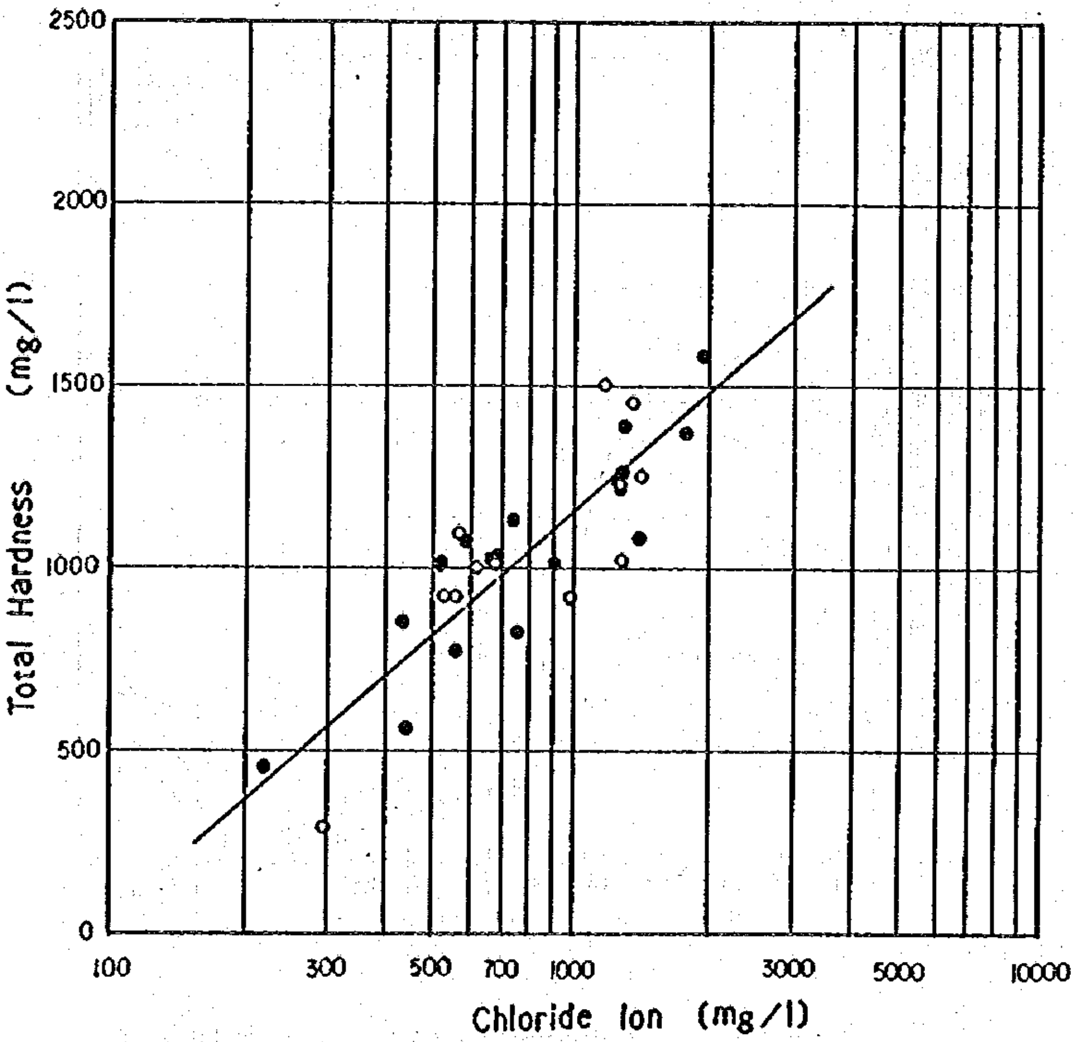


Figure 2.8.2.(2) Relationship of Chloride Ion and Total Hardness

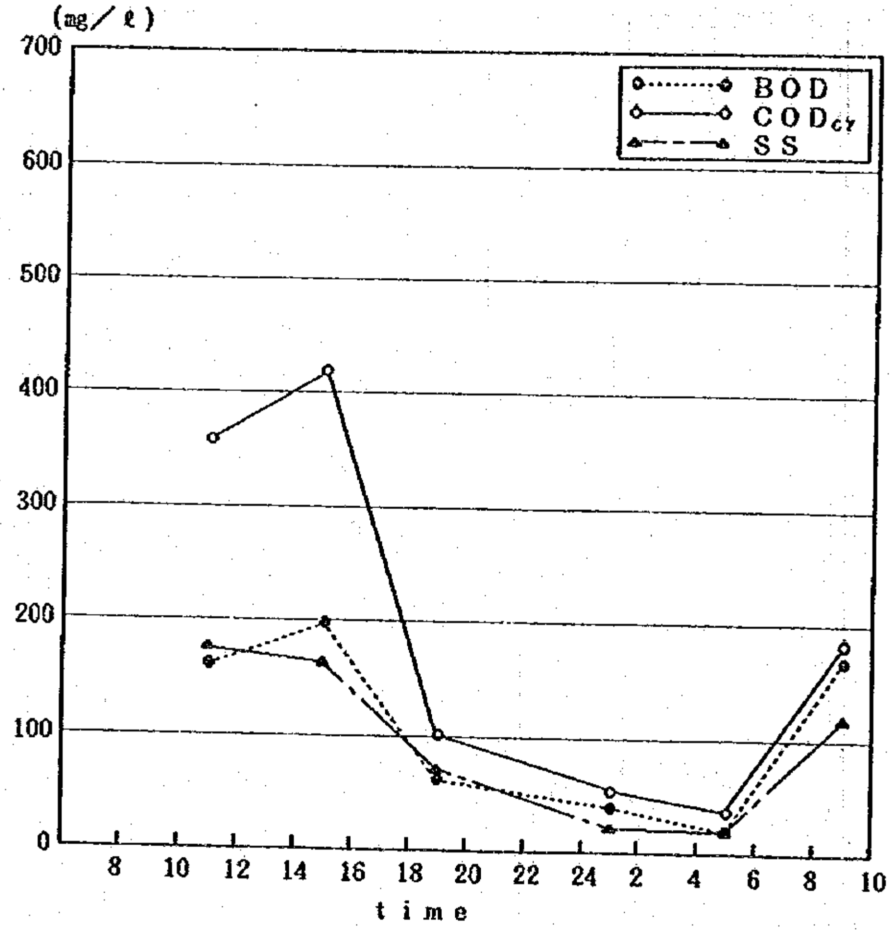


Figure 2.8.2.(3) Daily Variations of Domestic Sewage BOD, COD and SS

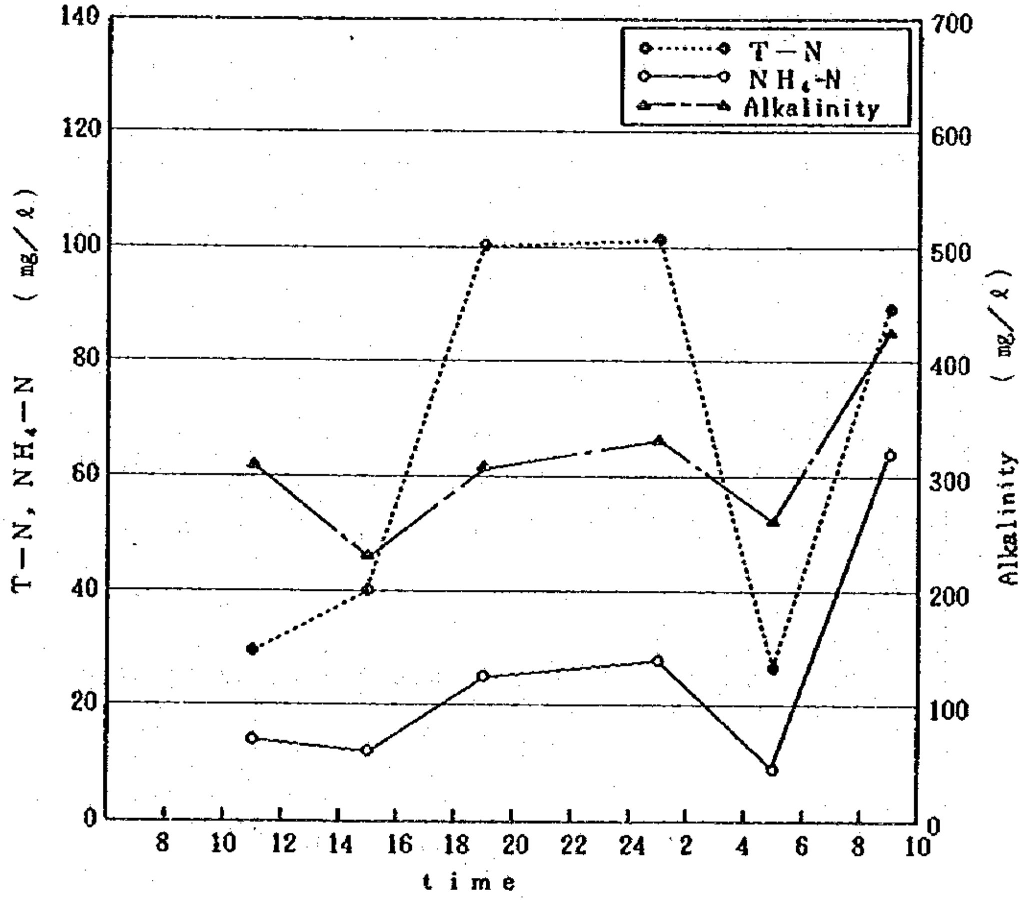


Figure 2.8.2.(4) Daily Variations of Domestic Sewage T-N, NH_4-N and Alkalinity

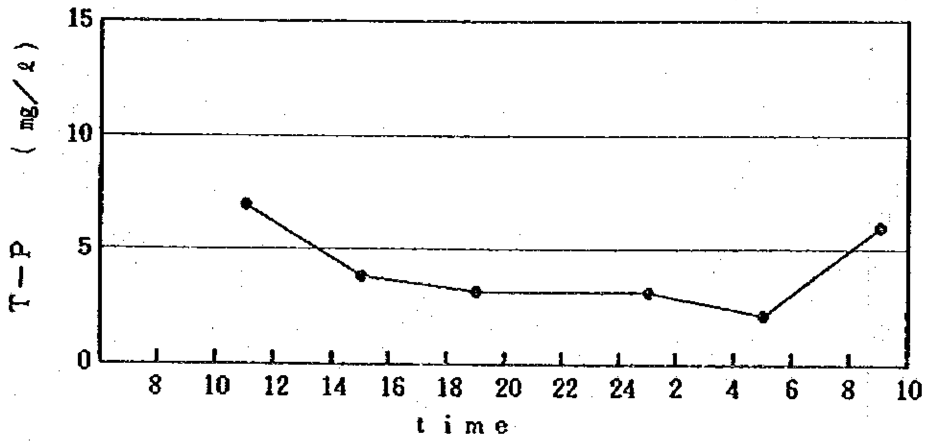


Figure 2.8.2.(5) Daily Variations of Domestic Sewage T-N

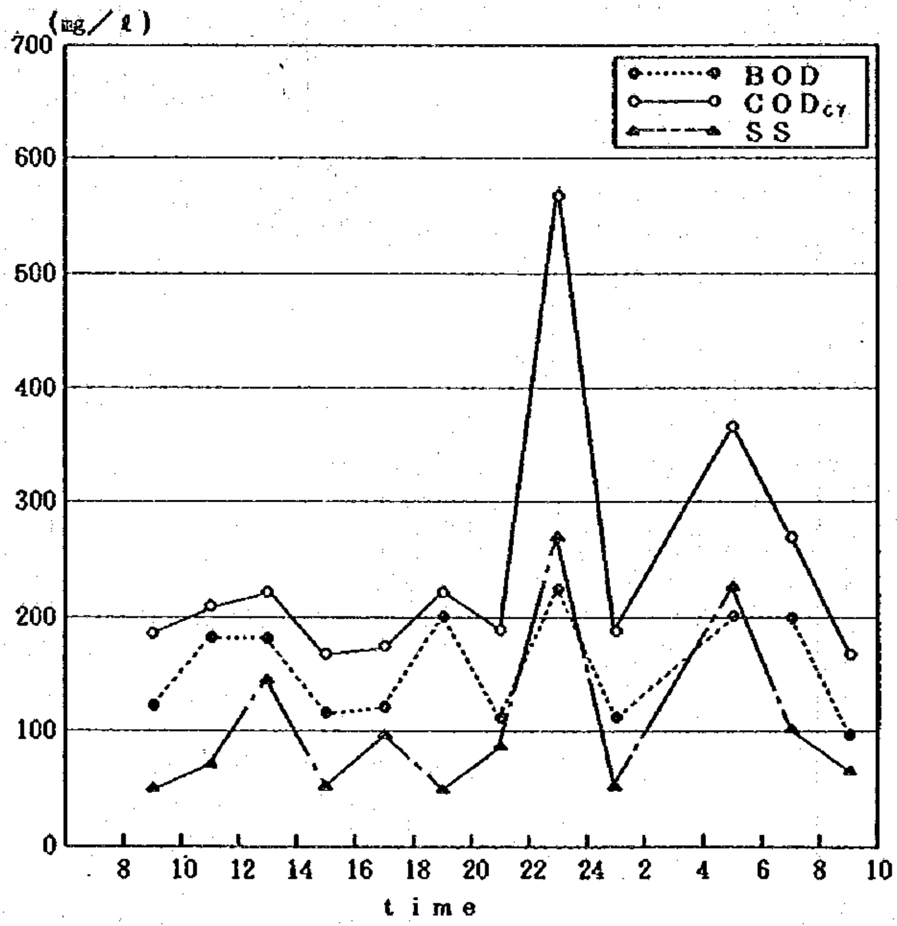


Figure 2.8.2.(6) Daily Variations of Septic Tank Effluent BOD, CODcr and SS

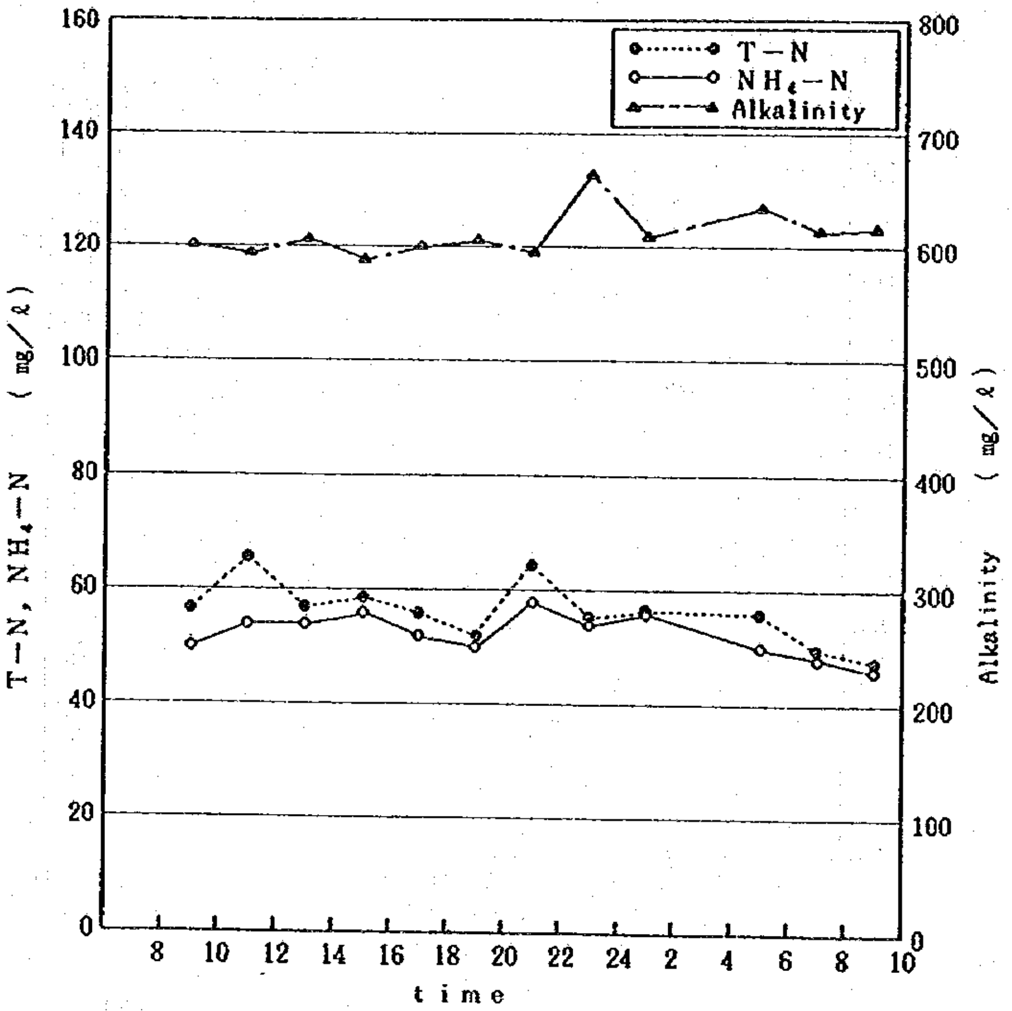


Figure 2.8.2(7) Daily Variations of Septic Tank Effluent T-N, NH4-N and Alkalinity

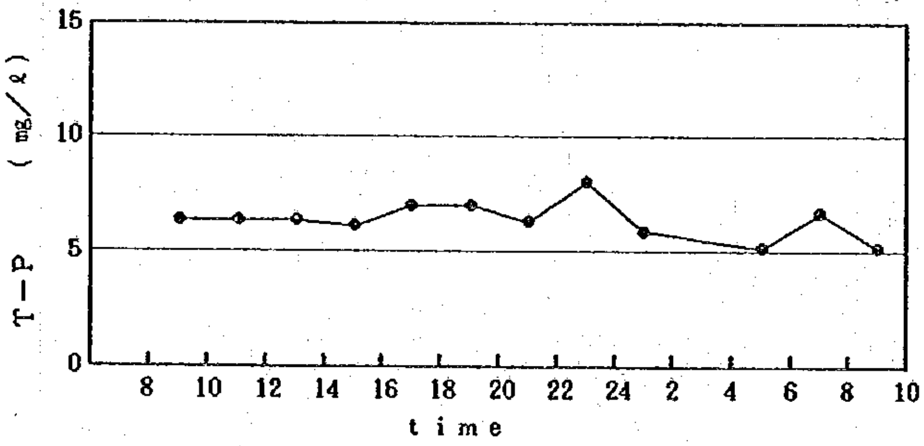


Figure 2.8.2.(8) Daily Variations of Septic Tank Effluent T-P

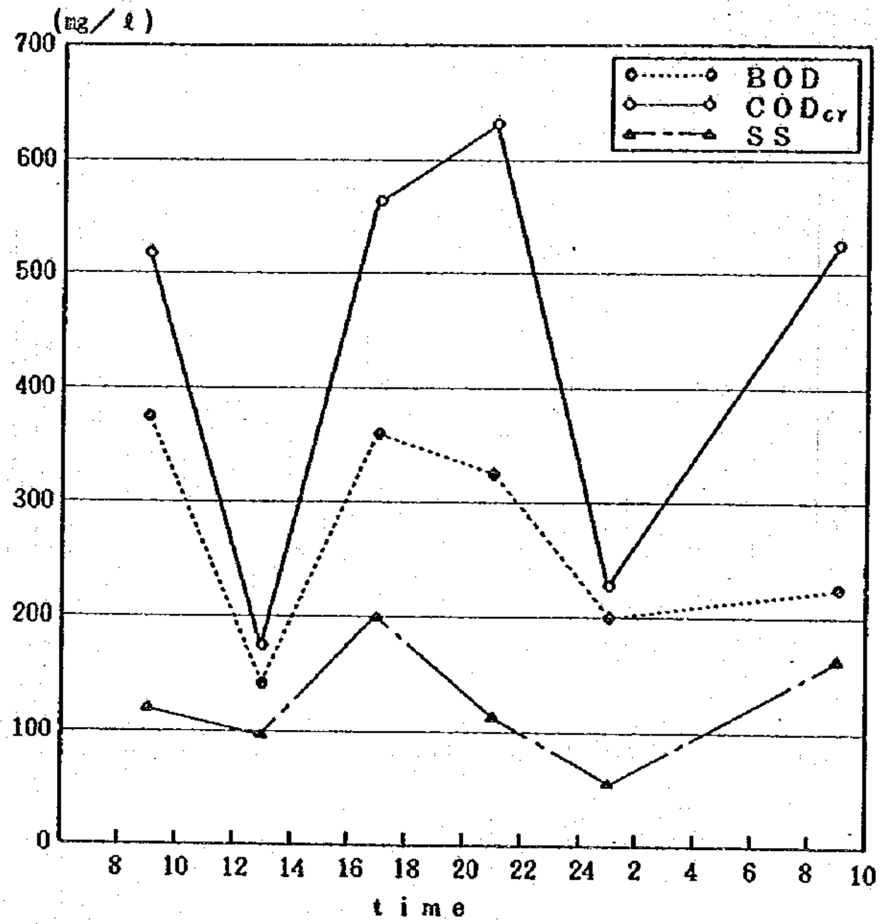


Figure 2.8.2.(9) Daily Variations of Hotel Wastewater BOD, CODcr and SS

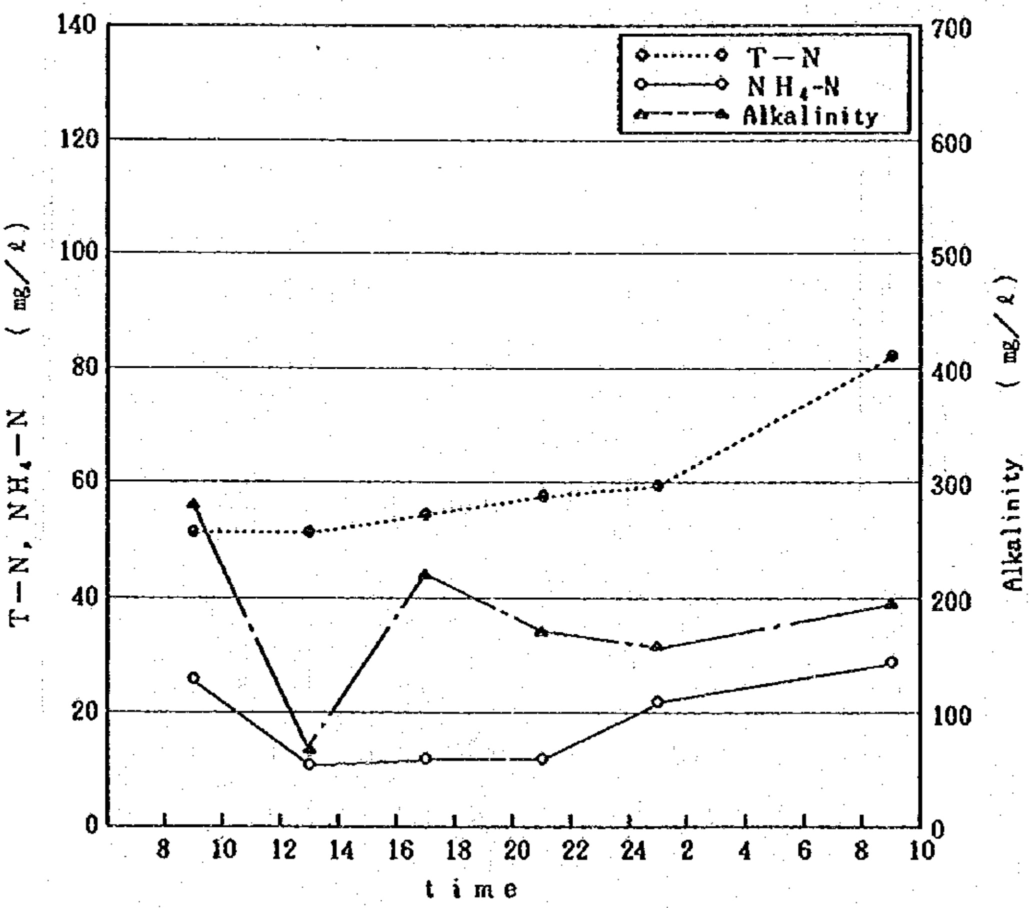


Figure 2.8.2.(10)　Daily Variations of Hotel Wastewater T-N, NH4-N and Alkalinity

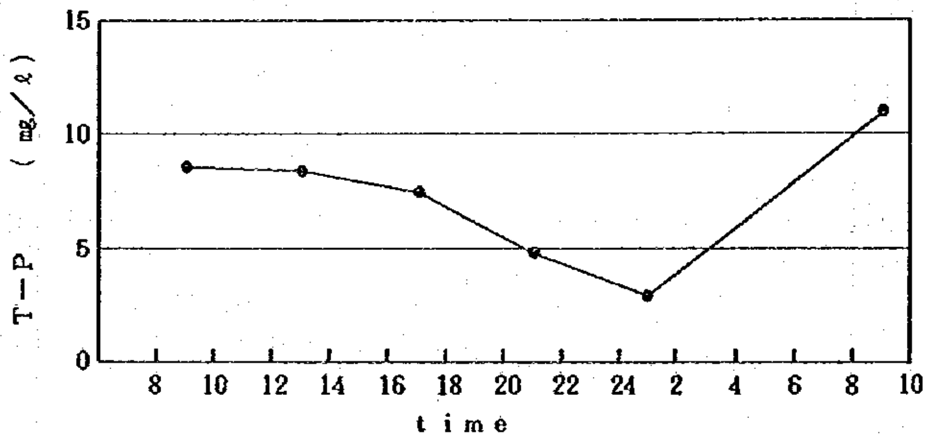


Figure 2.8.2.(11)　Daily Variations of Hotel Wastewater T-P

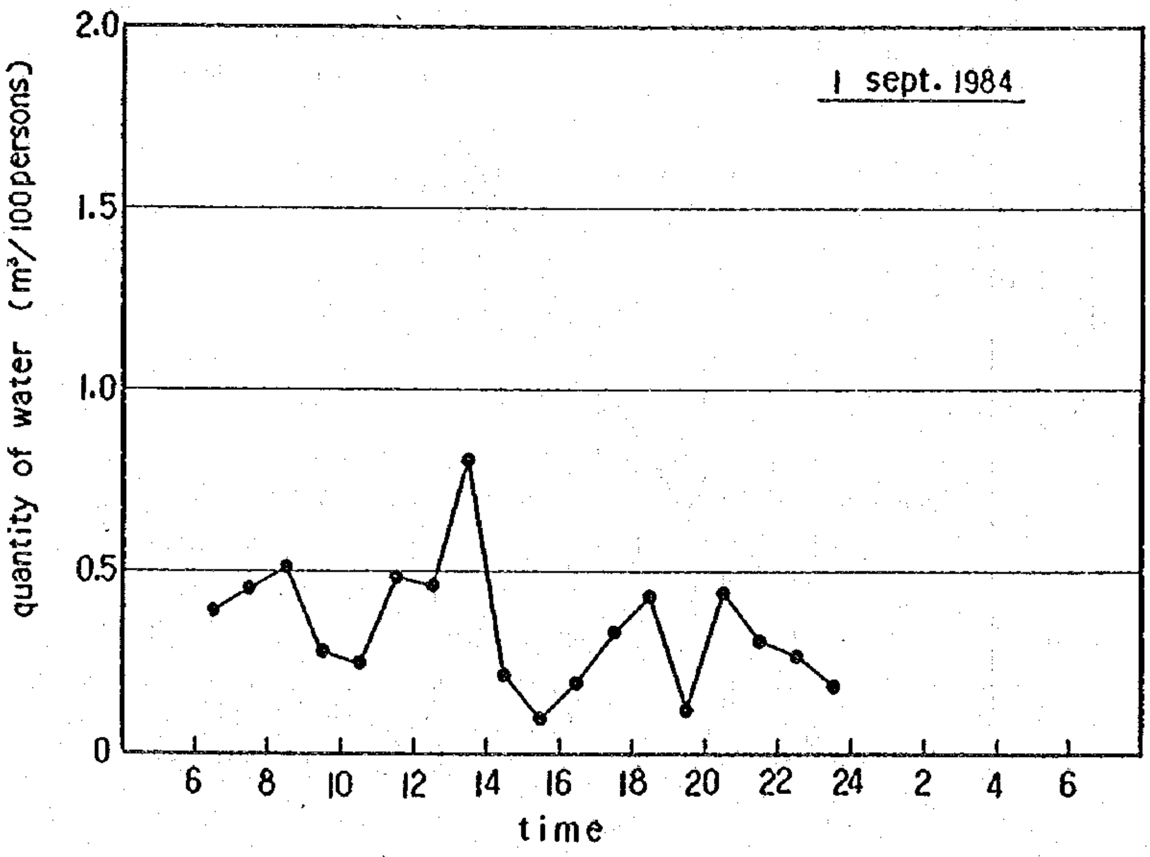


Figure 2.8.2.(12) Daily Household Water Use

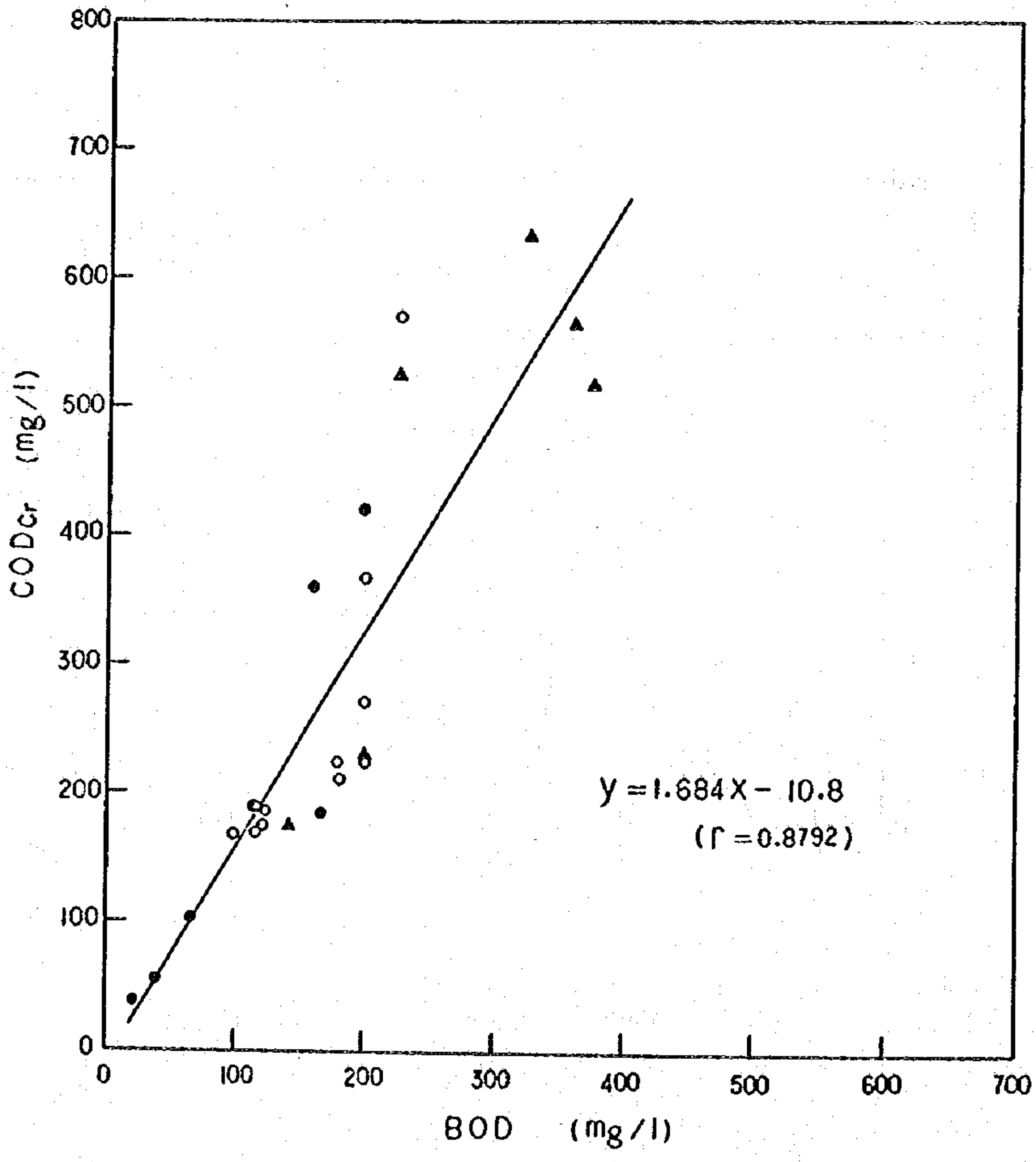


Figure 2.8.2.(13) BOD versus CODcr

2.9. STORMWATER DRAINAGE SYSTEM

2.9.1. Flood Problems and Drainage Requirements

As described in Subsections 2.5.2. and 2.5.3. of this volume, the region has low precipitations and rainstorms of more than 10 mm daily intensities occurred only 3.5 times a year. During the dry period from June through August, there has been no recorded rainfalls in El-Arish area. Most of the precipitations infiltrate into ground because of high permeability of the soil, and in general no noticeable overland flow of rain water has occurred.

Because of the relatively low precipitations coupled with the soil characters, stagnation of rainfall runoffs has not been serious problem in the past, except at some places in low-lying districts where the land is depressed and the proper drainage facilities are not provided.

The Wadi El-Arish runs through the city area and is the major topographic feature of the Sinai Peninsula. The tributary area of the Wadi is approximately 27,000 km2 or almost of all the peninsula, collecting rain storm runoffs from such wide area. Under the normal rainfall conditions, however, rainfall runoffs do not reach El-Arish because of high evapolation and permeability of the soil. In every 5 or 6 years, there have been flooding of the Wadi reaching the river mouth and caused severe damages in the city particularly at the low-lying area s close to the Wadi.

Estimates have been made of total flood quantities occurring at Rawafaa Dam on the lower Wadi El-Arish for 1948, 1950, 1951, and 1965/1966 seasons, as shown in Table 2.9.1. According to the estimates, the maximum amount of flood flow was 21 million m3 as observed in March 1947. Although no accurate information is available to indicate the flood damages, significant damages had been caused by the floods in the districts within about 100 metres from the Wadi, breaking down many of the houses by the high water levels of the Wadi.

Table 2.9.1 Reported Floodwater Quantities Flowing in the Lower Part of Wadi El-Arish

Date	Magnitude of Flood(Torrent)	Annual Rainfall in El-Arish (mm)
Oct. 1925	Very strong	114
Dec.11928	Strong	64
Dec. 1930	Strong	110
Oct. 1931	Medium	69
Dec. 1933	Strong	82
Oct. 1935	Strong	37
Oct. 1937	Very strong	123
Oct. 1938	Medium	126
Oct 1940	Medium	94
Dec. 1942	Strong	98
Dec. 1943	Weak	121
Jan. 1945	Very strong	-
1945	7 days continuous flow at Mitmenti (a)	-
1948	21×10^6 m^3 (b)	87
1950	$18 \times 10^6 m^3$ (b)	127
1951	$\sim 3 \times 10^6 m^3$ (b)	45
1953	$\sim 800,000$ m^3 (c)	126
1954 - 1964	Information missing	-
1965/1966	$>1 \times 10^6$ m^3 (b)	193/63
1967 - 1974	Information missing	-
1975	Large flood at El-Arish	-
1976 - 1979	Information missing	-
1980	Large flood at El-Arish	-

Source: pp. B-33, Volume VII, Sinai Data Book, Sinai Development Study, Phase I, Draft Final Report.

Note: (a) Observed by Dr. A. Shata.

(b) At Rawafaa dam.

(c) The quantity reaching Wadi El-Arish from Wadi Hareidin.

2.9.2. Physical System

In low-lying areas in El-Arish City, stormwater pipes have been provided by the City Council to discharge the surface runoffs of rainstorm to the Wadi by gravity. However, no comprehensive drainage planning system planning has been developed so far to cover the entire city area, and for this reason, stormwater runoffs stagnated at many places and iterrupted public traffics during the heavy rainstorms.

When the roads in low-lying area are flooded, manhole covers of the drains are opened so as to discharge the stagnated surface water into the manhole. Where it is difficult to provide drain pipes to discharge the stagnated water due to traffic or topographic reason, a transh is installed to allow the water to infiltrate into the ground. When the stormwater floods, the cover of the transh is open and lead the water to the stormwater transh.

Many of the existing drains are clogged by either sand, debris, or sludge, preventing smooth flow in the pipes. In view of this, these pipes are cleaned by vacuum trucks or other means before rainy season starts. Since it is difficult to remove the sand deposit in the drains by the vacuum trucks at many locations the pipes are choked and preventing the smooth flow in the pipes. At many locations, house-hold sanitary pipes are illegally connected to the stormwater drains. Presently, collected stromwater is discharged to the Wadi through five outlet facilities. All of the five outlets are more or less clogged by the accumulated sand and debris. Of the five outlets, two systems are receiving the discharge from the central part of the City and are provided with the transh system to alleviate the sand intrusion into the drain pipes. At every manholes in the drainage system, a sand trap is provided to prevent the sand from entering the system.

Existing drainage facilities and locations of the improved areas by the provision of drainage system are shown in Figure 2.9.1.

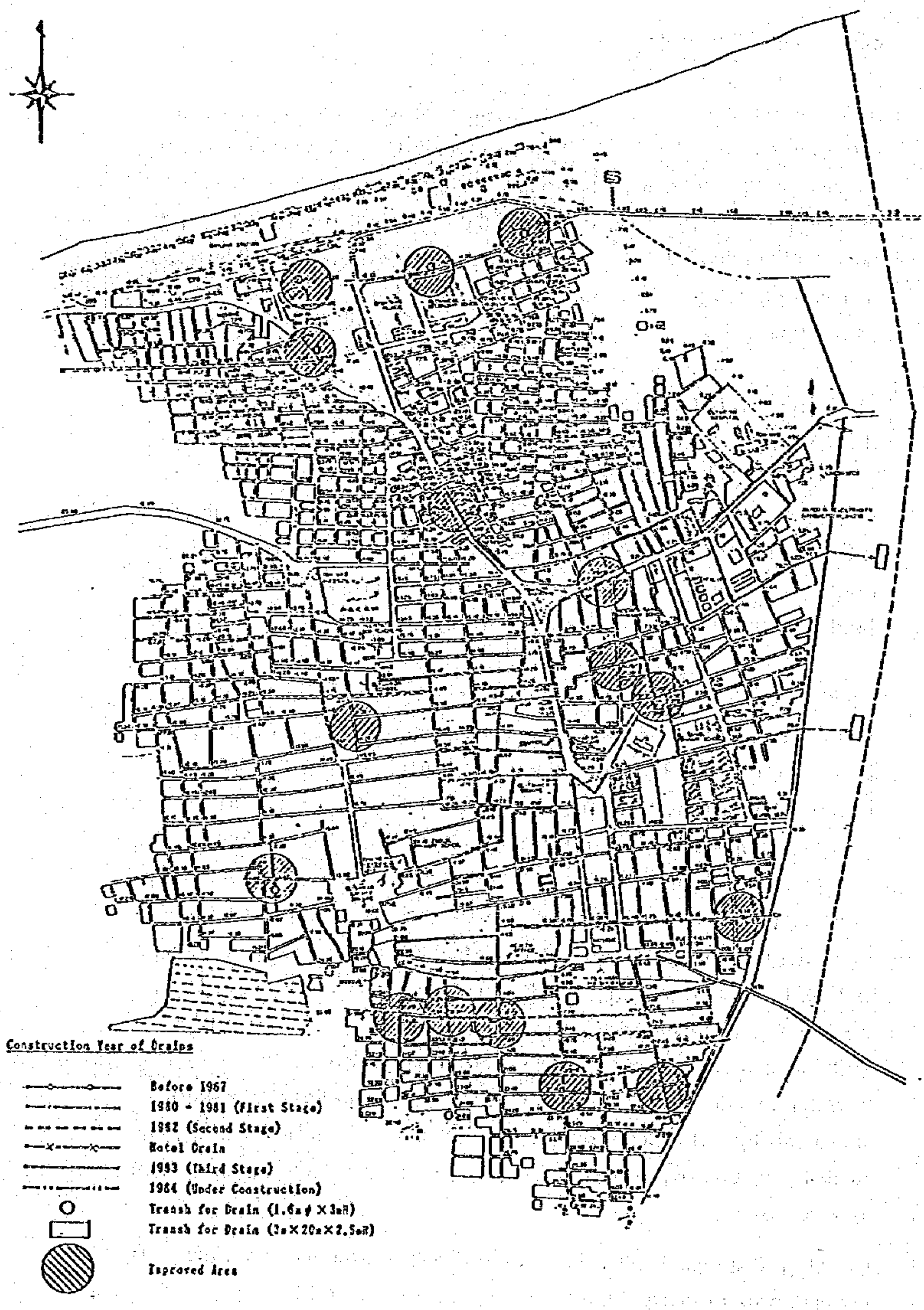


Figure 2.9.1. Existing Stormwater Drainage System in El-Arish

2.10. PUBLIC HEALTH CONDITIONS

2.10.1. Medical and Health Services

A general hospital of the North Sinai Governorate has been in service in El-Arish as the centre of medical and health services for the people in the region. According to the information by the hospital, only this general hospital provides a full range of medical services concentrating more unusual cases of ill health; however, a plan is now under consideration to provide three new general hospitals in the region.

The main hospital is provided with modern facilities and equipment for medical treatment, operation and examinations sufficient to take care of most of the work required. The hospital has a total of 140 beds as of August 1984, which are planned to be increased to 200 within August 1984. The services have been provided by 50 doctors and 6 physicians, assisted by 55 permanent nurses and 25 part-time nurses despatched from Cairo, to examine and treat daily an average of 700 out- and in-patients. A central laboratory is attached to the hospital, however, the laboratory has no equipment for water quality analysis.

Under the jurisdiction of the main hospital are 40 health centres and and travelling clinics to take care of the people in isolated villages and towns in the region. Private health care services, theoretically an alternative to public services, are little developed in the region and do not seem probable.

2.10.2. Water-borne Diseases

Information were obtained from the main hospital with respect to the number of water-borne diseases in the region. According to the information, the number of such diseases as cholera, typhoid fever and dysentery in the region were annually only few cases and are ignore-ble level so far as the sewerage planning is concerned. This low level of the incidences of water-borne diseases may be attributable to the present transh system through which all sullage and excreta are directly discharged into the ground without a chance of direct contact with human.

2.11. OTHER PUBLIC FACILITIES

2.11.1. Roads

Roads in El-Arish are classified into two categories depending upon the purpose for the use and scale as follows:

(a) Outside roads; connecting cities, towns and villages.

(b) Inside roads; comprising main roads in the City and second class roads common in the City.

The outside roads have 7.5 metres wide pavement and 50 metres wide reserved land for the future expansion. The total thickness of the pavement is 36 cm, consisting of 5 cm thick surface asphalt finish, base asphalt layer of 6 cm thick, and a gravel layer of 25 cm thick. The outside roads are under the control of the General Authority of Road and Bridge, which plans, constructs and maintain these roads.

The width of the main inside roads in the City ranges between 21 and 25 metres. These are further classified into two types, two-lane and four-lane, of which the four-lane roads are provided with a central separation belt and footpaths of 2 to 3 metres wide. The pavement is 30 cm thick, comprising a 5 cm thick surface asphalt layer and a gravel foundation of 25 cm thick.

The second class inside roads have widths ranging from 6 to 10 metres, having the pavement of 6 metres wide and footpaths of 1 to 2 metres wide. The pavement of these roads is the same as that for the main roads, but most of the existing roads are still unpaved.

The Management Road and Transportation of N.S.G. is responsible for the inside roads planning, construction and maintenance. Locations of the existing outside and inside roads are shown in Figure 2.11.1.

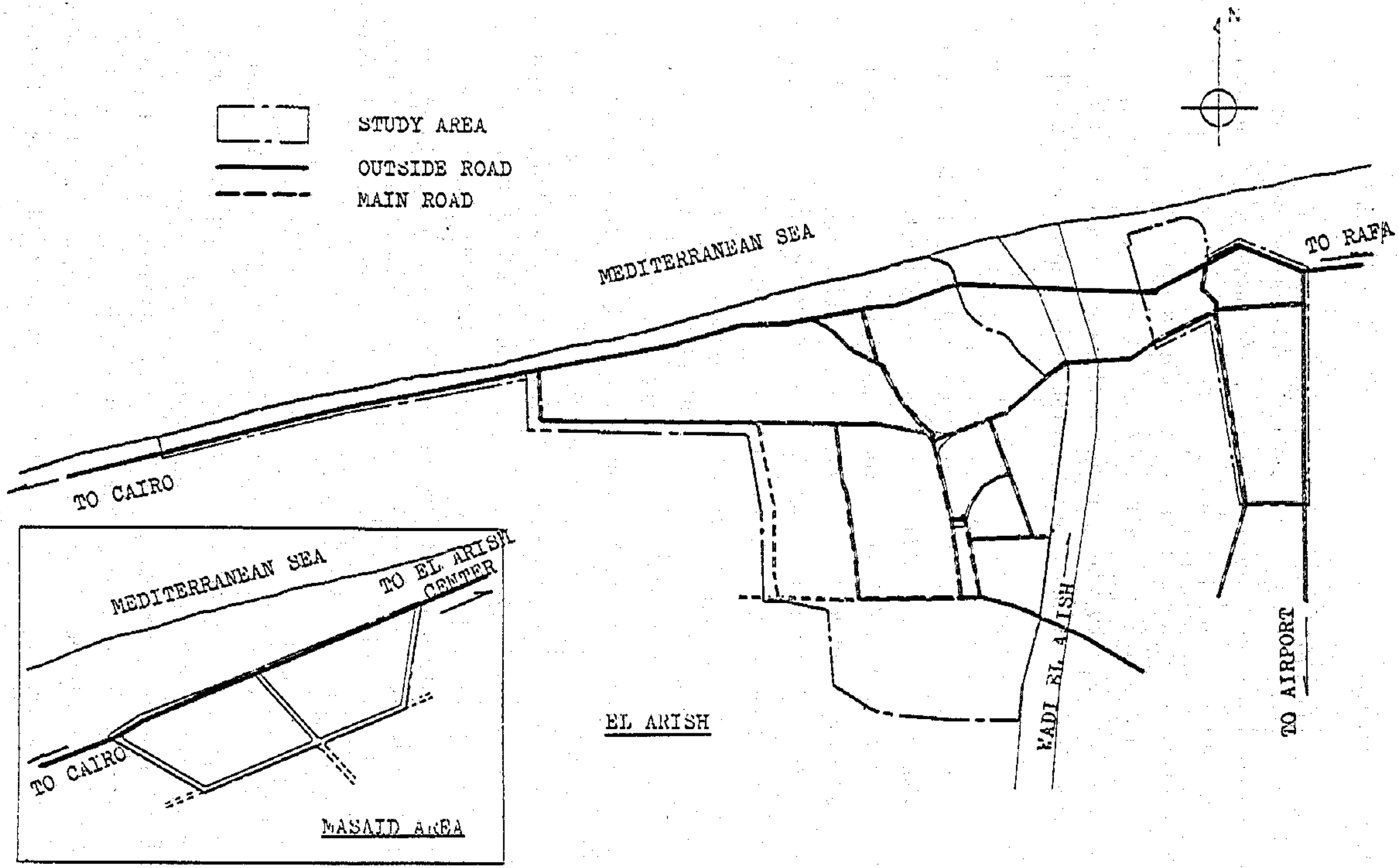


Figure 2.11.1. Locations of Major Roads in El-Arish

2.11.2. Electricity

Canal Electricity Distribution Company (CEDCO) is responsible for development and operation of electric power generation, transmission and maintenance in Sinai. All the demands in Sinai are met by electricity generated in Sinai and no electricity is transmitted from the national grid into Sinai. According to the information by CEDCO (Ref. No. 6), an average 35,000 kWh daily electricity was consumed by a total of 10,280 houses and buildings in 1981. This accounted for about 80 per cent of the total electricity consumption in Sinai in the same year.

In accordance with the first phase of a plan by CEDCO prior to the Israeli withdrawal from east Sinai, diesel and gas turbine generators were being installed in principal settlements. Large gas turbine generators using natural gas would be installed in a second phase. In the third phase, Sinai will be linked to the national grid by establishing a 220-kV network and matching transformer stations in the region.

Currently, the maximum generating capacity of power plants in El-Arish is 7,000 kW according to the infomration by the Electric Authority. A plan is now underway to add 16,000 kW capacity within 1984, thus the total rated capacity of power stations by the end of 1984 will be 23,000 kW. The capacity is further planned to increase to 29,000 kW in 1986, 58,000 kW in 1988, and 120,000 kW by 1990. Electric power generators presently available for El-Arish City are 2 units of 7,500 kW and 2 units of 3,200 kW, with a total available power of 9,000 kW, according to the information.

2.12. PREVIOUS REPORTS AND STUDIES

Reports and studies, important to this study, have been available on several aspects of Egypt, Sinai and El-Arish, which are crucial to preparing the strategic plan and identifying the proposed project. Those of most significance to this study are discussed here. In addition, various other valuable information and reports used for this study are listed at the end of this report. References are made as needed throughout the body of the report by reference number.

"Feasibility Study for Sewerage Scheme and Re-Use of Treated Sewage for the City of El-Arish," North Sinai Governorate, prepared by KUP Engineer Consult, July 1983.

This feasibility study carried out various surveys and studies and proposed a comprehensive sewerage, drainage and sewage treatment plant effluent reuse systems for El-Arish City and its environs. The study covers fundamentals required for the systems planning, preliminary engineering design of the facilities, and feasibility study on the proposed systems. The proposed systems, planned for the year 2020, consist of a conventional activated sludge process, effluent reuse facilities at Jarada, intermediate pumping stations, sewer reticulations, and stormwater drains, at the total estimated construction cost of L.E. 12.87 million for the first stage programme up to the year 1988. The total sewage to be treated and reused was estimated to be approximately 20 million m3 per year.

"Sinai Development Study, Phase I, Draft Final Report," by Dames and Moore in association with Industrial Development Programme SA, submitted to the Advisoy Committee for Reconstruction, Ministry of Development, Arab Republic of Egypt, June 1983.

In November 1980, the Ministry of Development authorized the start of the Sinai Development Study - Phase I, as a step towards preparing a preliminary development strategy for the peninsula. The study was carried out under the guidance of the Advisory Committee for Reconstruction through a specially appointed Steering Committee. The report of this study is presented in seven volumes, giving findings, the process of synthesis, an various other elements , through which various specialized studies were brought together into a single recommended strategy for development. The strategy is a closely woven bundle of ways and means of establishing within one generation a permanent six-fold increase of the current population (to about one million) of the Sinai peninsula in such a way that the residents of Sinai are integrated with Egyptian society and well distributed throughout the peninsula.

Seven arrays of studies were carried out under the programme:

- Natural resources
- Social
- Economic
- Infrastructure and services
- Management/administration
- Data base
- Research and development/projects

"Detailed Frame of the Five Year Plan for Economic and Social Development 1982/83 - 1986/87," Ministry of Planning, Arab Republic of Egypt, December 1982.

Primary objective of the Five Year Plan for Economic and Social Development is to establish short- and long-term plans for futuristic studies and prognosis whichcover the period until the end of the century. These studies are aimed to make clear the pace of economic and social development from 1982/83 to 1986/87, in which GDP is projected to increase 8.1 per cent per annum and expected per capita of L.E. 573 in 1986/87 at the constant price of 1981/82. The conceptual quintessence of the Plan consists in the stable improvement of the living standard together with the increasing efforts for self-sustaining economy.

The Plan also emphasizes to solve the invariable deficit of the balance of payment even as the goal for the Plan. The total cost of public debt service, interest plus installment, has reached L.E. 1.6 billion in 1981/82. The Plan also aims to increase export at the base of 10.5 per cent of commodity. With projection for surplus in the service balance and in the factor income balance, the Plan expects a gradual decline of the deficit in the balance of payment from L.E. 2 billion in 1981/82 to L.E. 0.5 billion in 1986/87. The Government role would be symbolically recognized in social services. A total of 68.8 per cent of the social services is shared with the Government services which means 13.2 per cent for the total GDP in 1986/87.

2.13. NEED FOR A PROJECT

At present, El-Arish and its suburban areas have no comprehensive modern sewerage system except for Masaid housing development area, and all the domestic and commercial wastewaters are being discharged directly into soil through the transh system without receiving any treatment.

Areas dependent on soakaway drainge for wastewater disposal are very likely to have problems if water use is significantly increased, since the maximum amount of water that can be disposed of in this way is limited by the rate of seepage into the ground. Depending upon the density of population a maximum seepage rate from soakaways into the underlying soil of between 25 and 100 lcd may be possible. This may be considerably less than the rate of water consumption of an urban household with an individual water connection.

The ever-increasing population and the improvement of the living conditions in the area have rapidly increased the water use, and now the groundwater contamination by the uncontrolled wastewater discharge has become a deplorable level, requiring immediate actions to prevent further degradation of the groundwater as well as the beach. Further groundwater contamination in the area will no doubt cause serious damage to the environment and development of the area, especially for tourism and development of the rich recreational use of the sand beach.

The observation of existing situation apparently indicates that a comprehensive sewerage system construction programme be immediately implemented. If no modern sewerage system is provided, the sanitary conditions will become progressively worse. Moreover, the sewage treatment plant effluent will be fully utilized for agricultural and other purposes as a new water source. If the project is not implemented at this time, the cost escalation due to the world wide inflation might hamper the project implementation at later stage.

CHAPTER-THREE

FUNDAMENTAL PLANNING CONSIDERATIONS

Chapter Three

FUNDAMENTAL PLANNING CONSIDERATIONS

3.1. INTRODUCTION

This chapter deals with the planning fundamentals for the sewerage and drainage facilities. The design basis for the component facilities have been developed and various alternative plans of major facilities studied so that the most appropriate system planning can be worked out for the project, Following a review of the alternative technologies, the best solution for each of the components has been selected.

Details of the basic technical and economic information, including such detailed computations as hydraulic computations for sewers, pumping stations and treatment plant, are enveloped in Volume Three - Appendices and Volume Four - Drawings.

3.2. DEFINITION OF THE STUDY AREA

As defined in the Scope of Work for the Study, the Study Area encompasses a total area of approximately 800 ha, including El-Arish City, Masaid and Salem districts. The Study Area has been investigated and reviewed in the light of existing physical and development conditions, future development schemes, and the results of the topographic surveys conducted under the Study, to designate area for the study. The defined Study Area is shown in Figure 3.2.1,

In defining the Study Area, the following factors are taken into account:

(a) Population density.

(b) NSG's housing development schemes now under construction and the settlemnt is expected in the near future.

(c) The districts where private housing plans are either planned or in progress.

(d) Resort area along the beach where provision of tourist accommodation is expected.

(e) Areas adjacent to the designated Study Area where housing development is expected in the immediate future and from where all the wastewaters will have to be discharged into the Study Area due to natural topography of the areas.

The Study Area for the Project thus defined comprises the following districts:

- Existing urban district	El-Arish	432 ha
	Abu-Saghal	43 "
- NSG's housing development area	El-Masaid	106 "
	El-Salem	114 "
- Housing development area expected in immediate future	North of El-Salem	34 "
- Tourism development area	Along the beach	111 "
Total		840 ha
- Tributary area due to topographic features		160 ha
Total of sewerage tributary		1,000 ha

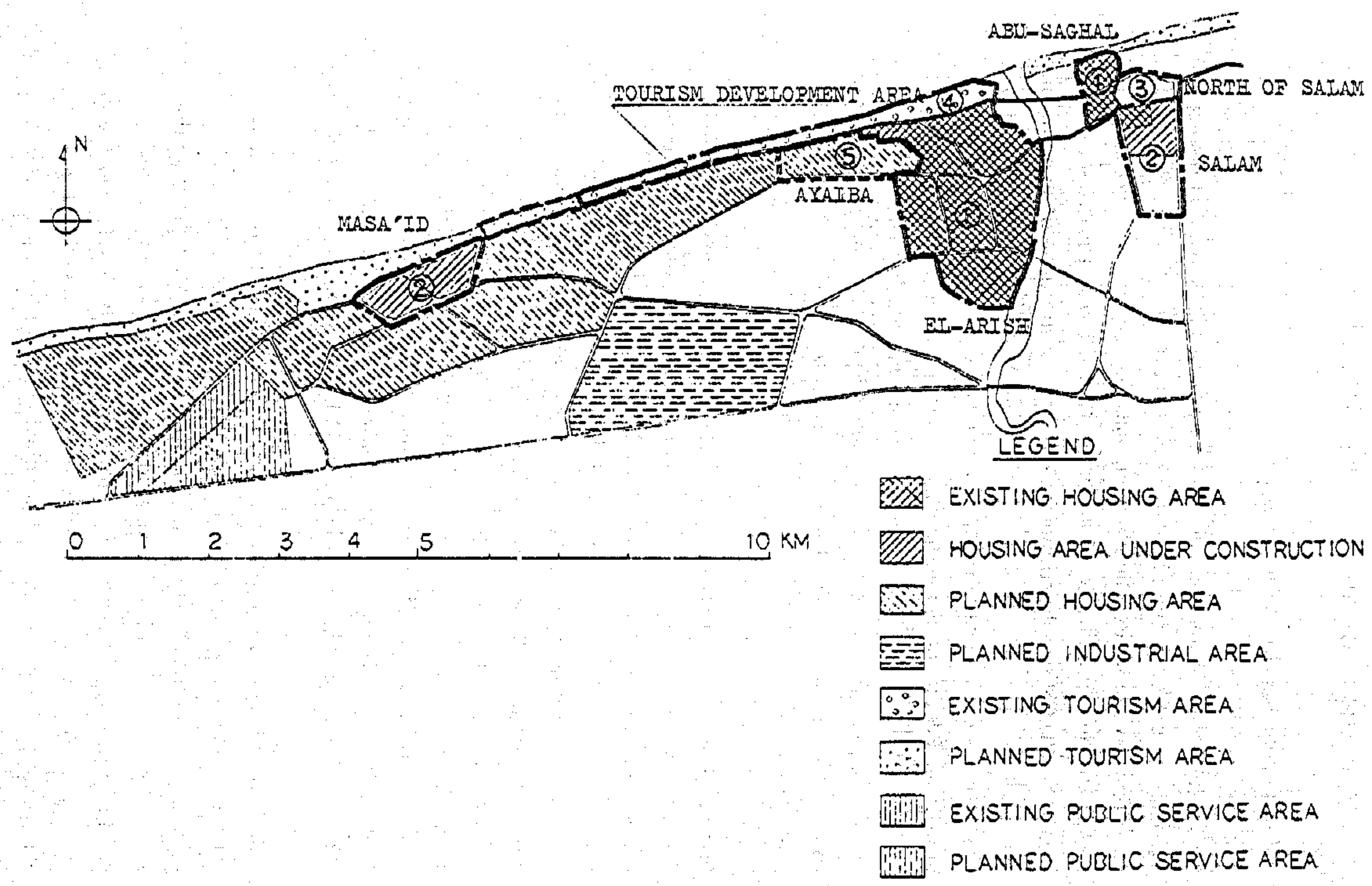


Figure 3.2.1. Study Area

3.3. POPULATION ESTIMATES

3.3.1. Population Growth in Sinai

A comprehensive study has been made by Dames & Moore and government agencies to estimate the population in Sinai peninsula (Ref. No. 6). According to the study, Sinai's population is projected to grow from 172,000 to one million by the year 2000. Migratiion will provide an estimated 500,000 to 600,000 people to Sinai. The natural increase of current and immigrant population will provide the additional 150,000 to 200,000 population. The net productive rate, or population growth rate, in Egypt in 1981 was 2.9 per cent per annum. Sinai is an area where rapid population growth is called for, however, the net productive rate of existing population, calculated to be an average 2.2 per cent per year, is projected to drop to 2.0 per cent for the period of 1987 - 1992 and 1.7 per cent for 1992 - 2000.

Another factor in the projection of the future population in Sinai is the role of migration to Sinai. Migration to Sinai is necessary to expand current population and provide the labour skills and range of human talents necessary to develop the peninsula. The report also estimates that out of a million people in Sinai in the year 2000, approximately 700,000 to 735,000 will be supplied from migration. Migrants to Sinai will be primarily young married couples with the beginnings of families. They will therefore have a reproductive rate that exceeds the averages for Sinai or Egypt. On account of these factors, it is estimated that residents of Sinai in the year 2000 will include 600,000 to 620,000 migrants. Another 130,000 to 150,000 will be the offspring of immigrants.

Sinai's population is projected to grow from 170,000 to one million by 2000, of which the growth in subregion that are currently least populated will be the greatest. The Northeast Coast subregion (including El-Arish) will quadruple in population by 2000, growing at a slower rate than any other subregions. The projected population, by phase and subregion, for recommended strategy have been calculated by Dames & Moore, which indicate that the population in the Northeast Coast subregion in the year 2000 will be 453,822.

3.3.2. Population Growth in the Study Area

(a) Previous Population Estimates

Reasonably accurate enumeration of the population in the Study Area has been difficult to obtain. Because of uncertainties of population counts, there is insufficient confidence in growth rate to warrant a conventional calculation of the population growth. In view of the situation, an attempt was made by a previous study (Ref. No.8) to estimate the population growth rate in the Study Area. The projection in the study was made on the estimates that El-Arish City had 70,000 residents, and the City and its surrounding areas up to the distance of 30 km would have the population of 91,531 in the year 198[illegible]. These figure were allegedly confirmed in meetings during the study and the committee for the study recommended a figure of 100,000 as the population in El-ASrish City and its environs as of the end of 1983, with a population growth rate of 3.2 per cent per annum, a settlement rate of 3 per cent per annum, and 15 per cent for tourists. The report projected the population in the City and its environs in 2005 at 375,612.

(b) Methods for Population Estimates

Commonly used methods to estimate the future population may be classified as:

- Estimates based on the past census data
- Estimates by the future land use plan, considering population densities in planned zones
- Estimates on the basis of analysis of the components that make up population growth, namely, natural increase and migration
- Estimates based on the availability of water resources

For El-Arish City and its environs, sufficient data are lacking to develop reliable population computations because of the long Israeli occupation, and the estimates based on the census data are unlikely to be appropriate. The future land use planning of the City has not been authorized yet and thereby definite data are not available with regard to the future population projection. Further, the necessary data for the natural increase and migration are also lacking for using such method.

On account of the situation it seems that the most reliable method will be that on the basis of the projected future population densities in the area which will show the future capacity to absorb the increased population. The Study Area is considered to develop mostly as residential area, and no drastic change of the presently dominating conditions is unlikely to occur within the coming 20 years. This situation allows to make reliable population forecast easy by using the estimated population saturation.

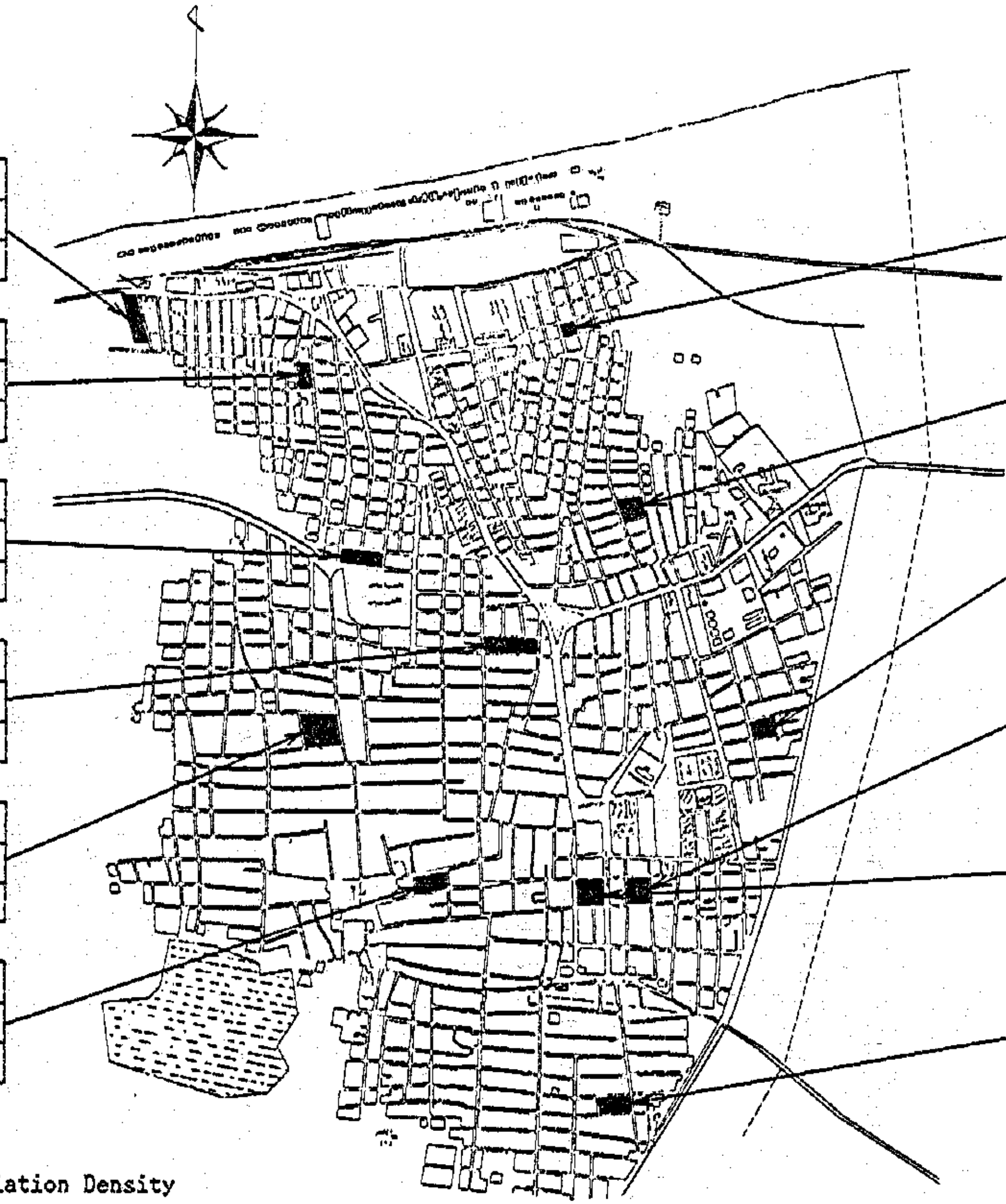
(c) Population Estimates Based on Population Densities

Presently, buildings of 5 to 6 stories have been constructed mainly along main streets in urban builtup areas, however, the number of the buildings are small and the majority of buildings are 1 or 2 stories residences. Since there still exists some vacant lots in urban builtup districts, it seems that there are allowances for absorbing the future population increase.

To check the capacity of the builtup urban area of the City for absorbing the future population increase, several representative districts of the builtup area have been selected and the populations in the districts counted, and then saturated populations estimated. As shown in Figure 3.3.1., totally 12 representative districts which are considered to be saturated were selected and then areas, family numbers and populations checked.

As may be seen from the figure that in the northern part of the City where buildings are relatively new the population densities are in general low, while those in the southern part of the City are high. The population densities range from 100 to 250 persons per ha but variations of the densities are relatively small. The average population density of these districts is 145 persons per ha, therefore, it is considered reasonable to use the figure for projection of the future population in the Study Area.

The population outside the sewerage planning area tends to decline, although the rate is slow, and that the population outside the sewerage planning area in the year 2005 is estimated to be 5,000.



Area (ha)	1.26
House	22.0
P.D. (p/ha)	105.0

Area (ha)	0.49
House	9.0
P.D. (p/ha)	110.0

Area (ha)	0.87
House	22.0
P.D. (p/ha)	152.0

Area (ha)	0.84
House	19.0
P.D. (p/ha)	114.0

Area (ha)	1.26
House	21.0
P.D. (p/ha)	100.0

Area (ha)	0.61
House	16.0
P.D. (p/ha)	157.0

Area (ha)	0.28
House	6.0
P.D. (p/ha)	129.0

Area (ha)	1.03
House	20.0
P.D. (p/ha)	117.0

Area (ha)	0.37
House	16.0
P.D. (p/ha)	246.0

Area (ha)	0.72
House	23.0
P.D. (p/ha)	192.0

Area (ha)	0.93
House	18.0
P.D. (p/ha)	116.0

Area (ha)	0.92
House	20.0
P.D. (p/ha)	194.0

note: P.D. =Population Density

Figure 3.3.1. Population Densities in Saturated Urban Districts

The future population in Masaid area of 106 ha is estimated to be 20,000. The Study Area excluding Masaid area of 894 ha is assumed to have the average population density of 145 persons per ha, thus the total population in the year 2005 will be 130,000 persons. The total population in the sewerage planning area is then projected as follows:

Sewerage planning areas		
Masaid sewerage district	106 ha	20,000 persons
El-Arish sewerage district	861 "	130,000 "
Total of sewerage planning areas		150,000 "
Outside of sewerage planning area		5,000 "
Total of El-Arish		155,000 persons

(d) Population Estimates Based on Available Water Resources

As previously mentioned in Section 2.7. of this report, the present water supply of about 20,000 m3/day has been consumed by a total of about 70,000 residents in the area. In the immediate future, the additional 20,000 m3/day water will be made available for the city water system, thus totally approximately 40,000 m3/day water will be the maximum water resources to be shared to the City.

The water consumptions in 2005 can be estimated as follows:

Daily average consumption	155,000 x 0.15 = 23,250 m3/day
Daily maximum consumption	155,000 x 0.20 = 31,000 m3/day

The average water production during summar seasons may increase about 1.2 times of the daily average rate by fully operating the existing well pumps (See Table 2.7-1), thus the average summar season consumption will be:

Average summar season consumption 23,250 x 1.2 = 27,900 m3/day

The maximum water supply capacity of the system is the sum of the 20,000 m3/day water from El Qantara and the maximum groundwater production from the existing deep wells. As shown in Table 2.7-1, the average capacity of the well pumps is 50 m3/hr and the total operable pumps are 21, however, some of the pumps are old and impellers worn out, thereby the operational efficiency seems to be low. Moreover, due to ocassional electric failure, the operation has been often suspended. In view of these conditions, the over-all pump operation efficiency is assumed to be 80 per cent. The maximum supply of well water will be:

50 m3/hr x 21 No. x 24 hr x 0.8 = 20,160 m3/day (under 1984 conditions)

By adding the additional water of 20,000 m3/day from El Quantara, the total water available will be 40,160 m3/day.

If the ratio of accounted-for water is assumed to be 0.75, the water production required will be:

40,160 x 0.75 = 30,120 m3/day

The above calculations verify that the water production will meet the requirements of daily average and summar season.

(e) Conclusions

The preceding discussions reveal that the estimated total population of 155,000 in the Study Area in the year 2005 is reasonable from the viewpoints of available water resources and the future holding capacities of the land in the area. It is concluded therefore that the population distribution in the area in the year 2005 be as follows:

- Sewerage planning area	150,000 persons
- Outside the sewerage planning area	5,000 "
Total of El-Arish	155,000 persons

With due considerations to the factors cited,the population increase in the sewerage planning area (excluding that in Masaid), Table 3.3.1. has been constructed as an approximation of the sewerage planning area population over the next 23 years from 1983.

Table 3.3.1. Population Forecasts for Sewerage Planning Area

Year	Population	Year	Population
1983	70,000	1996	105,500
1984	72,900	1997	108,200
1985	75,500	1998	110,900
1986	78,200	1999	113,600
1987	80,900	2000	116,400
1988	83,600	2001	119,100
1989	86,400	2002	121,800
1990	89,100	2003	124,500
1991	91,800	2004	127,300
1992	94,500	2005	130,000
1993	97,300		
1994	100,000		
1995	102,700		

3.4. WASTEWATER FLOW QUANTITIES AND QUALITIES

3.4.1. Wastewater Quantities

(a) Daily Average Wastewater Flow

In El-Arish neither large scale water-consuming industries nor commercial development exist, and that the water use of domestic component accounts for more than 80 per cent of the total water consumption of the area while other uses are small in proportion. On account of the fact, it is reasonable to estimate the wastewater flows on the basis of the quantity of the domestic component.

An average per capita sewage flow rate can be estimated based on the water consumed by the inhabitants, however, no reliable data on the domestic water consumptions were available from the City Council. Because of the deficiency of water meters in the water supply system, the only possible means to obtain the actual water consumptions was to measure the well pump capacities in operation and the served population by the system.

The operational conditions of the existing wells are shown in Table 2.7.1. As may be seen from the table, these pumps have been in operation for years and some of the pump impellers have apparently worn-out and obsoleted, and the overall pump efficiencies lowered. Moreover, the operation of these pumps have been occasionally suspended due to electric failure and other maintenance problems, thus actual rate of operation being significantly reduced. The efficiency, although no data indicating an exact figure were obtained, could be conservatively assumed at around 0.8 from our field observations. If it is assumed that the average capacity of the pumps is 50 m3/hr, the average water production in the year 1983 can be estimated at;

$$50 \text{ m3/hr} \times 15 \text{ pumps} \times 24 \text{ hr} \times 0.8 = 14{,}400 \text{ m3/day}$$

An accounted-for water ratio in a water supply system is estimated by means of water bills, however, in El-Arish water system no such reliable data obatined. The field investigations under this study revealed that almost a half of the existing water meters were out of order and were not usable for reading. According to the City Council's information, the water consumed by a family without a provision of water meter has been assumed by a meter reader on the basis of the past consumptions, as such the records of the City Council coud not be used for engineering purpose. The water consumption as recorded by the Water Charge Collection Department of the City Council was approximately 2,100,000 m3. Besides, about 200,000 m3/year water was utilized for such other purposes as irrigation and construction works, thus making the total water consumption to be 2,300,000 m3/year in 1983. From these figures, an average accounted-for water ratio may be estimated at;

$$2,300,000/(14,400 \times 365) = 0.44$$

The accounted-for water ration above seems to be low compared with those in other cities in similar nature. This cause of this low ratio may be attributable to water meter deficiency and rather arbitrary way of meter readings and current billing system. For these reasons, the accounted-for ratio of 44 per cent is considered not reflect the actual present conditions and a slightly higher value should be used for the planning purpose. Considering the present conditions of the area and also available information elsewhere, a ratio of 60 per cent has been assumed for the estimation of present water consumption. For the condition in the year 2005, the accounted-for water ratio will be higher than that at present because of the expected improvement in water pipe materials and rehabilitation of the existing facilities. A study report by the Ministry of Housing prepared for provincial water supply systems suggests target losses of water systems by the year 2000 as shown in Table 3.4.1.

Table 3.4.1. Target Losses of Water in Provincial Water Schemes

Item	Target Losses (%)		
	by 1985	by 1990	by 2000
As percentage of production			
Small schemes	25	20	18
Large schemes	30	30	25
As percentage of consumption			
Small schemes	33	25	22
Large schemes	42	42	33

Source: Provincial Water Supplies Project, Ministry of Housing, General Organization for Potable Water, Volume 3, pp.41, February 1980.

As may be seen from Table 3.4.1., the water loss from the production for large schemes is targeted at 25 per cent for the year 2000. The target is to reduce approximately 15 per cent from the present level of 60 per cent. cent. This target is considered reasonable for El-Arish water system, since great improvement in water supply facilities can be expected by 2005. In view of these, the accounted-for water ratio in the year 2005 is assumed to be 75 per cent of the production.

From the foregoing discussions, a daily average per capita water consumption in 1983 is estimated assuming the served population as 70,000 and daily average water production as 14,400 m3:

$$14{,}400 \text{ m3/day} \times 0.6/70{,}000 = 0.12 \text{ m3/c.d.}$$

For planning the future per capita water consumption, Sinai Development Study Phase I, Draft Final Report (Ref. No.6) shows a giudeline as summarized in the following table.

Table 3.4.2. Suggested Guideline for Per Capita Water Consumption (l/c.d.)

Population	1981	1987	1992	2000
50,000 or more	20	40	100	150
20,000 to 50,000	-	25	50	100
5,000 to 20,000	-	17.5	30	50
less than 5,000	-	10	20	30

Note: Including commercial water but exclude industrial water

Daily average wastewater flow rates in various other cities in Egypt are listed in the following table.

Table 3.4.3. Daily Average Wastewater Flow Rates in Egyptian Cities

City	Year	Daily average (lpcd) Min.	Average	Maximum	Remarks
1) Port Said	1980	70	138	202	Residential zone
"	2000	88	156	220	

(to be continued)

(Continued)

City	Year	Daily average (lpcd) Min.	Average	Maximum	Remarks
2) Ismailia	1985	63	-	413	Res-commercial
"	2000	120	-	390	
3) Suez	2000	-	145	-	Residential zone
4) Helwan	2000	200	-	250	"
5) Alexandria	2000	100	-	136	"
"	1980	75	-	120	"

Source: 1) Ref. No.38. 2) Ref. No.37. 3) Ref. No.20. 4) Ref. No.39. 5) Ref. No. 40.

From the above mentioned discussions and comparison of the study results with the data obtained from other similar cities in Egypt, the average daily per capita sewage flow rate in the year 2005 is estimated to be 150 lpcd. Although accurate information to indicate the future sewage flow rates, the 150 lpcd daily average sewage flow rate is considered reasonable from the viewpoint of the available water resources in the future.

(b) Daily Maximum Wastewater Flow

The daily maximum wastewater flow has been estimated on the basis of the actual operation of the well pumps over the three years from 1981 through 1983 as shown in the following table.

Table 3.4.4. Operational Conditions of Well Pumps

Year	Average in winter	Yearly average	Summar average	Maximum
1981	6 Nos.	10	13	15
1982	8 Nos.	12	15	17
1983	13 Nos.	15	17	19

Source: Eng. Saleh Mohamound Elaseniy, Manager of Water Supply Dept. City Council.

As may be seen from the above table, the numbers of the well pumps in operation varied according to the seasonal water demands. The ratio of the daily average flow to daily maximum flow in each of the years is then calculated as follows:

1981 10/15 = 0.67 1982 12/17 = 0.71 1983 15/19 = 0.79 Average = 0.72

Fom the above calculations, the ratio of the daily average to daily maximum flow has been estimated to be 0.75 for the 2005 conditions. Thus, the per capita daily maximum sewage flow rate is estimated at:

$$150 \text{ lpcd}/0.75 = 200 \text{ lpcd}$$

(c) Hourly Maximum Sewage Flow

The results of field investigations under the present study carried out at selected houses representative of average family in the area revealed that the ratio of the hourly maximum sewage to the daily average was 2.3. As shown in Table 3.4.5., the water consumption of domestic use accounts for about 80 per cent of the total water consumption, and this water use pattern is unlikely to change in the foreseeable future. In the sewerage system serving more population than those in these households, the ratio or peaking factor will be lower than this value, and hence the factor of 2.0 can be reasonably applied to the sewer planning and design. The hourly maximum sewage flow rate is then estimated at:

$$150 \text{ lpcd} \times 2.0/24 \text{ hr} = 12.5 \text{ lpch}$$

Table 3.4.5. Water Consumption by Use in El-Arish (1983)

Item	Water Consumption (m3/year)
Households	1,763,627
Factories	48,372
Hotels	35,015
Restaurants	4,567
Gas stations	8,531
Bakeries	900
Government offices	239,467
Miscellaneous uses	x
Total	2,100,479 + x

Source: Water Charge Collection Department, El-Arish City Council.

Wastewater flow variations showing the typical household water use surveyed in Salem housing complex are illustrated in Figure 2.8.2(12).

(d) Infiltration

As previously mentioned, groundwater elevations in the areas adjacent to the sea coast are in general high, particularly in Masaid sewerage district where significant portion of the land lies at around 2 metres above mean sea water level.

In the El-Arish sewerage district, the total sewer length to be laid below the groundwater elevation will be approximately 5 km. Assuming that 71 m3/km/day groundwater may infiltrate into the sewer pipes (Ref. No.13), the total amount of the groundwater infiltration to the sewers can be estimated at 355 m3/day. This quantity accounts for only 2 per cent of the total daily average sewage flow rate, which is negligible in planning of sewer pipes in the El-Arish district as it is small in amount.

In the Masaid sewerage district, where significant portion of the land lies at around 2 to 3 metres above mean sea water level an appropriate amount of groundwater should be considered in determining sewer capacities. Since there is at present no sewerage system in the area from which information on groundwater infiltration may be obtained, an effort was made to derive such information from existing sewerage system in Egypt that may represent conditions similar to those in the district. As shown in Table 3.4.5., amounts of the design groundwater infiltrations in other cities in Egypt range from 8 m3/ha/day to 12 m3/ha/day. Taking into account the above figures and conditions of the district, an average infiltration of 8 m3/ha/day is determined for the use of sewer planning and design in the Masaid sewerage district. The total amount of the groundwater infiltration into the sewers in the Masaid district is then estimated to be:

$$8 \text{ m3/ha/day} \times 106 \text{ ha} = 900 \text{ m3/day}$$

Table 3.4.6. Design Groundwater Infiltration in Other Egyptian Cities

City	Infiltration Considered		
1) Port Said	Existing developed area	-	12 m3/ha/day
2) Ismailia	New area	-	8 "
3) Suez	General area	-	10 "
	Low-lying area adjacent to canal water	-	12 "
4) Helwan	$Q = a.d.h^{2/3}$		
	where Q = flow per 1,000 metres		
	a = Constant (5 to 10)		
	d = exterior diam. of pipe in inches		
	h = average depth of pipe below the groundwater surface		
5) Alexandria	0.1 l/sec/ha		

Source: 1) Ref. No.38 2) Ref. No.37 3) Ref. No.39 5) Ref. No.40

(e) Design Flow Rates

From the foregoing discussions, design flow rates for the sewerage system have been determined and the over-all wastewater quantities estimated as shown in the table below.

Table 3.4.7. Design Sewage Flow Rates

Item	El-Arish Sewrage Dist	Masaid Sewerage Dist.
Served area (ha)	894	106
Served population	130,000	20,000
Groundwater(m3/day)	-	900
Daily average flow (m3/day)	19,500	3,900
Daily maximum flow (")	26,000	4,900
Hourly maximum flow(m3/hr)	1,625	288

Note: El-Arish district include the future inflowing area of 127 ha.

3.4.2. Wastewater Qualities

In order to estimate the wastewater qualities for both present and future conditions, the results of the wastewater survey have been analysed as described in the following:

(a) Present Daily Per Capita BOD

As described in Section 2.8., the average daily per capita BOD of the inhabitants in the apartment houses was 8.3 g, which appears to be too low to directly apply for the sewage treatment plant design. The reasons for such a low BOD value may be attributable to the following:

- Because of the short distance between the wastewater sources and the transh, sufficient mixing of excreta and sullage water did not undergo while flowing down, as such the wastewater collected seemed to consist mostly of sullage water.

- As mentioned in Section 2.8., the water quantities measurement was unable to carry out simaltaneously with the water quality samplings because of the lack of water meter provision. Thus there would heve been some discrepancies in quantities between the actually consumed and measured separately.

- The average BOD concentration of the sampled wastewater was 130 mg/l. which seems to be rather low compared with data in other similar cities in Egypt. This can explained by the reason mentioned in the first paragraph.

For the reasons above, it is only prudent to consider that the average daily per capita BOD of 8.3 g is contributed by the sullage water only and that another 13 g to be contributed by excreta should be added to 8.3 g, thus the total average daily per capita BOD value at present is 20 g.

(b) Future Wastewater Characteristics

For the estimation of the future daily per capita BOD contribution, some of the data of the cities similar in nature have been compared. Duncan

Mara (Ref.No.36) shows figures which have been obtained for the daily per capita BOD contributions as follows:

Zambia	36 g
Kenya	23 g
S.E.Asia	43 g
India	30 - 45 g
Rural France	24 - 34 g
UK	50 - 59 g
USA	45 - 78 g

He also suggests that as the design value for tropical developing countries probably about 40 g/c.d. is appropriate. The breakdown of the 40 g is as follows:

Water Use	USA	Tropics
Personal washing	9	5
Dishwashing	6	8
Garbage disposal	31	-
Laundry	9	5
Toilet faeces	11	11
Urine	10	10
paper	2	1
Total (average adult)	78	40

From the foregoing discussions and analyses, it is consiederd appropriate to estimate the present average per capita BOD contribution is 20 g, and this will be increased at a rate of 1 g annually reaching at 40 g by the year 2005. As previously estimated, the average daily per capita wastewater flow is 150 l, therefore, the BOD concentration of the wastewater in the year 2005 will be;

$$40 \text{ g} \times 10^3 / 150 \text{ l} = 270 \text{ mg/l}$$

In the same manner, concentrations of other items are estimated below;

SS	250 mg/l
COD	220 mg/l
T-N	80 mg/l
T-P	10 mg/l

(c) Evaluation of Existing Sewage Treatment Plant

To supplement the data and information, further investigation was carried out at Hotel Obeori on 18th August 1984. The numbers of hotel guests and monthly water consumptions obtained from the hotel are as follows:

Table 3.4.8. Numbers of Guests and Water Consumed at Hotel Oberoi

Month	No. of Guests (persons/month)	Water Consumed (m3/month)	Remarks
1983 Sept.	968		
Oct.	565		
Nov.	506		
Dec.	728		
1984 Jan.	599		
Feb.	648	9,598	5.3 m3/c.d.
Mar.	548		
Apr.	2,140	1,107	0.5 m3/c.d.
May	2,904	6,478	2.2 "
June	5,040	8,107	1.6 "
July	7,057	10,500	1.5 "
Aug.	21,703		up to 18th day

In general, the average daily per capita wastewater quantity in hotels is from 190 to 380 l, so that these wastewater quantities were unusually high and the variations of the quantities were also significant. According to the allegation by the hotel personnel in charge, the average daily per capita water consumption was 1.5 m3, including those for sprinkling, cleansing, pool and cooking waters.

The figures obtained are unlikely to indicate the conditions accurately, if not at all incorrect. The hotel has its own sewage treatment plant and has been in operation since its opening, however, unfortunately no detailed information on the plant facilities were available except for certain drawings. For the purpose of the investigation and evaluation of the facilities, dimensions were measured at the field and flowsheet prepared. From these data, the existing system seemed to be the extended aeration process using mechanical aerators.

From the drawings and actual measurement at the field, the aeration tank capacity was estimated to be 380 m3 followed by the final settling basin of 90 m3 capacity with the surface area of 45 m2. The wastewater quantities in the treatment plant in the cases of 16, 20 and 24 hours aeration time are estimated at:

Table 3.4.9. Wastewater Quantities in Hotel Plant by Different Aeration Times

Aeration Tank		Wastewater	Settling Basin		
Capacity (m3)	Aeration time (hr)	(m3/d)	Capacity (m3)	Overflow Rate (m3/m2.d)	Detention Time (hr)
380	16	570	90	13	3.8
"	20	450	"	10	4.8
"	24	380	"	8.5	5.7

In view of the design criteria for the extended aeration process elsewhere it is considered that 450 m3/day wastewater quantity is reasonable for the facilities. This hotel has 150 rooms with 300 beds. Assuming 300 guests as maximum, then the average daily per capita water consumption is 1.5 m3, which coincides to that alleged by the hotel. The hotel also claims that about 25 per cent of the water is lost due to dedsalination process. If the actual consumption is assumed to be 80 per cent of the production, the average daily per capita water consumption will be:

$$1.5 \times (1 - 0.25) \times 0.8 \times 10^3 = 900 \text{ l}$$

Since the average BOD concentration of influent was 270 mg/l, the daily average per capita BOD contribution is estimated to be:

$$270 \times 900 \times 10^3 = 243 \text{ g}$$

This value is exceedingly higher than those in other similar cities.
Even if the general water consumption of 190 to 380 l/c.d are used, the values are:

51 - 103 g/c.d.

The above evaluation was made on the basis of the figures alleged by the hotel personnel, however, there were significant differences between the personnel, Furthermore, there has been no reliable means to measure the exact amount of water produced in the desalination plant, the accuracy of these data are doubtful and justified not to be used for the estimation of the per capita waste loads.

3.5. STORMWATER QUANTITIES

On the basis of the data and discussions as shown in Appendix - 'Quantities of Stormwater' of Volume Three, studies and analyses of rainfall records and physical conditions of the Study Area have been made to develop the basic design criteria for drainage system.

3.5.1. Runoff Fromula

Using the results of studies as described in Appendix - , the Rational Method is selected for drainage conduits and channels planning and design, as expressed by the following relationship:

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

where

Q = Peak discharge of the watershed above the point in question due to the maximum storm assumed.

C = Runoff coefficient, which is the ratio of the amoun amount of rainfall

I = Rainfall intensity based upon time ofconcentration.

A = Area of the watershed.

3.5.2. Rainfall Intensity Formulae

Various rainfall intensity formulae have been put forward for the calculation of rainfall intensities in terms of duration of the storm. Among the formulae, the 'Talbot type' formula is considered to best fit to the conditions of relatively short duration rainfalls ranging between 10 and 60 minutes. To develop rainfall intensity formulae for the different frequencies of storm, rainfall data have been collected from the Meteorological Authority in Cairo for the last ten years, from 1960 through 1966, 1980, 1981 and 1983. These data have been analysed and the following rainfall intensity-duration relationships for the different frequencies developed:

Frequency of recurrence (once in years)	Rainfall Intensity Formulae (mm/h)
3	$I_3 = 980 / t + 23$
5	$I_5 = 1060 / t + 22$
7	$I_7 = 1120 / t + 22$
10	$I_{10} = 1190 / t + 21$

where I = rainfall intensity, mm/hr
t = time of concentration, minutes

The computed rainfall intensity-duration relationships for the different frequencies have been shown in Figure 3.5.1.

3.5.3. Runoff Coefficients

Runoff coefficients to be used for drainage system design are determined, taking into account soil condition, soil permeability, ground surface types, and groundwater elevations of the area. For each drainage district, a composite runoff coefficient has been developed based on the percentage of the different type of surface in the drainage district. The recommended coefficients by year are as follows:

Year	Composite coefficients
1983	0.3
2005	0.5

3.5.4. Time of Concentration

The time of concentration consists of the inlet time plus the time of flow in the drain from the most remote inlet to the point under consideration. Inlet time of 10 minutes is recommended as a design criterion for drainage design. Time of flow in the drain may be estimated closely from the hydraulic properties of the conduit. For the computation of time of flow, the velocity of full flow may be used. Values of time of concentration generally used in stormwater drainage design are shown in Table 3.5.1.

Table 3.5.1. Standard Time of Concentrations

Generally Used In Japan		ASCE Standard
High population density area	5 minutes	-
Low population density area	10 "	–
Trunk drains	5 "	–
Brach and laterals	7 - 10 "	–
Average	7 "	–
All paved, high pop. density completely sewered area	–	5 minutes
Relative;ly flat developing area	–	10 - 15 "
Average housing area	–	20 - 30 "

Source: Guideline for Sewerage Planning, Japan Sewage Works Association, 1972.

3.5.5. Rainfall Frequencies

The average frequency of rainfall occurrence to be used for design determines the degree of protection afforded by a given storm sewer system. The characterictics peculiar to El-Arish with regard to ranfalls are 1) low rainfall precipitation, 2) low rainfall intensities, and 3) high permeability of soil. Because of these characters of rainfalls, there have not been severe damage caused by rainfalls, except those caused by floodings of the Wadi El-Arish.

During rainstorms, it has been observed that in some low-lying areas, storm water runoffs overran on streets and in certain districts flooding in house floors facing the streets. According to the City Council, such floodings occurred once every five years in the past. For this reason, the frequency of 5 years is determined for drainage system design.

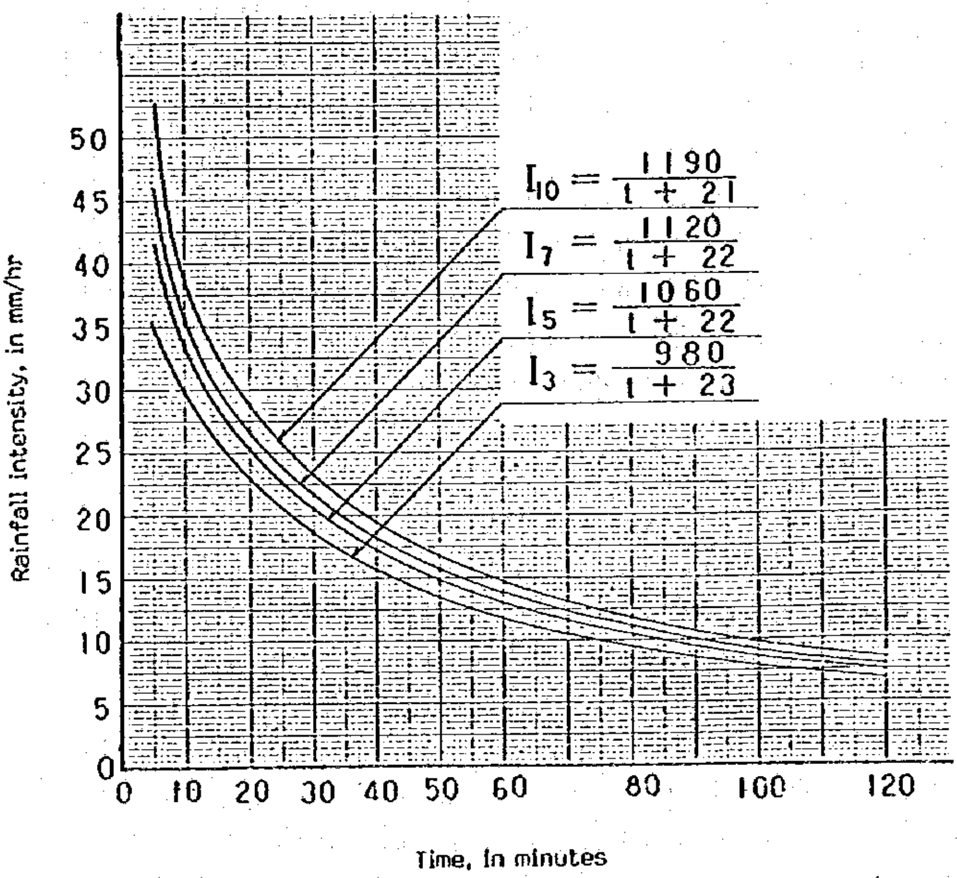


Figure 3.5.1. Intensity-Duration Rainfall Curves

3.6. ENGINEERING CONSIDERATIONS FOR SYSTEM PLANNING

3.6.1. Alternative Wastewater Systems Considered

For the control of wastewaters and for improvement of public health in the area over the next 20 years up to 2005, all feasible alternative wastewater collection, treatment and disposal systems for the area have been delineated and analysed from technical, environmental and economic viewpoints.

There have been many sanitation technologies options available for both on-site and off-site sanitations that can meet the requirements to alleviate the present wastewater loads to the soil in the area; however, most of the households in the area have already been served by the flush toilet with transh system, and thereby, such low grade sanitation facilities as pit latrine have been excluded from the alternative analysis. After preliminary screening of various sanitation options, three most appropriate sanitation alternative systems have been selected for further analysis. The alternatives include:

- On-site sanitation system; A combination of the septic tank and transh system. The mixture of sullage and flush toilet excreta enters the septic tank and thence flows to the transh to soak into soil as it does in the existing transh system.

- Smallbore sewers; Comprises a household toilet, an influent pipe to an interceptor tank in which settleable solids are retained, a tank effluent pipe, street laterals and mains, pumping stations as required, and treatment works at the terminal of the conveyance system.

- Conventional sewerage system; Comprises house connections, public sewers, pumping stations, and sewage treatment works

Each of the above alternative wastewater systems is described in details and evaluated for the final selection of a system in the following paragraphs.

(a) On-site Sanitation System

Flush toilets of existing transh system will be used without modification but only modify part of the existing sewer pipes connecting the toilets to the transh. The septic tank will be installed within the plot in between the flush toilet and the transh. During 1 to 3 days retention period, solids settle to the bottom of the tank where they are digested anaerobically. The supernatant will then inflow to the transh for soakaway to the soil. The effluent from the septic tank is, from a public health viewpoint, as dangerous as raw sewage and so requires further treatment before disposal if the groundwater pollution is to be prevented.

Although the septic tank system can reduce organic loadings to the ground to some extent, the effluent cannot be recovered. Also, one of the disadvantages in providing the system will be the availability of a space for the septic tank structure. Most of the houses in the central portion of of the city, a sufficient land space for the provision of both septic tank and transh may not be available.

The costs of septic tank installation, as well as periodic desludging, make it inappropriate for the population. There are indications that in urban areas septic tanks will often cost more on a per household basis than conventional waterborne sewerage system (Ref. No.45). The septic tank may emanate odour if maintenance is not properly done.

Another disadvantage, perhaps the most important, by the provision of this system is that all the wastewaters are discharged to soil and no sewage can be reused for crop irrigation or other purposes. Thus, in terms of benefits from the system, this alternative plan is least economical although initial costs for the system may be less than other alternative plans, particularly in consideration of the scarcity of the water resources in the region. Moreover, in the low-lying districts where the groundwater elevation is high and soil permeability is low, no improvement can be expected to solve transh clogging problems.

In view of the above mentioned reasons, the septic tank and transh system is justified not appropriate to apply as the wastewater system for the area.

(b) Small Bore Sewers

This system requires the provision of interceptor tank to retain settleable solids within households. In this system, existing transh may be converted to the interceptor tank by a minor modification of the structure. Removal of settleable solids in the interceptor tank allows the effluent to flow by gravity and does not require the self-cleansing velocities necessary in conventional sewers. No large solids being discharged into the small bore sewer system allow for reduction of pipe diameters (up to 60 per cent in the case of street laterals and mains). Use of PVC pipes requires no special corrosion protection and prevents significant infiltration of groundwaters. Other variances from the conventional sewerage design give rise substantial savings, including reduced pipe slopes and depths and minimal requirements for manholes and pumping stations under normal conditions.

Sludge may accumulate in the interceptor tank, which is to be collected by vacuum tank or other means once every few years and transported to treatment plant for final treatment. The collected sewage will be further treated in the sewage treatment plant and the effluent will be utilized for various purposes as that from the conventional sewerage system.

The interceptor tank will be about 1.5 metres in diameter and 2 metres deep, providing a minimum of 40 litres per capita per year of sludge storage volume, and a two days retention period to ensure adequate removal of settleable solids. All interceptor tank effluent pipes may be as small as 50 mm in diam. but for branch and lateral sewers, the diameters will be from 100 mm and most of the sewer pipe diameters will be smaller than those for the conventional sewerage system. No manholes are necessary for the small bore system, either for cleaning out or at junctions of sewers or at changes in grades. A comparison of the system with the conventional sewerage system indicates that the small bore systems are estimated at roughly 60 per cent of the conventional system.

The major costs of the conventional sewer systems are the street laterals and manholes. Small bore system is more cost effective than the conventional system. Under normal conditions, operation and maintenance costs will be lower than the conventional sewers because of the less solids content in the sewage flow thus eliminating to a great extent the necessity of sewer cleansing. Pits and tanks of the small bore sewer system require desludging every few years at an additional cost. Maintenance and repair of the small bore sewer system would be less frequent and costly than the conventional system under normal conditions because the latter is used both liquids and solids, whereas the small bore sewer only contains liquids. Although small bore sewer system has many advantages as mentioned above, there is a grave constraint for the adoption of small bore sewer system to El-Arish area. As observed in many places in the existing pipe systems of the area, a considerable amount of sand enter the pipes and at many locations pipes are completely cologged, requiring frequent cleaning of sand to properly maintain the system's performance. The small bore sewers of 50 to 100 mm diameter will no doubt necessitate much more frequent pipe cleansing than other system and there will be a great possibility of chocking of sewer pipe lines, thus making operation and maintenance costs of this system tremendously higher than other comparable alternative plans. In consideration of these conditions, the small bore sewer system is likely to be less advantageous than other alternatives.

(c) Conventional Sewerage System

This system is one of the most reliable modern sewerage system with a long experience in construction, operation and maintenance, and is most widely used throughout the world, but in general most costly among alternative sanitation systems available for safe disposal of wastewaters. The effluent from the system will be of the level which can be reused for various purposes including crop irrigation. The Government of Egypt places the treated sewage reuse and prevention of groundwater contamination in the area as the top priority programme. The effective reuse of the treated sewage will bring about a high level of benefits even though the construction, operation and maintenance costs will be higher than other alternatives.

A study undertaken by the World Bank (Ref.No.16) indicates that the costs of conventional and small bore sewerage systems planned for about 73 ha district serving a population of 39,420 are in the ratio of 1.581 to 1.013, or comparable costs for small bore sewers are at roughly 60 per cent. The major costs of conventional sewer system are the street laterals and manholes. Conventional sewers and street laterals are sized to facilitate solids cleaning equipment and are therefore larger than peak flows would require. The submains of conventional sewers are designed to accommodate peak flow factors of from 4 to 6. That is they are designed to accommodate peak flows which are 4 to 6 times greater than average flows. The overall length of pipe between houses and street laterals are somewhat higher in the case of conventional sewerage system than small bore due to inefficiencies of conventional sewerage hook-ups as compared to the small bore sewers' shared use of pits and tanks.

Convetional sewer system has many merits: they provide the greatest user convenience of all the waste disposal systems, for they permit the discharge of large amounts of water; they do not pose any risks to health when functioning properly; their maintenance is assumed by the municipality, and they generally operate with few service interruptions or emergencies. Yet sewer systems also have disadvantages; they are, first of all, expensive to construct; they require skilled contractors for the construction, a municipal organization for operation and maintenance, and a substantial amount of flushing water, which adds to the operating costs.

Given the high convenience level of sanitary sewerage, this system of excreta disposal has been the one of the choice almost to the exclusion of other alternatives.

(d) Conclusions

The foregoing discussions have led to the conclusions that the conventional sewerage system is as a whole superior to other alternative plans because of its expected various advantages will sure overcome the disadvantages and is believed to be most appropriate system among the options available particularly in consideration of the long term benefits derived from the system.