APPENDIX D

WATER QUALITY ANALYSIS

1. Introduction

Water quality analyses have been conducted during the period of the first on-site works in Aden. Purposes of the analysis for the sewerage feasibility study include the following three objectives.

1) to examine the characteristics of the raw sewage in the project area.
2) to determine per capita waste loading, such as BOD and SS.
3) to investigate the present pollution in Tawahi Bay.

For these purposes, various sampling points were selected with consultation with counterpart personnel. Time schedule of the sampling is as follows.

1st Sampling
- 30th and 31st, January
- At Al-Shaab STP
- 24 hours sampling for raw sewage
- Several grab samples of stabilization pond water

2nd Sampling
- 5th and 6th, February
- At Al-Shaab STP
- 24 hours sampling for raw sewage
- Several grab samples of stabilization pond water

3rd Sampling
- 13th February
- In and around Tawahi Bay by boat
- 5 grab samples of sea water

4th Sampling
- 20th and 21st February
- At Hedjuff pumping station in Ma'alla and one pumping station and one manhole in Tawahi
- 24 hours sampling at Hedjuff pumping station
- 2 grab samples at a pumping station and a manhole in Tawahi

5th Sampling - 26th and 27th February
- At Hedjuff pumping station in Ma'alla and one pumping station and one manhole in Tawahi
- 24 hours sampling at Hedjuff pumping station
- 2 grab samples at a pumping station and a manhole in Tawahi

All the samples taken have been analyzed at the laboratory attached to the Al-Shaab STP by a water quality specialist of the study team. However, five composit samples, two raw sewage, one treated effluent and two sea water, were sent to Japan for the analysis of total phosphate (T-P) and Kjeldahl nitrogen (K-N). These items were analyzed in Japan. Methods of sampling and analysis, and results are described in the following sections together with some considerations.

2. First and Second Samplings at Al-Shaab STP

2.1 Sampling Points

First and second samplings were carried out at Al-Shaab sewage treatment plant. This treatment plant was completed and commissioned in 1986, and the only one treatment plant in operation at present in Greater Aden. This treatment plant currently receives sewages from Sheik Othman district and Al-Mansura industrial area. There are two pumping stations, one in Sheik Othman and another in Al-Mansura industrial area, which deliver sewage to the plant. This treatment plant was selected because flow rates to the plant can easily be obtained from the pump operation records of the two pumping stations. Per capita waste loading can also be calculated based on the concentrations of certain pollutants, flow rates and population served.

Treatment process adopted in the plant is stabilization pond system. There are two series of stabilization ponds currently in operation, viz. new and old systems. Both systems were designed to have an anaerobic pond followed by two stages of facultative ponds. During the sampling a few grab samples from each pond were taken and analyzed in an attempt to examine the function of the pond system. Therefore, several sampling points were established as described below.

No. of Sampling Point	Location	Object
No.1	Inlet work, at the channel after screen	Raw sewage
No.2	Outlet of Pond A	Anaerobic pond effluent
No.3	Outlet of Pond 6	First stage facultative pond effluent
No.4	Outlet of Pond 8	Final effluent from one of two new pond series
No.5	Outlet of Old Pond	Final effluent of old pond system
No.6	Junction well of outlet channels from ponds 7 and 8	Composite of final effluent from new pond system
No.7	Outlet of Pond 5	First stage facultative pond effluent
No.8	Outlet of Pond 7	Final effluent from one of two new pond series

Sampling points mentioned above are illustrated in Figure D.1.

2.2 Results of Analysis

2.2.1 Raw Sewage

Characteristics of raw sewage obtained from 24 hours sampling are shown in Tables D.1 and D.4. Items such as ambient and water temperatures, pH and DO were measured at each sampling time on the spot. Composite samples of every three hour time intervals were made for the analysis of Chemical Oxygen Demand by manganese and chromium (CODmn, and CODcr), 5 day Biochemical Oxygen Demand (BOD_5), Suspended Solids (SS), and Ammonia Nitrogen (NH_4-N). Composite samples were made as mixtures of four samples taken at each time, quantities of each of the samples into the composite sample were proportionate to flow rates measured at every sampling time on the spot. It should be noted, however, that flow rates in the inlet work channel changed sharply within a few minutes at sampling time, particularly in day time, and they did not represent the average or total flow rates for the specific duration of time. These sharp changes of flow seemed to be caused by frequent on and off of the pumps, indicating too small capacity of the pump well.

Average temperatures and pH and weighted averages of CODmn, CODcr, BOD_5, SS and NH_4-N of the two analyses are as follows. Total phosphate and Kjeldahl nitrogen for the composite sample of the second analysis are also shown.

<u>Characteristics of Raw Sewage to Al-Shaab STP</u>

	Temperature (°C) ambient	water	pH	CODmn (mg/l)	CODcr (mg/l)	BOD_5 (mg/l)	SS (mg/l)	NH_4-N (mg/l)	K-N (mg/l)	T-P (mg/l)
First samples	25.1	29.6	7.4	174	763	384	339	43	-	-
Second samples	24.8	29.0	7.5	136	662	328	407	44	62	11

It can be said from the results of the analyses that raw sewage contains relatively high organic substances and suspended solids. Water temperature is also very high ranging from 28°C to 31°C. Characteristics of raw sewage will be discussed in detail in Section 4 of this appendix.

Some points recognized during sampling should be noted. It was fine weather all the time of the first sampling, but it rained occasionally from around 11:00 a.m. on 5th February till the end of sampling next day of the second sampling. Rain falls on 5th and 6th February were reported to be 14 to 15 mm/day. Storm water run off might have affected the results of analysis. It was also observed that flow had never stopped even in midnight, and water looked clearer. Concentrations of COD, BOD, and SS in day time and night time are obviously different on both samplings, higher in day time. These facts suggest the possibility of groundwater infiltration. This requires further considerations.

2.2.2 Stabilization Pond Water

Samples were taken at various points in the treatment plant in order to investigate and assess the function of the treatment processes. Eight sampling points were selected as mentioned in the previous section. Results of the analyses are shown in Tables D.2, D.3, D.5 and D.6. At the outlets of each pond, anaerobic, fist and second stage of facultative, and old ponds, samplings were carried out twice on both days, at 17:00 in the afternoon and at 6:00 in the morning, since treatment process of stabilization pond mostly depends on photo-plankton. Dominant species of photo-plankton in facultative pond are green algae, which produce oxygen by photosynthesis during day time when solar radiation is available, while they consume oxygen by respiration during night time. In order to evaluate activity of algae, DO levels in the facultative ponds were measured at every two hour intervals. Fluctuation of the DO levels are shown in Tables D.3 and D.6.

There are no meaningful differences in COD, BOD, SS, and NH_4-N between morning and evening samples from every sampling points. Main reasons for this are long retention time in the pond system, longer than ten days at least as a whole. Average values of each measuring items according to the flow of process train are as follows.

Characteristics of Treated Effluent

	pH	CODmn (mg/l)	CODcr (mg/l)	BOD_5 (mg/l)	SS (mg/l)	NH_4-N (mg/l)	K-N (mg/l)	T-P (mg/l)
1. Anaerobic Pond								
Pond A	7.6	107	363	138	127	39	-	-
2. First Stage Facultative Pond								
Pond 5	8.4	95	362	38	118	36	-	-
Pond 6	8.6	89	278	41	213	32	-	-
3. Second Stage Facultative Pond								
Pond 7	8.4	92	296	54	90	20	35	15
Pond 8	8.5	73	229	54	153	13	-	-
Old Pond	8.7	90	-	62	215	14	-	-

Reductions in COD, BOD, SS, NH_4-N and K-N from values in raw sewage are clearly recognized. Reduction in BOD is highest among all the items measured. Ammonia Nitrogen is reduced to less than half that in raw sewage. These reduction and high pH values in effluent indicate active metabolism of bacteria and algae taken place in all the pond, and consequent effective treatment. However, final effluent from the second stage facultative pond shows still high COD, BOD and SS concentrations, higher than those in the first stage facultative pond effluent sometimes. T-P concentration in effluent is also higher than that in raw sewage. This phenomenon was thought to be caused by carry over of green algae, which was observed during sampling. Thus, COD and BOD concentrations in filtered samples were analyzed in an attempt to evaluate COD and BOD originated from solids. Comparison of those concentrations in non-filtered effluent and in filtered one is as follows.

Comparison of COD and BOD in
Non-filtered and Filtered Effluents

		CODmn (mg/l)	CODcr (mg/l)	BOD_5 (mg/l)
Pond 7	Non-filtered	92	296	54
	Filtered	26	117	4
Pond 8	Non-filtered	73	229	54
	Filtered	31	93	15
Old Pond	Non-filtered	90	-	62
	Filtered	28	-	8

COD and BOD concentrations of filtered effluent significantly lower than those of non-filtered effluent, BOD in particular. Although CODcr concentrations in filtered effluent are still high, it is considerably reduced to less than half. These facts mean that most of BOD values in non-filtered effluent are originated from bio-degradable solids, i.e. algae. Therefore, if algae were removed efficiently, highly treated effluent can be obtainable. Figures D.6 and D.7 show fluctuation of DO levels in Ponds 6 and 8 on 30th and 31st January. Fluctuation curves in the figures are typical ones in facultative pond, which rises at highest level at 15:00 p.m. and falls down to zero 19:00 to 21:00 p.m. some time after sunset. DO levels in Ponds 5 and 6, first stage of facultative pond stayed at zero on 5th and 6th February as shown in Table D.6, because of the rain and clouded weather at that time. However, DO levels in day time rose to a certain point in Ponds 7 and 8, second stage facultative pond, on the same days, although a rise was not so significant as before. DO level fluctuation also confirmed the presence and active metabolism of green algae.

During the sampling, it was observed that Pond 5 and all the Old Ponds were colored pink on both sampling days. Pond 6 was also colored pink on the second sampling day. This was caused by sulfur bacteria. Microscopic pictures of bacteria and algae in facultative ponds are shown in Photos D.1 to D.4.

Number of coliform groups in the final effluent was measured with a test paper. Numbers of coliform group are 150 and 550 for the effluents from Ponds 7 and 8 respectively. Although this test method is simplified one and only order of magnitude can be determined, it can be said that reduction of coliform took place satisfactory in the series of facultative ponds since the number in raw sewage is usually of seven orders. The existing stabilization pond system functions efficiently.

2.3 Flow Rates to the Treatment Plant

Flow rates to the treatment plant on both sampling days were measured by operating records of all the pumps in the two pumping stations which are sending all the flows to the plant. One pumping station is located in Al-Mansura district of which flow is domestic origin. Another pumping station is located in industrial area delivering industrial wastewater to the plant.

Meter readings at the starting and ending time of the sampling, i.e. 9:00 a.m. on both 30th and 31st January were carried out for the first sampling. Total flows for 24 hours can only be obtained by that reading and nominal capacity of the pumps. Calculation of flows is as follows.

Total Flows to the Treatment Plant (30th - 31st, January)

Name of P/S	Operating Hours (hrs)	Pump Capacity (m^3/hr)	Flow Rates (m^3/day)
Al-Mansura Main	6.6	1,014	6,692
Industrial Area	12.3	187	2,300
Total			8,992

During the first sampling, sharp changes of flow were observed and flow measurement at the plant was found to be impossible due to changes of flow and break down of measuring device. Meter reading at every sampling time with one hour intervals was intended for the second sampling in order to obtain hourly flow fluctuation pattern, in particular, peak flow. Hourly flow rates from 9:00 a.m. 5th to 9:00 a.m. 6th February calculated from meter readings of pumps are shown in Table D.13 and Figure D.4, and total flow is summarized below.

Total Flows to the Treatment Plant (5th - 6st, February)

Name of P/S	Operating Hours (hrs)	Pump Capacity (m^3/hr)	Flow Rates (m^3/day)
Al-Mansura Main	9.2	1,014	9,329
Industrial Area	13.0	187	2,432
Total			11,761

A total flow of 11,761 m^3/day is larger than that of 8,992 m^3/day of the first sampling by 30 %. Industrial wastewater did not change significantly as domestic wastewater, 2,300 m^3/day for the first measurement and 2,432 m^3/day for the second. Domestic wastewater from Al-Mansura pumping station substantially increased from 6,692 m^3/day to 9,329 m^3/day, by 40 %. Since sampling days, from Monday to Tuesday for the first and from Sunday to Monday for the second, are normal week days in Aden, there seems no reason for this increase. One probable reason might be an infiltration of rain water into the sewerage system.

As can be seen in Figure D.4, flow in day time from 8:00 in the morning till 19:00 p.m. are larger than daily average flow with peak flow from 13:00 p.m. However, hourly peak flow of 1,743 m^3/hr between 13:00 and 14:00, 3.6 times the average flow, is unreasonably high. This indicates that pump operation dose not necessarily reflect the flow rates in the sewerage system. In order works, some storage capacity in sewers and pumping wells cause time lags. The same reason can explain a sudden rise of flow at 20:00 p.m. If these two rises are disregarded, peak flows for three hours from 14:00 to 17:00 are around two times the average flow.

3. Third Sampling in and around Tawahi Bay

Sampling of sea water in and around Tawahi Bay was conducted on 13th February, 1989 at five sampling points illustrated in Figure D.2. Two boats were arranged by YPA, a small boat for sailing on shallow water in Inner Harbor and a big pilot boat for inner bay and near harbor limit.

Results of the analysis made on the boat and in the laboratory are shown in Table D.7. Heavy pollution of sea water at the sampling point S1 is recognized in Transparency, Turbidity, DO, CODmn, NH_4-N and Number of Coliform. Values of these items indicate pollution caused by raw sewage discharged from Hedjuff ocean outfall, which discharge all the sewage in Ma'alla district in quantity. Odor of foul sewage was smelt around the sampling point. Although pollution of sea water is limited to a localized area at present, it will expand as quantity of sewage increases and accumulation of organic sediments will give impetus to total pollution in Inner Harbor.

Sea water from the sampling point S2, near Labor Island in Inner Harbor, shows pollution though it is not so significant as point S1. Turbidity, CODmn, NH_4-N and Number of Coliform are slightly higher than those of samples from points S3, S4 and S5 in outer harbor. Transparency is lower. However, these values are still of good quality sea water. Taking into the fact that there are no district causes of pollution other than sewage outfalls around Inner Harbor, sea water in this area represented by S2 point is influenced by sewages from Ma'alla and Tawahi.

Samples from points S3, S4 and S5, locating in line from off Hiswa to the harbor limit, shows no trace of organic pollution by any indicator analyzed.

Two samples from points S2 and S3 were brought back to Japan for the analysis of Kjeldahl nitrogen (K-N) and total phosphorus (T-P). Results are as follows.

Point	K-N(mg/l)	T-P(mg/l)
S2	0.16	0.07
S3	0.13	0.06

Above figures include not only dissolved nitrogen and phosphorus but also those of particles such as plankton. Kjeldahl nitrogen does not count nitrites and nitrates. Nevertheless, nitrogen and phosphorus levels presumed with these figures are in a normal range of non-polluted sea water.

Chloride ion concentrations measured in the sampling can be converted theoretically to salinities by the following equation.

$$S = 1.80655\ Cl^-$$

where S : Salinity

Cl^- : Chloride ion concentration

Salinites calculated from chloride ion concentrations are as follows. These values are in a range of normal sea water.

Salinity (%)

S1	S2	S3	S4	S5
34.3	35.8	36.0	36.0	36.0

4. Fourth and Fifth Samplings

4.1 Sampling Points

Fourth and fifth samplings were carried out on 20th/21st and 26th/27th February respectively in Ma'alla and Tawahi. Sampling points in Ma'alla are in Hedjuff pumping station. All the sewage produced in Ma'alla flows into this pumping station by two lines. Sewage from main line, one of the two lines, is pumped in the station to discharge into the sea by a outfall. Sewage flow in this line can be measured from operation records of the pumps. The other line connects to the outfall directly. There is no provision for flow measurement in this line. Considering this situation, two sampling points were set up, one on the main line for 24 hours sampling and another on the other line for grab samples.

There are four ocean outfalls in operation in Tawahi. The entire sewer service area is divided into four smaller areas by the outfalls. Sewage from these areas discharge into the sea by gravity. Moreover, down stream sewers near outfalls are influenced by sea water. Hence, flow measurement in sewer network is very difficult in Tawahi. Under this situation, two sampling points, which are considered to be most representative of Tawahi district, were selected for grab sampling. Sampling points in Ma'alla and Tawahi are as follows and illustrated in Figure D.3.

No. of Sampling Points	District	Location
M1	Ma'alla	Hedjuff P/S, inlet manhole of main line
M2	Ma'alla	Hedjuff P/S, junction point of C-class housing line
T1	Tawahi	Manhole in Bingisar line
T2	Tawahi	Alfath P/S

4.2 Results of Analysis

Results of the analysis for 24 hours sampling from point M1 are shown in Tables D.8 and D.10. Results of the analysis for two grab samples from points M2, T1 and T2 are shown in Tables D.9 and D.11. Methods of measurement and making composite samples are the same as those for raw sewage in Al Shaab STP mentioned in Section 2.2 of this appendix. Average temperatures and pH and weighted averages of CODmn, CODcr, BOD_5, SS and NH_4-N of the samples from the four points are as follows.

Point	Temperature (°C) ambient	Temperature (°C) water	pH	CODmn (mg/l)	CODcr (mg/l)	BOD_5 (mg/l)	SS (mg/l)	NH_4-N (mg/l)	K-N (mg/l)	T-P (mg/l)
M1										
1st	25.7	29.0	7.1	150	632	321	251	53	-	-
2nd	26.2	29.2	7.5	126	794	326	274	67	65	13
M2	26.6	29.3	7.1	211	1299	501	495	46	-	-
T1	27.0	28.7	7.2	274	1690	638	657	44	-	-
T2	27.2	29.9	7.3	174	1278	287	308	44	-	-

Note: M1; Figures are weighted averages of 24 hours sampling
M2, T1 and T2; Averages of 4 grab samples

Average concentrations of CODmn, CODcr, BOD_5 and SS of the grab samples from points M2, T1 and T2 are generally higher than the averages of those of 24 hours sampling from point M1. Main reason for this is due to the fact that grab samples were taken at peak flow hours when concentrations of these items are high. In addition to this, it should be taken into account that service area of the point M1 is largest among the four points, and the other three service areas are far smaller than that of M1. Consequently, sewage flow at point M1 accounts for more than half of the total flow in Ma'alla and Tawahi. Therefore, to discuss the representative characteristics of raw sewage in the project area, figures obtained from Point M1 should be emphasized.

In order to examine the general characteristics of the raw sewage in Aden, concentrations of the main items obtained from 24 hours samples at Al Shaab STP and point M1 are compared. Around the clock sampling were carried out twice at both sampling points. Average concentrations of CODmn, BOD_5, SS, NH_4-N, K-N and T-P from those samplings are as follows.

	CODmn (mg/l)	BOD_5 (mg/l)	SS (mg/l)	NH_4-N (mg/l)	K-N (mg/l)	T-P (mg/l)
Average Al Shaab STP (No.1)	155	356	373	44	62	11
Average Hedjuff P/S (M1)	138	324	263	60	65	13

Although raw sewage to the Al Shaab STP includes industrial wastewater, at most 25 % of the total flow, distinct differences are not recognized between the figures in the above table. Sewages to Al Shaab STP and to Hedjuff P/S are considered to be domestic origin. Figures in the above are higher than those of the industrialized countries such as Japan, but still in a normal range of domestic sewage. High concentrations of the above items are mainly due to inhabitants low water consumption. In consideration of the results of the analysis and situation of the existing sewerage system connected to Al Shaab STP and Hedjuff P/S, the above figures can be taken as representative ones in Greater Aden and are to be a basis for future projection for the project.

4.3 Flow Rates to Hedjuff P/S

Flow rates to Hedjuff P/S (point M1) on both sampling days were measured with pump operation record. Operating time in every one hour was read for 24 hours. Readings and flow rates calculated with readings are shown in Table D.13 and Figure D.5. Total flows, hourly peak and average are as follows.

	1st (20/21 Feb.)		2nd (26/27 Feb.)	
Total Flow (m3/day)	3,968		4,522	
Hourly Average (m3/hr)	169		188	
Hourly Peak (m3/hr)	402	(2.38)	352	(1.87)

Note: Figures in parentheses are peaking ratios (peak/average)

Total flows to Hedjuff P/S were 3,968 and 4,522 m3/day for the first and second measurements respectively. These figures were converted to hourly averages of 169 and 188 m3/hr. On the other hand, peak hourly flow read on the meter were 402 and 352 m3/hr respectively. Peaking factors calculated based on those figures are 2.38 and 1.87.

Comparing hourly flow fluctuations of Al Shaab STP and Hedjuff P/S, almost same pattern was recognized. Sewage flow starts to increase from 7:00 and reches to its highest in the evening from 14:00 to 17:00. Flow goes down from peak hours gradually to around 21:00 and becomes almost negligible from then to 6:00 in the morning. Peaking factor, a ratio of peak flow to average flow, is around 2.

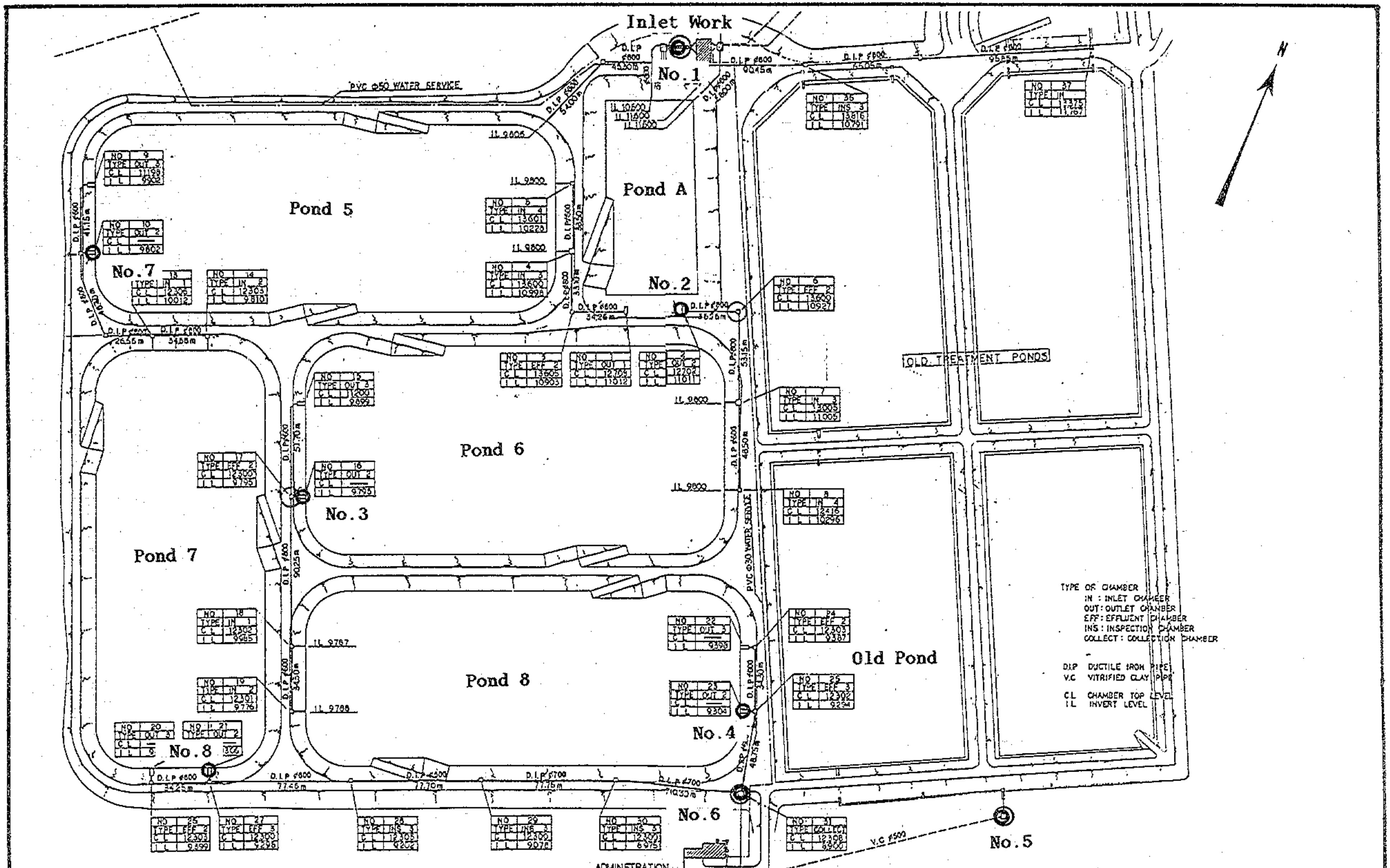

Fig. D.1 Sampling Points at Al-Shaab STP

Table D.1 Characteristics of Raw Sewage (1)

Date: 30th and 31st January, 1989
Sampling Point: Inlet work, Al-Mansura STP (No.1 in Figure D.1)

Time	Temperature(°C)		pH	DO (mg/l)	CODmn (mg/l)	CODcr (mg/l)	BOD5 (mg/l)	SS (mg/l)	NH4-N (mg/l)	Flow *2 (m3/s)
	ambient	water								
9:00	26.0	30.1	7.6	0						0.33
10:00	28.0	30.6	7.9	0	*1 172	*1 625	*1 330	*1 310	*1 53	0.35
11:00	29.0	31.9	8.8	0.2						0.52
12:00	29.0	30.1	7.9	0						0.22
13:00	29.0	29.3	6.8	0.3	215	1000	460	410	50	0.15
14:00	29.0	30.1	7.2	0.2						0.16
15:00	29.0	29.6	6.9	0.4						0.22
16:00	28.0	29.5	6.8	0.5	217	1110	520	510	34	0.14
17:00	28.0	29.5	6.9	0.3						0.30
18:00	26.0	29.6	7.0	0.2						0.21
19:00	25.0	29.4	7.2	0.4	161	643	fail	400	34	0.15
20:00	24.5	29.6	7.2	0						0.22
21:00	24.1	29.2	6.8	0						0.26
22:00	24.0	29.4	6.9	0.2	171	468	350	300	38	0.17
23:00	23.5	29.3	7.2	0						0.05
24:00	23.0	29.1	7.5	0						0.21
1:00	22.5	29.3	7.4	0	158	591	350	270	45	0.12
2:00	22.5	29.4	7.6	0						0.15
3:00	22.0	28.6	7.8	0						0.21
4:00	21.8	28.4	7.7	0	128	503	260	170	47	0.02
5:00	22.0	27.4	8.1	0						0.14
6:00	22.0	30.5	7.6	0						0.04
7:00	22.0	29.4	7.4	0						0.04
8:00	23.0	31.2	7.0	0	136	1240	fail	200	39	0.21
9:00	25.0	29.4	7.4	0						0.22

Note: *1 Composite sample of four hours.
*2 Average flows at sampling time, used only for composite sample.

Table D.4 Characteristics of Raw Sewage (2)

Date: 5th and 6th February, 1989
Sampling Point: Inlet work, Al-Mansura STP (No.1 in Figure D.1)

Time	Temperatue (°C)		pH	DO (mg/l)	CODmn (mg/l)	CODcr (mg/l)	BOD5 (mg/l)	SS (mg/l)	NH4-N (mg/l)	Flow *2 (m3/s)
	ambient	water								
9:00	26.0	30.3	7.9	0						0.14
10:00	27.0	30.2	7.3	0	*1 166	*1 1010	*1 530	*1 480	*1 48	0.21
11:00	27.5	29.3	7.5	0						0.15
12:00	25.5	29.7	7.6	0						0.12
13:00	25.0	30.1	7.7	0	183	1070	440	730	41	0.11
14:00	25.5	29.9	8.6	0						0.07
15:00	29.0	27.8	8.8	0						0.06
16:00	27.5	27.8	8.7	0	183	599	300	1050	25	0.05
17:00	26.0	27.9	8.7	1.0						0.10
18:00	25.1	28.2	7.8	0						0.05
19:00	24.8	28.3	7.9	0	153	648	340	550	33	0.05
20:00	24.8	29.1	7.1	0.6						0.26
21:00	24.7	28.3	7.9	0						0.15
22:00	24.5	29.7	7.5	0	95	289	210	150	47	0.22
23:00	24.0	28.4	8.9	0						0
24:00	24.0	28.4	8.0	0						0.03
1:00	23.5	28.5	7.8	0	102	435	250	140	49	0.01
2:00	23.2	28.8	7.7	0						0.03
3:00	23.3	29.4	7.4	0						0.11
4:00	23.4	28.5	7.6	0	96	559	200	120	48	0.09
5:00	24.0	28.2	7.7	0						0.05
6:00	23.0	29.8	7.5	0						0.17
7:00	22.0	28.6	7.4	0						0.19
8:00	22.5	29.0	7.4	0	112	518	260	230	50	0.14
9:00	23.0	29.8	7.6	0						0.23

Note: *1 Composite sample of four hours.
*2 Average flows at sampling time, used only for composite sample.

Table D.2 Characteristics of Stabilization Pond Water (1)

Date: 30th and 31st January,1989
Sampling Points: Outlets of anaerobic, 1st and 2nd facultative ponds and old pond (No.2, No.3, No.4, No.5 and No.6 on Figure D.1)

Sampling Point	Time	Temperature(°C)		pH	CODmn (mg/l)	CODcr (mg/l)	BOD5 (mg/l)	SS (mg/l)	NH4-N (mg/l)	Cl (mg/l)
		ambient	water							
Pond A (No.2)	17:00	28.0	27.8	7.7	118	405	140	88	33	-
	6:00	22.0	26.1	7.6	100	343	130	100	30	-
Pond 6 (No.3)	17:00	28.0	26.4	8.8	88	266	57	220	29	-
	6:00	22.0	24.7	8.5	88	278	43	220	26	-
Pond 8 (No.4)	17:00	28.0	25.6	8.8	(37) 77	(95) 252	(17) 78	170	11	470
	6:00	22.0	24.3	8.4	(33) 80	(116) 237	(26) 59	150	9	470
Old Pond (No.5)	17:00	28.0	25.7	8.9	(26) 113	(53) 378	(7) 30	320	20	460
	6:00	22.0	24.6	8.6	(24) 107	(70) 406	(11) 33	320	21	450
Pond 7,8 (No.6)	17:00	28.0	25.4	8.8	(27) 94	-	(7) 58	230	14	470
	6:00	22.0	23.9	8.6	(28) 85	-	(8) 66	200	14	470

Note: Figures in pharentheses are filtered samples.

Table D.3 DO Level Fluctuation in Stabilization Ponds (1)

Date: 30th and 31st January, 1989
Measuring Points: Outlets of Pond A, Pond 6 and Pond 8 (No.2 No.3 and No.4 on Figure D.1)

Pond and Sampling Point	Time	Upper Layer		Lower Layer	
		Temp.(°C)	DO (mg/l)	Temp.(°C)	DO (mg/l)
Pond A (No.2)	17:00	27.8	0	27.5	0
	6:00	26.1	0	26.4	0
Pond 6 (No.3)	9:00	25.0	0	25.2	1.0
	11:00	25.8	1.7	25.6	1.0
	13:00	26.5	3.9	26.4	2.4
	15:00	26.9	5.6	26.7	3.0
	17:00	26.4	3.2	26.4	2.5
	19:00	26.0	0.7	26.0	0
	21:00	25.6	0	25.6	0
Pond 8 (No.4)	9:00	24.1	0	24.1	0
	11:00	24.6	1.7	24.5	0.8
	13:00	25.1	1.3	25.0	0.7
	15:00	25.9	3.6	26.0	3.6
	17:00	25.6	2.2	25.9	1.6
	19:00	25.6	0.9	25.6	0.7
	21:00	25.4	0	25.4	0

Table D.5 Characteristics of Stabilization Pond Water (2)

Date: 5th and 6th February, 1989
Sampling Points: Outlets of anaerobic, and 1st and 2nd facultative ponds (No.2, No.3, No.4, No.7 and No.8 on Figure D.1)

Sampling Point	Time	Temperature(°C)		pH	CODmn (mg/l)	CODcr (mg/l)	BOD5 (mg/l)	SS (mg/l)	NH4-N (mg/l)	Cl (mg/l)
		ambient	water							
Pond A (No.2)	17:00	26.0	27.8	7.6	107	*1	170	230	45	-
	6:00	23.0	26.4	7.6	102	352	110	90	46	-
Pond 6 (No.3)	17:00	26.0	26.2	8.5	91	*1	39	200	35	-
	6:00	23.0	25.2	8.4	88	283	24	210	38	-
Pond 8 (No.4)	17:00	26.0	25.8	8.4	(27) 66	*1	(7.3) 37	140	15	470
	6:00	23.0	24.8	8.4	(26) 69	(81) 213	(7.9) 41	150	16	470
Pond 5 (No.7)	17:00	26.0	26.3	8.5	94	*1	36	160	36	-
	6:00	23.0	25.1	8.3	95	362	40	76	36	-
Pond 7 (No.8)	17:00	26.0	25.9	8.4	(25) 89	*1	(4.2) 62	91	20	460
	6:00	23.0	24.8	8.4	(26) 94	(117) 296	(3.8) 46	89	20	460

Note: Figures in pharentheses are filtered samples.
*1 composite sample of 17:00 and 6:00

Table D.6 DO Level Fluctuation in Stabilization Ponds (2)

Date: 5th and 6th February, 1989
Measuring Points: Outlets of Pond A, Ponds 6, 8, 5 and 7 (Nos. 3, 4, 7 and 8 on Figure D.1)

Pond and Sampling Point	Time	Upper Layer		Lower Layer	
		Temp.(°C)	DO (mg/l)	Temp.(°C)	DO (mg/l)
Pond A (No.2)	17:00	27.8	0	27.8	0
	6:00	26.4	0	26.5	0
Pond 6 (No.3)	9:00	25.9	0	26.0	0
	11:00	26.3	0	26.3	0
	13:00	26.2	0	26.3	0
	15:00	26.6	0	26.6	0
	17:00	26.2	0	26.3	0
	19:00	26.0	0	26.0	0
Pond 8 (No.4)	9:00	25.3	0	25.4	0
	11:00	25.8	0.4	25.8	0.3
	13:00	25.7	0.7	25.8	0.3
	15:00	25.8	0.5	25.8	0.3
	17:00	25.8	0.2	25.9	0
	19:00	25.4	0.2	25.7	0
	21:00	25.4	0	25.6	0
Pond 5 (No.7)	9:00	26.0	0	26.1	0
	11:00	26.4	0	26.4	0
	13:00	26.2	0	26.3	0
	15:00	26.5	0	26.5	0
	17:00	26.3	0	26.3	0
	19:00	26.1	0	26.1	0
Pond 7 (No.8)	9:00	25.5	0.5	25.6	0
	11:00	25.9	1.4	25.9	0.3
	13:00	25.7	1.7	25.8	1.6
	15:00	26.0	1.2	26.0	1.1
	17:00	25.9	1.0	26.0	0.9
	19:00	25.4	0	25.7	0
	21:00	25.1	0	25.5	0

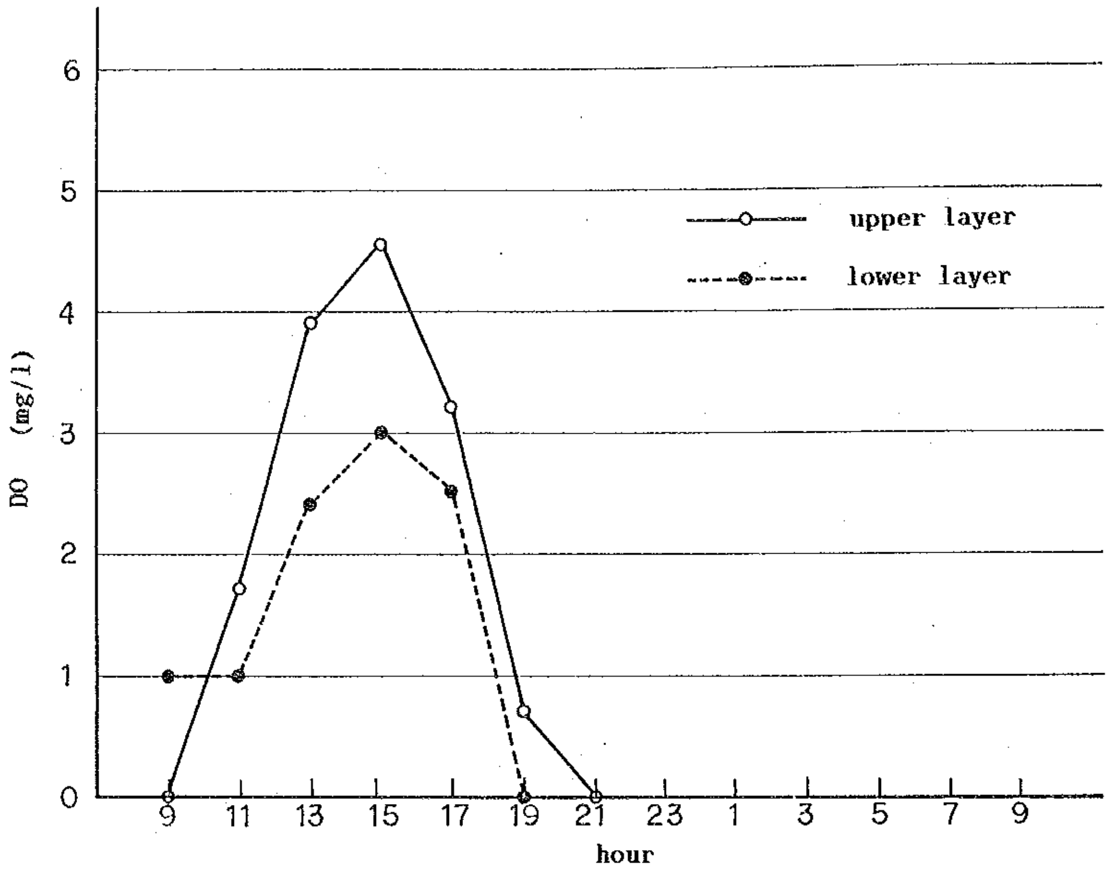

Fig. D.6 DO Level Fluctuation in Pond 6

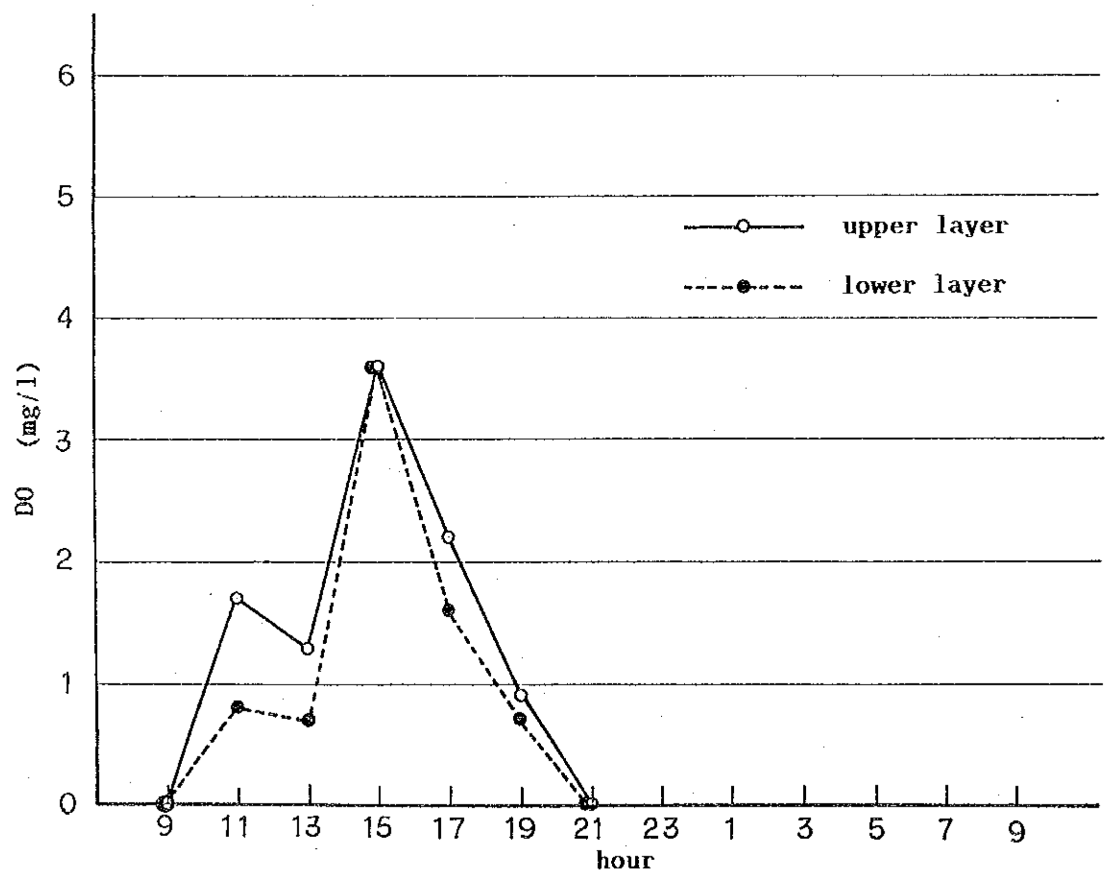

Fig. D.7 DO Level Fluctuation in Pond 8

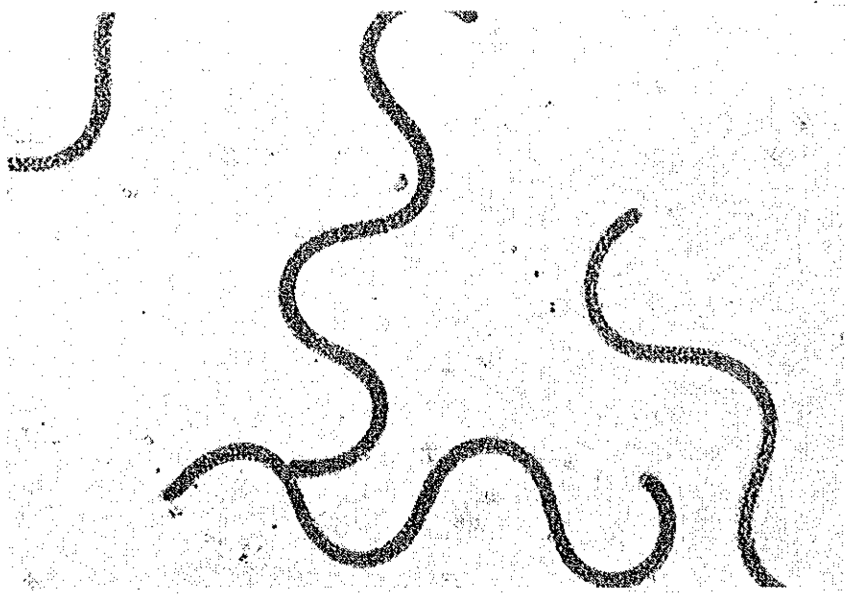

Photo D.1

Pond 7
Lyngbya sp.
(x400)

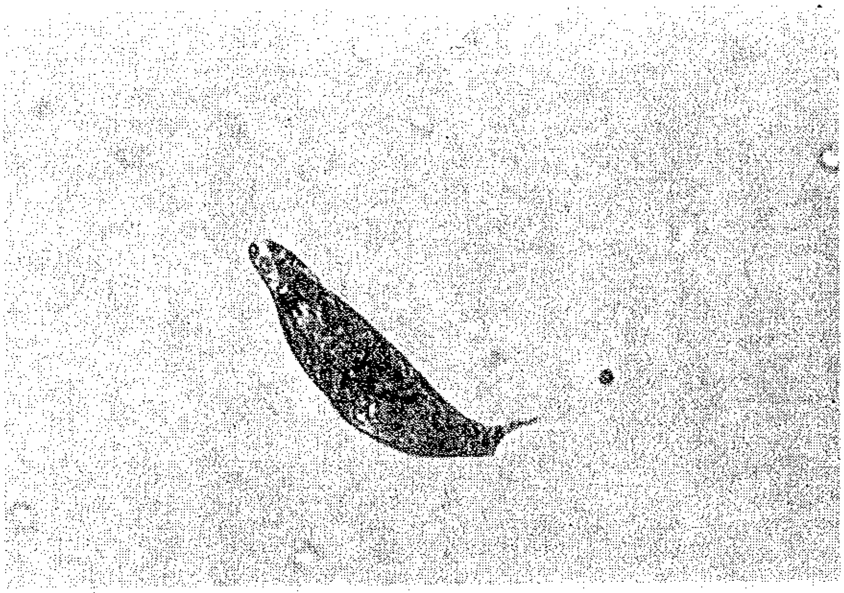

Photo D.2

Pond 7
Euglena sp.
(x800)

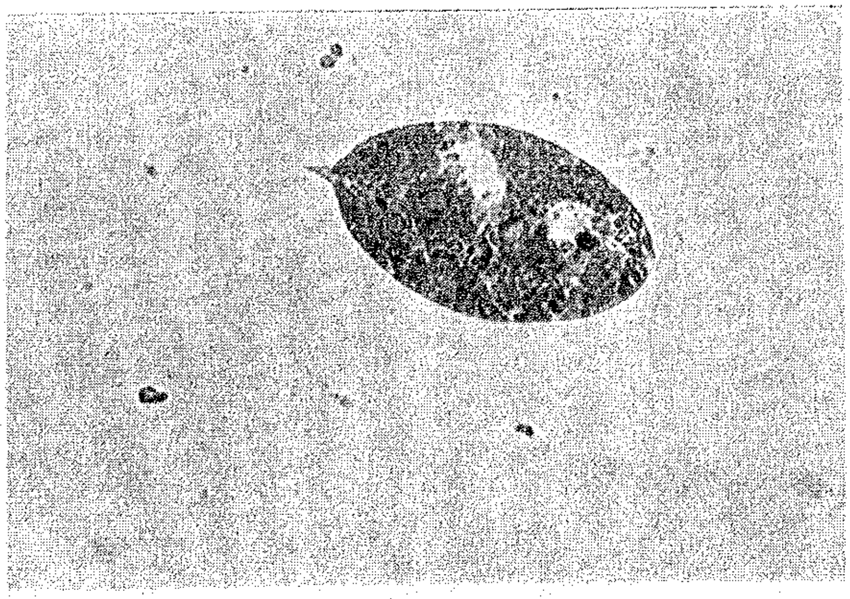

Photo D.3

Pond 7
Euglena sp.
(x800)

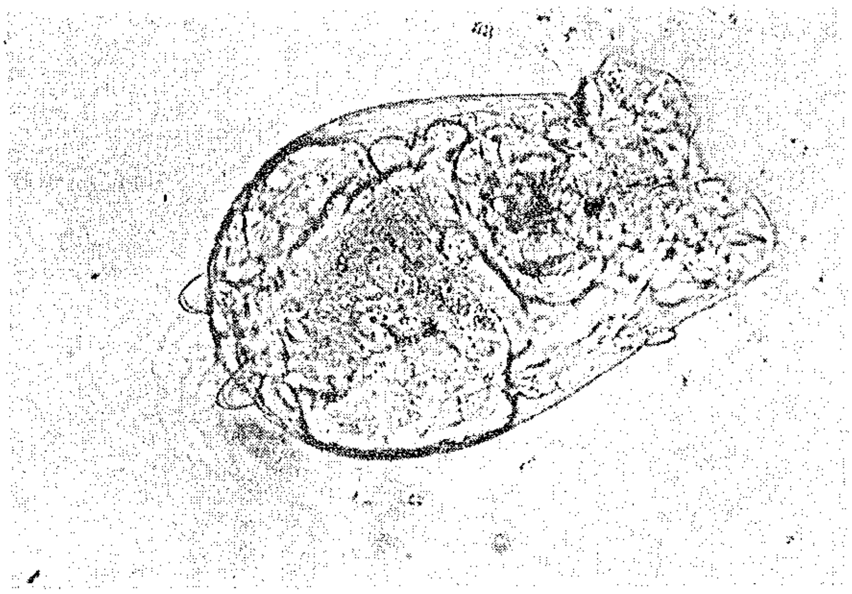

Photo D.4

Pond 7
Euchlanis sp.
(x400)

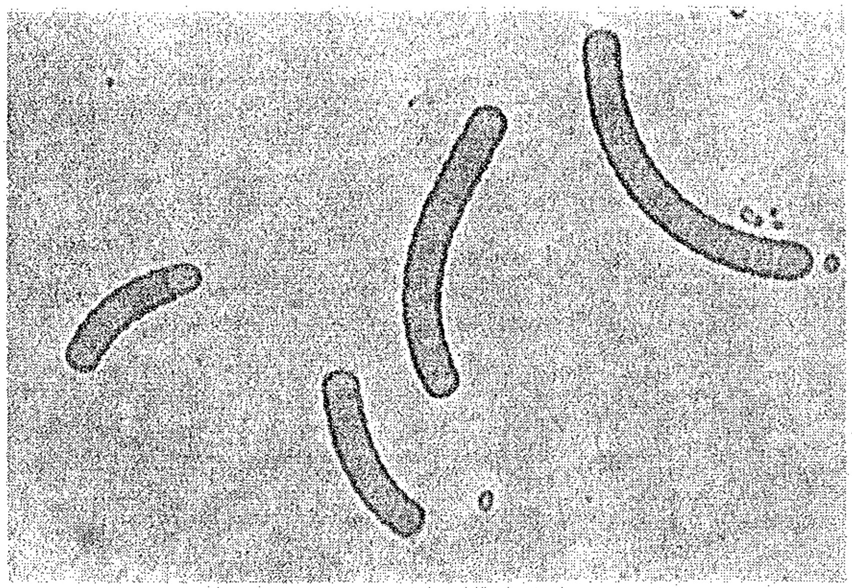

Photo D.5

Pond 8
Hormidium sp.
(x800)

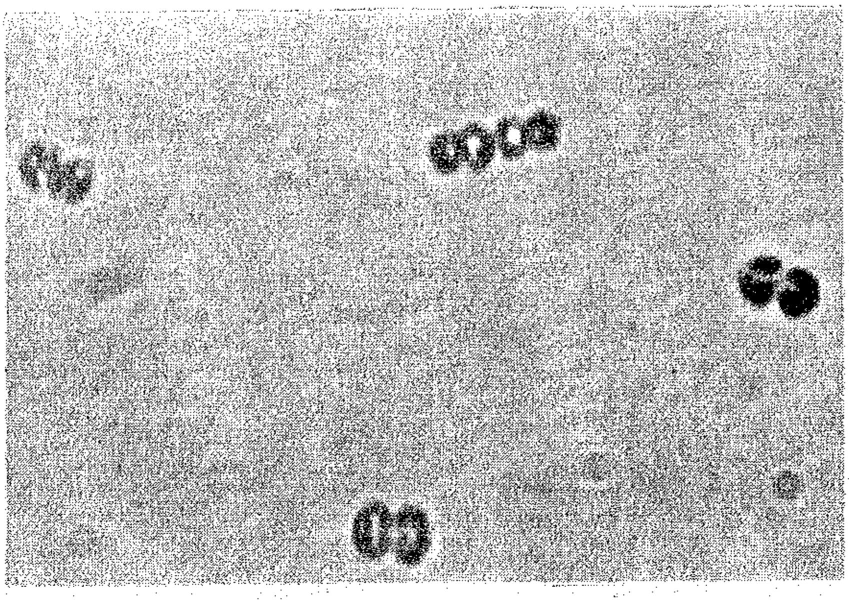

Photo D.6

Old Pond
Chromatium sp.
(x4,000)

Table D.12 Sewage Flow to Al-Shaab STP

Time and Date: 9:00 am 5th to 9:00 am 6th, February, 1989

Time	from Al-Mansura Main PS		from Industrial Area PS		Total Flow (m3)
	OH (hr)	(m3)	OH (hr)	Q (m3)	
9:00 - 10:00	0.5	507	0.2	37	544
10:00 - 11:00	0.4	406	1.6	299	705
11:00 - 12:00	0.4	406	0.7	131	537
12:00 - 13:00	0.5	507	0.8	150	657
13:00 - 14:00	1.7	1724	0.1	19	1743
14:00 - 15:00	0.7	710	1.2	224	934
15:00 - 16:00	0.5	507	1.8	337	844
16:00 - 17:00	0.9	913	0.7	131	1044
17:00 - 18:00	0.5	507	0.4	75	582
18:00 - 19:00	0.4	406	0.5	94	500
19:00 - 20:00	-	-	0.7	131	131
20:00 - 21:00	0.9	913	0.9	168	1081
21:00 - 22:00	0.1	101	-	-	101
22:00 - 23:00	-	-	0.5	94	94
23:00 - 24:00	-	-	0.1	19	19
24:00 - 1:00	-	-	0.3	56	56
1:00 - 2:00	0.1	101	0.2	37	138
2:00 - 3:00	-	-	0.1	19	19
3:00 - 4:00	0.1	101	0.1	19	120
4:00 - 5:00	0.1	101	0.2	37	138
5:00 - 6:00	0.1	101	0.3	56	157
6:00 - 7:00	0.1	101	0.6	112	213
7:00 - 8:00	0.3	304	0.4	75	379
8:00 - 9:00	0.9	913	0.6	112	1025
Total	9.2	9329	13.0	2432	11761

Note: Pumping capacity; Al-Mansura Main 1014 m3/hr
Industrial Area 187 m3/hr

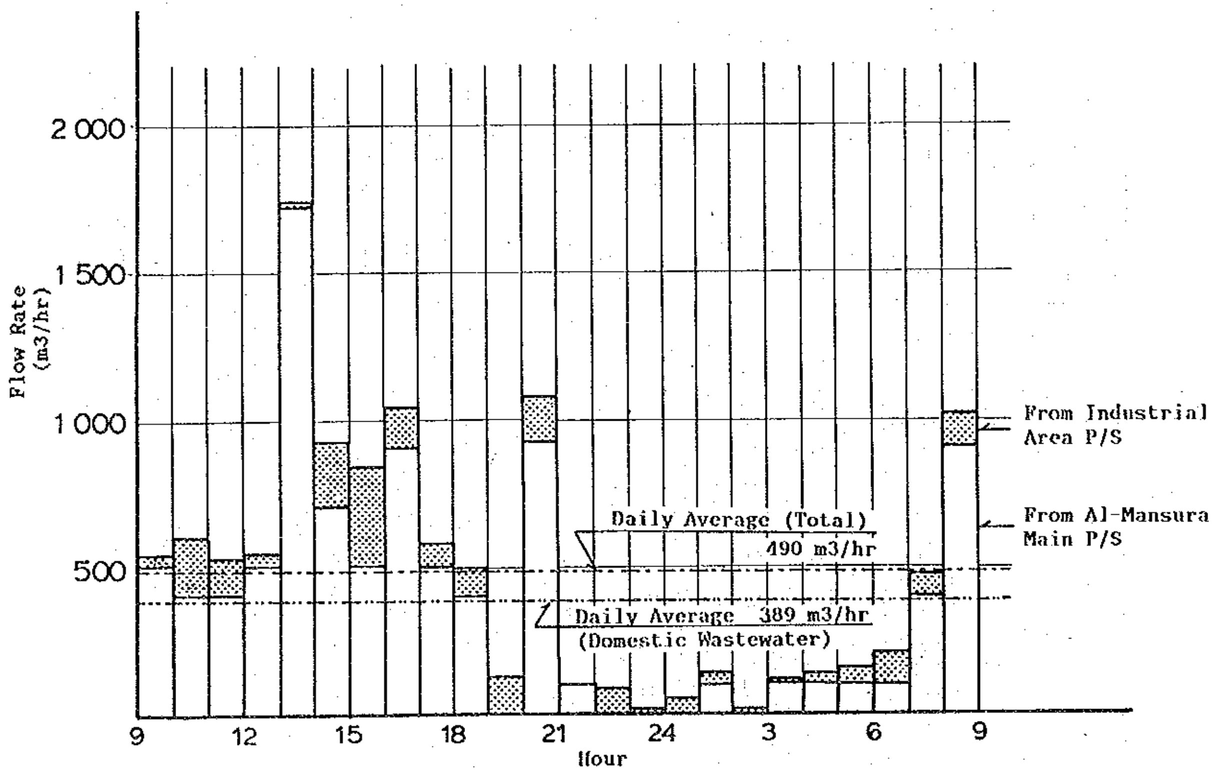

Fig. D.4 Sewage Flow Fluctuation at Al-Shaab STP

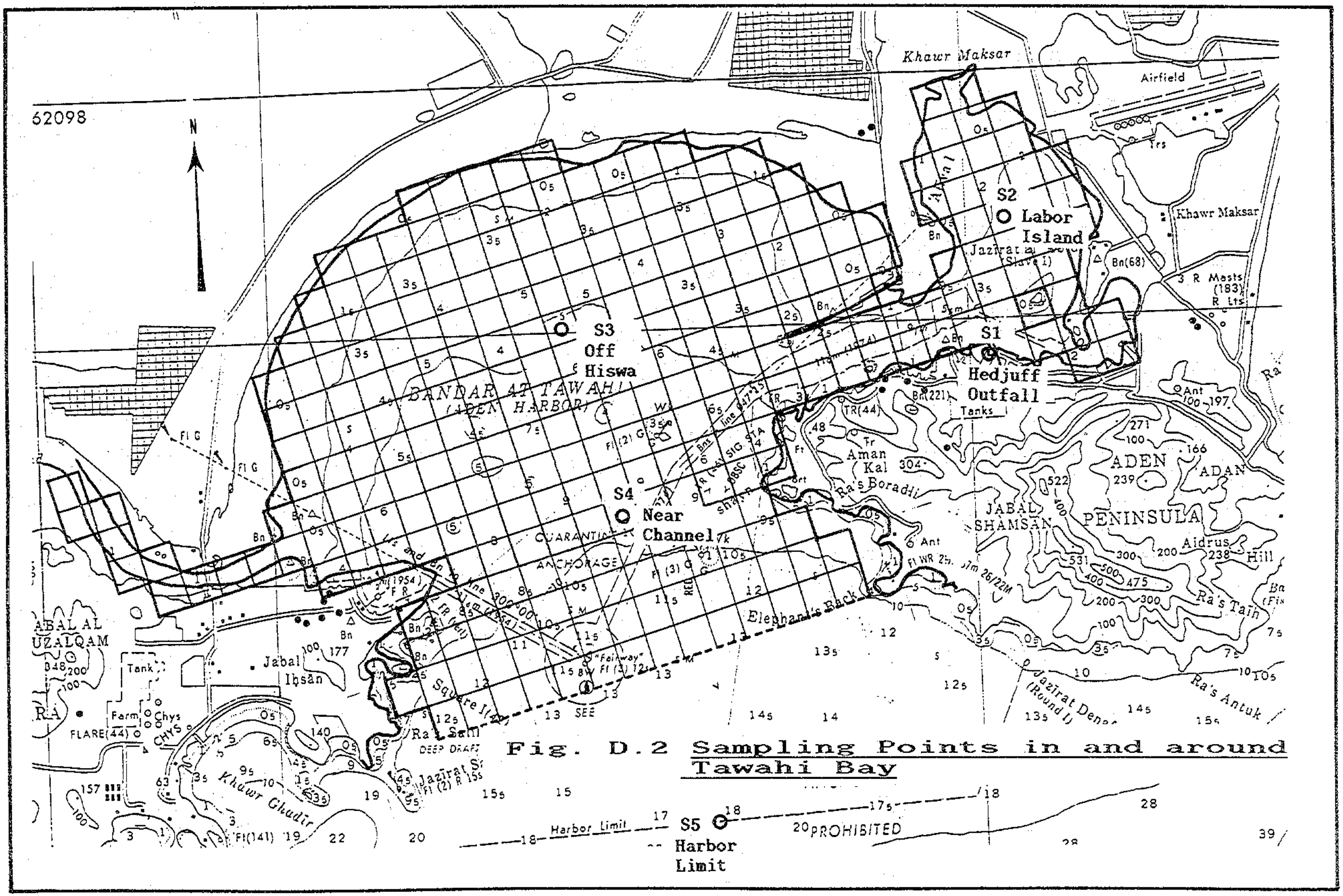

Fig. D.2 Sampling Points in and around Tawahi Bay

Table D.7 Characteristics of Sea Water (Tawahi Bay)

Date: 13th February, 1989
Sampling Points: In and around Tawahi Bay (Nos. S1, S2, S3, S4 and S5 in Figure D.1)

Sampling Point	S1 Hedjuff Outfall	S2 Labor Island	S3 Off Hiswa	S4 Near Channel	S5 Harbor Limit
Time	9:50	10:04	10:40	10:55	11:20
Water Depth (m)	3.4	3.9	-	11.5	22
Ambient Temperature (°C)	30.0	30.0	30.5	28.0	28.0
Water Temperature (°C)	26.0	25.5	26.0	26.0	26.0
Wind Direction	SE	SSW	S	SE	S
Transparency (m)	0.6	1.7	5.5	5.0	9.5
Turbidity (mg/l)	10	3	1	1	1
pH	7.2	8.0	8.2	8.2	8.4
DO (mg/l)	0.2	6.7	6.5	6.3	6.5
CODmn (mg/l)	5.9	1.8	1.4	0.9	0.8
CODcr (mg/l)	23.4	-	5.3	-	1.5
NH4-N (mg/l)	4.0	0.3	0.2	0.2	0.2
Cl (‰)	19.0	19.8	19.9	19.9	19.9
Coliform (Nos./ml)	40	20	2	3	2

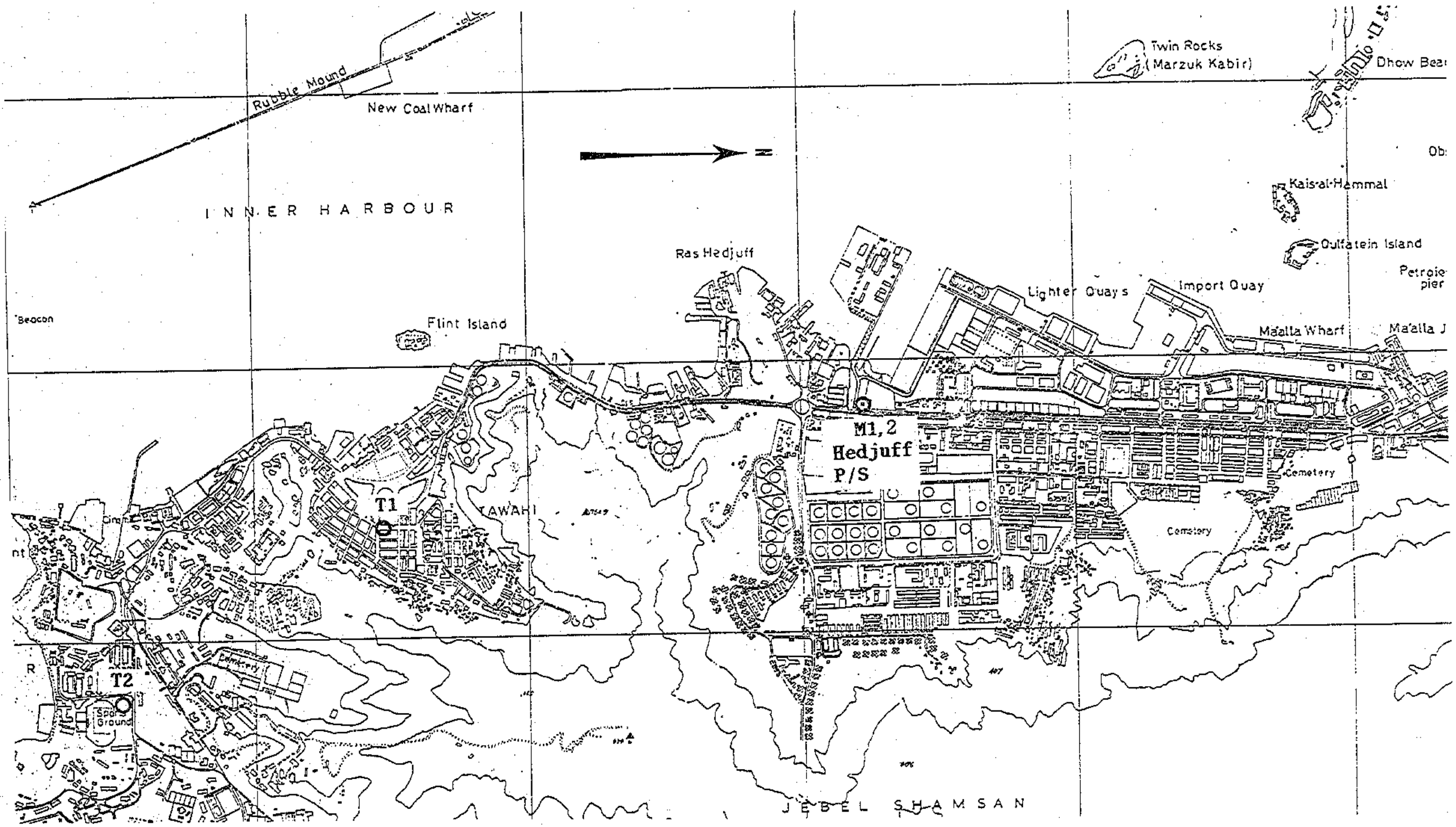

Fig. D.3 Sampling Points in Ma'alla and Tawahi

Table D.8 Characteristics of Raw Sewage (3)

Date: 20th and 21st February, 1989
Sampling Point: Hedjuff P/S (No. M1 in Figure D.3)

Time	Temperature(°C)		pH	DO (mg/l)	CODmn (mg/l)	CODcr (mg/l)	BOD5 (mg/l)	SS (mg/l)	NH4-N (mg/l)	Cl (mg/l)
	ambient	water								
9:00	26.0	28.5	6.9	0	179	795	396	300	47	350
10:00	26.0	28.5	7.0	0						
11:00	26.5	28.8	7.1	0						
12:00	27.0	28.9	7.1	0	139	554	309	210	47	360
13:00	27.0	29.1	7.2	0						
14:00	27.5	29.2	7.0	0						
15:00	27.5	29.1	7.2	0	146	530	292	210	45	360
16:00	27.0	29.5	7.3	0						
17:00	26.5	29.2	7.4	0						
18:00	26.0	28.9	7.4	0	130	657	270	220	46	350
19:00	26.0	29.0	7.5	0						
20:00	-	-	-	-						
21:00	25.0	29.1	7.4	0	142	609	324	200	53	370
22:00	25.0	28.5	7.0	0						
23:00	25.0	28.6	7.1	0						
24:00	25.0	28.9	7.5	0	105	348	193	120	55	380
1:00	25.0	28.7	7.6	0						
2:00	25.0	29.2	7.6	0						
3:00	25.0	29.3	8.0	0	103	267	141	87	58	510
4:00	24.5	29.7	7.8	0						
5:00	24.5	29.6	8.0	0						
6:00	24.5	29.3	7.8	0	149	617	315	330	90	510
7:00	24.5	28.4	7.6	0						
8:00	25.0	28.4	7.4	0						
9:00	26.0	28.9	7.3	0						

Table D.10 Characteristics of Raw Sewage (5)

Date: 26th and 27th February, 1989
Sampling Point: Hedjuff P/S (No. M1 in Figure D.3)

<table>
<tr><th rowspan="2">Time</th><th colspan="2">Temperature(°C)</th><th rowspan="2">pH</th><th rowspan="2">DO (mg/l)</th><th rowspan="2">CODmn (mg/l)</th><th rowspan="2">CODcr (mg/l)</th><th rowspan="2">BOD5 (mg/l)</th><th rowspan="2">SS (mg/l)</th><th rowspan="2">NH4-N (mg/l)</th><th rowspan="2">Cl (mg/l)</th></tr>
<tr><th>ambient</th><th>water</th></tr>
<tr><td>9:00</td><td>25.5</td><td>28.7</td><td>7.1</td><td>0</td><td rowspan="3">123</td><td rowspan="3">928</td><td rowspan="3">336</td><td rowspan="3">340</td><td rowspan="3">68</td><td rowspan="3">380</td></tr>
<tr><td>10:00</td><td>26.5</td><td>28.9</td><td>7.1</td><td>0</td></tr>
<tr><td>11:00</td><td>27.0</td><td>29.3</td><td>7.4</td><td>0</td></tr>
<tr><td>12:00</td><td>27.5</td><td>29.7</td><td>7.4</td><td>0</td><td rowspan="3">122</td><td rowspan="3">945</td><td rowspan="3">338</td><td rowspan="3">300</td><td rowspan="3">55</td><td rowspan="3">370</td></tr>
<tr><td>13:00</td><td>27.5</td><td>29.8</td><td>7.4</td><td>0</td></tr>
<tr><td>14:00</td><td>27.5</td><td>29.9</td><td>7.4</td><td>0</td></tr>
<tr><td>15:00</td><td>27.5</td><td>29.6</td><td>7.3</td><td>0</td><td rowspan="3">101</td><td rowspan="3">681</td><td rowspan="3">300</td><td rowspan="3">230</td><td rowspan="3">52</td><td rowspan="3">360</td></tr>
<tr><td>16:00</td><td>27.8</td><td>29.6</td><td>7.6</td><td>0</td></tr>
<tr><td>17:00</td><td>27.0</td><td>29.6</td><td>7.6</td><td>0</td></tr>
<tr><td>18:00</td><td>26.8</td><td>29.3</td><td>7.6</td><td>0</td><td rowspan="3">126</td><td rowspan="3">675</td><td rowspan="3">307</td><td rowspan="3">210</td><td rowspan="3">61</td><td rowspan="3">370</td></tr>
<tr><td>19:00</td><td>26.5</td><td>29.3</td><td>7.5</td><td>0</td></tr>
<tr><td>20:00</td><td>26.0</td><td>29.0</td><td>7.4</td><td>0</td></tr>
<tr><td>21:00</td><td>26.0</td><td>28.9</td><td>7.2</td><td>0</td><td rowspan="3">119</td><td rowspan="3">638</td><td rowspan="3">334</td><td rowspan="3">190</td><td rowspan="3">72</td><td rowspan="3">390</td></tr>
<tr><td>22:00</td><td>26.0</td><td>28.7</td><td>7.4</td><td>0</td></tr>
<tr><td>23:00</td><td>25.5</td><td>28.8</td><td>7.4</td><td>0</td></tr>
<tr><td>24:00</td><td>25.5</td><td>29.0</td><td>7.6</td><td>0</td><td rowspan="3">101</td><td rowspan="3">538</td><td rowspan="3">255</td><td rowspan="3">130</td><td rowspan="3">80</td><td rowspan="3">400</td></tr>
<tr><td>1:00</td><td>-</td><td>-</td><td>-</td><td>-</td></tr>
<tr><td>2:00</td><td>-</td><td>-</td><td>-</td><td>-</td></tr>
<tr><td>3:00</td><td>25.0</td><td>29.8</td><td>8.2</td><td>0</td><td rowspan="3">99</td><td rowspan="3">564</td><td rowspan="3">222</td><td rowspan="3">180</td><td rowspan="3">78</td><td rowspan="3">440</td></tr>
<tr><td>4:00</td><td>-</td><td>-</td><td>-</td><td>-</td></tr>
<tr><td>5:00</td><td>-</td><td>-</td><td>-</td><td>-</td></tr>
<tr><td>6:00</td><td>24.5</td><td>29.6</td><td>7.9</td><td>0</td><td rowspan="4">178</td><td rowspan="4">814</td><td rowspan="4">362</td><td rowspan="4">340</td><td rowspan="4">90</td><td rowspan="4">410</td></tr>
<tr><td>7:00</td><td>25.0</td><td>28.5</td><td>7.7</td><td>0</td></tr>
<tr><td>8:00</td><td>25.0</td><td>28.3</td><td>7.5</td><td>0</td></tr>
<tr><td>9:00</td><td>25.5</td><td>28.9</td><td>7.4</td><td>0</td></tr>
</table>

Table D.9 Characteristics of Raw Sewage (4)

Date: 20th and 21st February,1989
Sampling Points: Ma'alla and Tawahi
(Nos. M2, T1, T2 in Figure D.3)

Sampling Point	Time	Tem.(°C)		pH	DO (mg/l)	CODmn (mg/l)	CODcr (mg/l)	BOD5 (mg/l)	SS (mg/l)	NH4-N (mg/l)	Cl (mg/l)
		ambi	watr								
M 2	10:00	26.0	28.4	7.0	0	233	938	490	480	50	360
	14:00	27.5	29.4	7.3	0	216	-	431	380	35	370
T 1	10:00	26.0	27.9	7.2	0	334	1570	738	1090	48	380
	15:00	29.1	29.0	7.4	0	197	-	359	330	34	370
T 2	10:00	26.0	29.0	7.4	0	178	836	360	330	50	340
	15:00	29.1	30.3	7.6	0	126	-	168	140	38	350

Table D.11 Characteristics of Raw Sewage (6)

Date: 26th and 27th February,1989
Sampling Points: Ma'alla and Tawahi
(Nos. M2, T1, T2 in Figure D.3)

Sampling Point	Time	Tem.(°C)		pH	DO (mg/l)	CODmn (mg/l)	CODcr (mg/l)	BOD5 (mg/l)	SS (mg/l)	NH4-N (mg/l)	Cl (mg/l)
		ambi	watr								
M 2	10:00	26.5	29.0	7.1	0	228	1660	650	640	51	370
	15:00	26.5	30.0	7.1	0	166	-	434	480	47	360
T 1	10:00	26.5	28.4	7.1	0	259	1810	625	670	52	380
	15:00	26.5	29.6	7.2	0	307	-	828	540	43	370
T 2	10:00	27.0	29.7	6.9	0	281	1720	370	550	41	370
	15:00	26.5	30.6	7.3	0	109	-	248	210	47	380

Table D.13 Sewage Flow to Hedjuff P/S

Time and Date: 9:00 am to 9:00 am 20/21, 26/27, February, 1989

Time \ Date	20th-21st, February, 1989		26th-27th, February, 1989	
	OH (hr)	Q (m3)	OH (hr)	Q (m3)
9:00 - 10:00	0.5	251	0.6	301
10:00 - 11:00	0.7	352	0.5	251
11:00 - 12:00	0.7	352	0.6	301
12:00 - 13:00	0.7	352	0.6	301
13:00 - 14:00	0.8	402	0.7	352
14:00 - 15:00	0.4	201	0.7	352
15:00 - 16:00	0.4	201	0.6	301
16:00 - 17:00	0.7	352	0.7	352
17:00 - 18:00	0.6	301	0.5	251
18:00 - 19:00	0.6	301	0.-	0
19:00 - 20:00	-	301	0.	0
20:00 - 21:00	0.6		0.3	151
21:00 - 22:00	-	-	0.4	201
22:00 - 23:00	-	-	0.3	151
23:00 - 24:00	-	-	0.4	201
24:00 - 1:00	-	-	-	151
1:00 - 2:00	-	-	-	
2:00 - 3:00	-	-	0.3	
3:00 - 4:00	-	-	0.3	151
4:00 - 5:00	-	-		
5:00 - 6:00	-	-		
6:00 - 7:00	-	-	0.3	151
7:00 - 8:00	0.6	301	0.7	352
8:00 - 9:00	0.6	301	0.5	251
Total	7.9	3968	9.0	4522

Note: Pumping capacity; Hedjuff P/S 502.4m3/hr

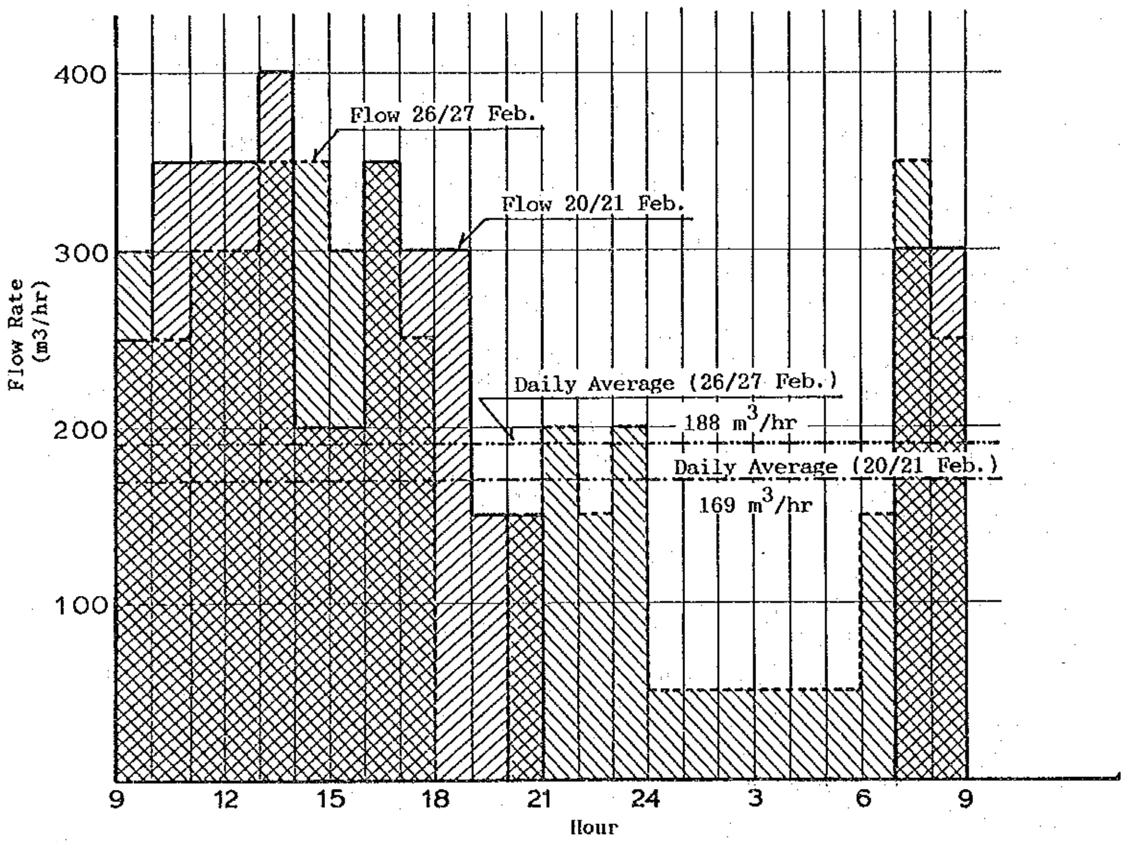

Fig. D.5 Sewage Flow Fluctuation at Hedjuff P/S

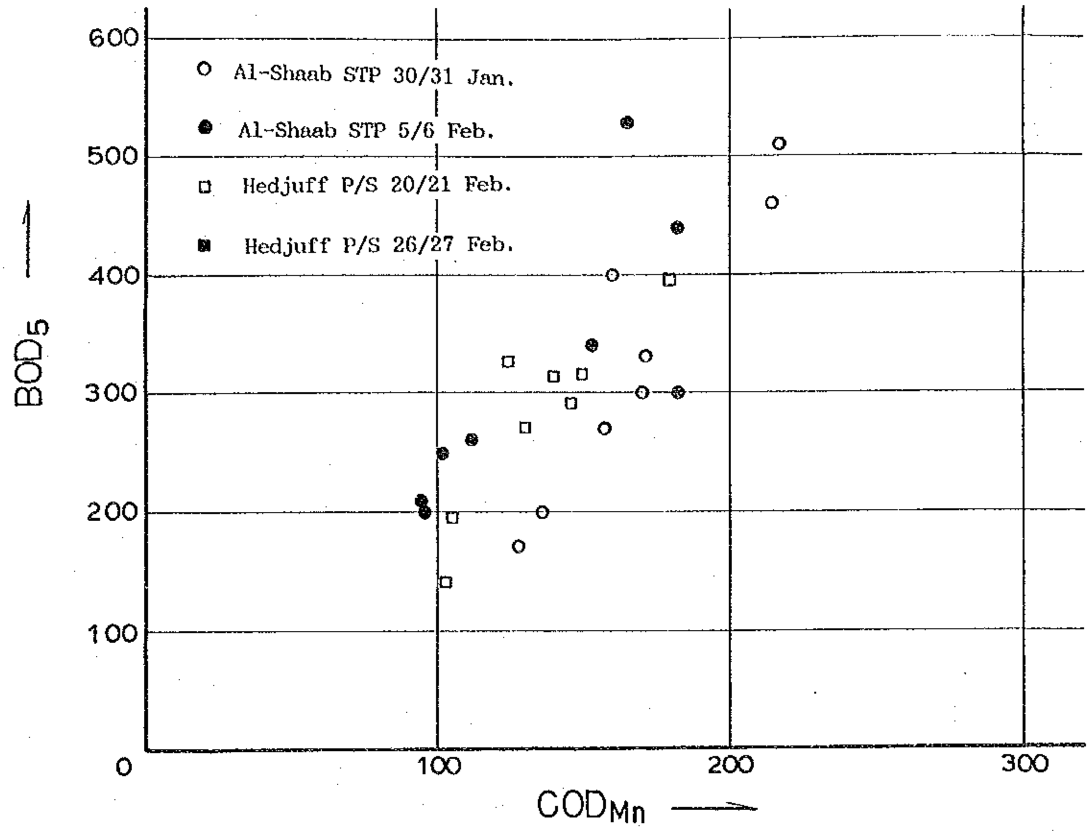

Fig. D.8 Correlation between BOD5 and CODmn
(Raw Sewage)

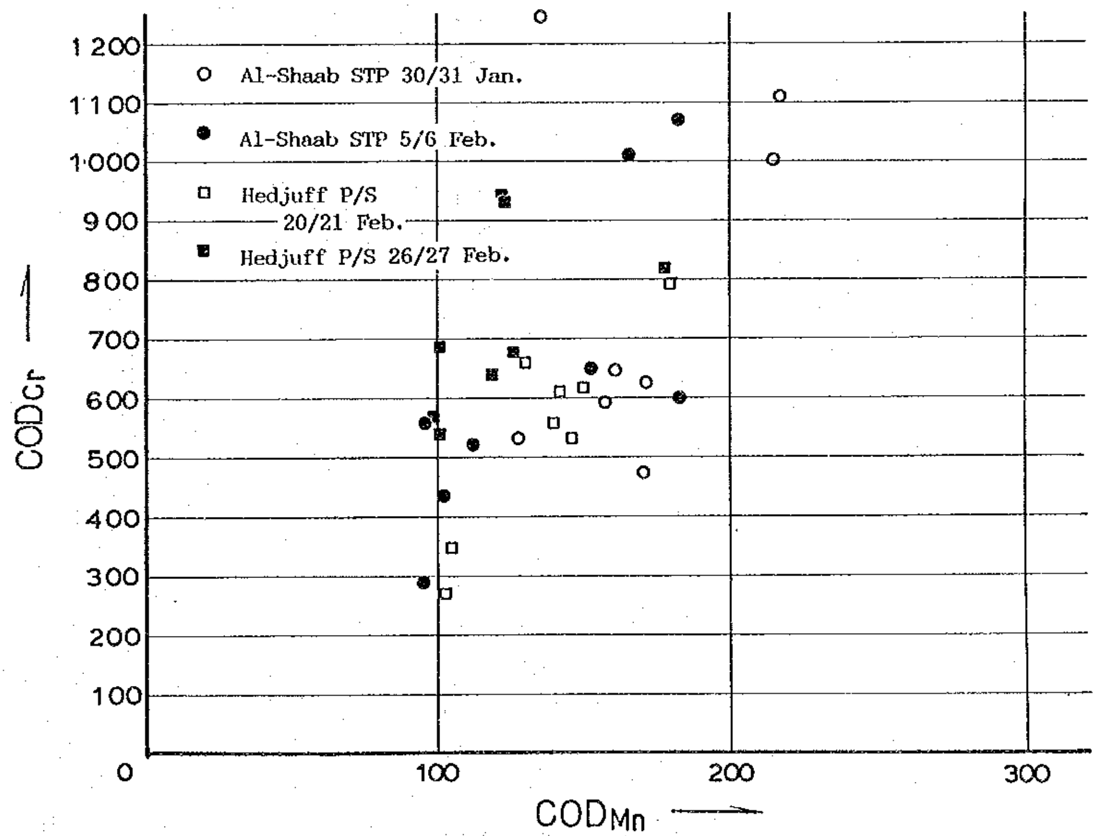

Fig. D.9 Correlation between CODcr and CODmn
(Raw Sewage)

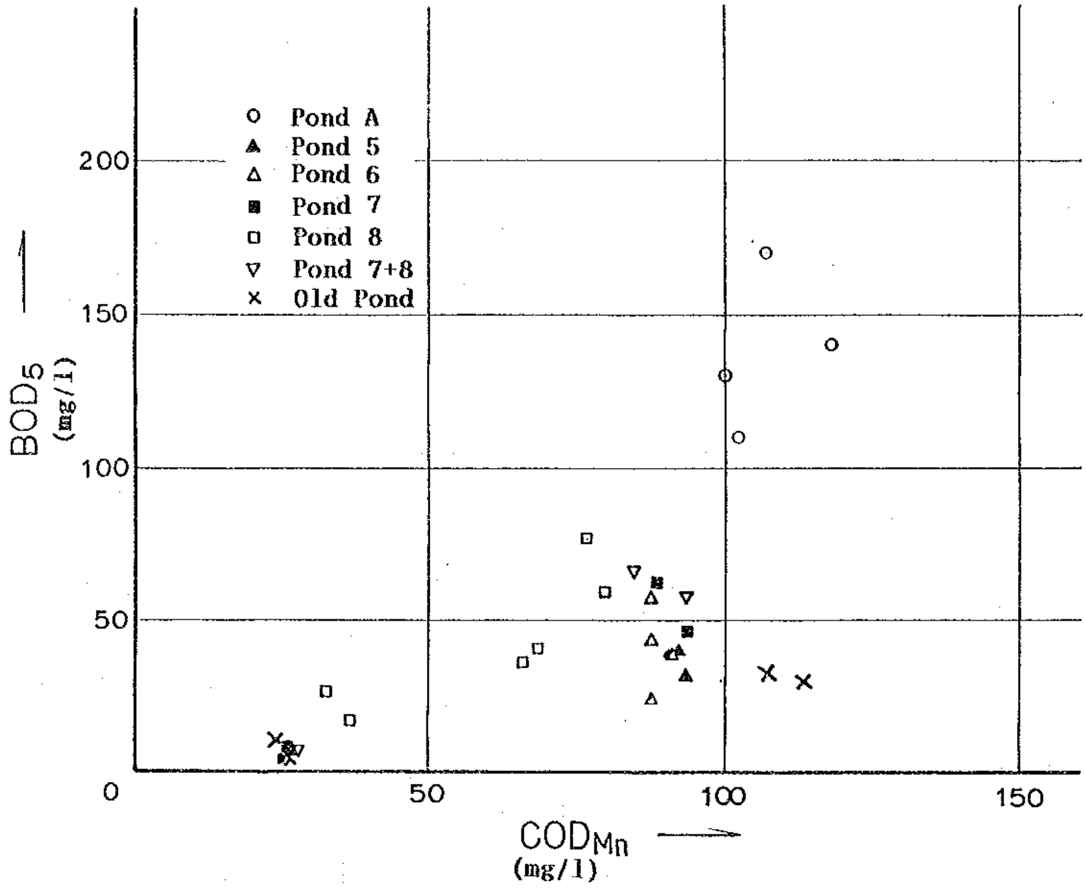

Fig. D.10 Correlation between BOD5 and CODmn
(Stabilization Pond Water)

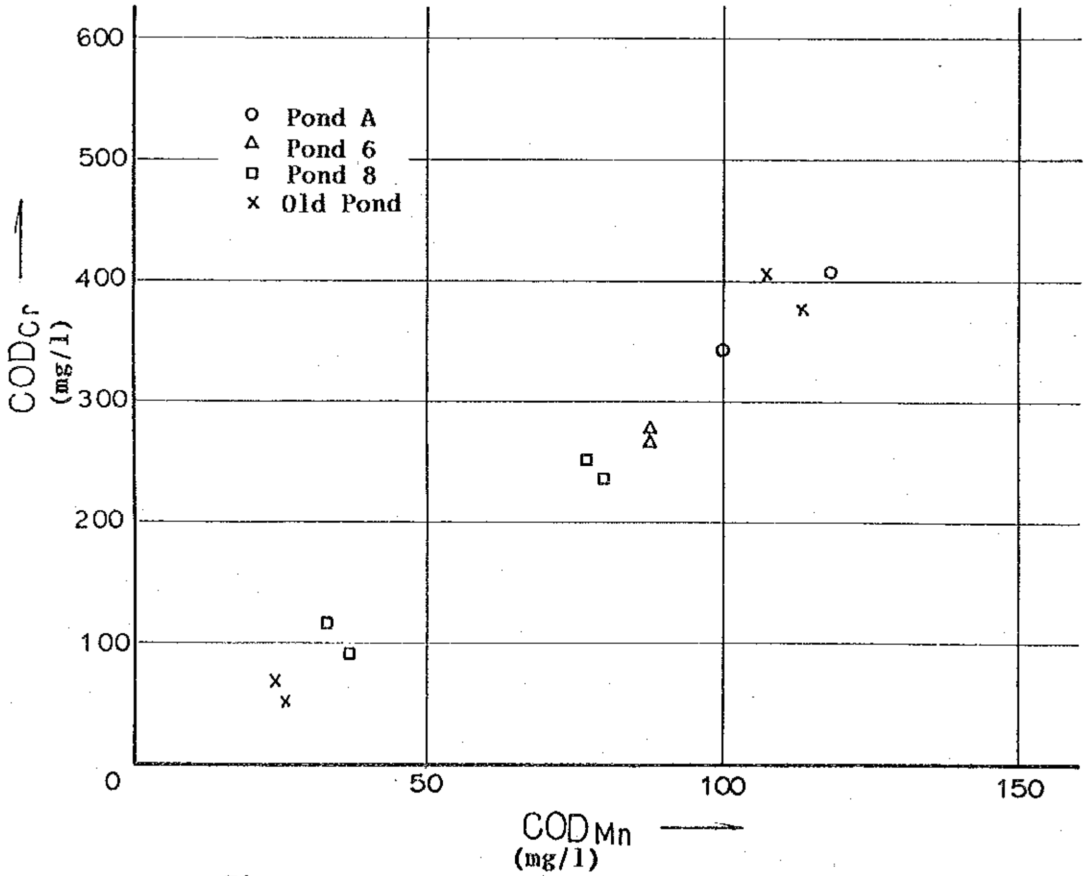

Fig. D.11 Correlation between CODcr and CODmn
(Stabilization Pond Water)

APPENDIX E

WATER POLLUTION ANALYSIS IN TAWAHI BAY BY MATHEMATICAL MODELS

APPENDIX E

WATER POLLUTION ANALYSIS IN TAWAHI BAY BY MATHEMATICAL MODELS

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APPENDIX E

WATER POLLUTION ANALYSIS IN TAWAHI BAY BY MATHEMATICAL MODELS

1. Introduction

As a part of the water quality analysis of the study, water pollution analysis in Tawahi Bay by using mathematical models has been intended. The purpose of the analysis is to obtain an useful measure to assess the pollution characteristics in the Tawahi Bay in evaluating the various alternative sewerage schemes. However, since it is the first attempt to use these kinds of the mathematical models for the analysis of the bay, parameters and coefficients used in the models are not necessarily readily available at present. Many hypotheses are used for calculation based on the experience of the consultant. This situation obliges the study team to use the results of the analysis for the limited purposes. Nevertheless, this approach will be of great use not only for assessment of sewerage schemes but also for evaluation of the various development schemes related to conservation of marine environment.

Mathematical models are composed of the two models, viz. hydraulic model and pollution model. A hydraulic model is to analyze tidal currents in the area of interest, the Tawahi Bay for the study. Physical characteristics of the Bay, such as spatial dimensions and tide levels are the most important elements to be considered in formulation of the hydraulic model. The model analyzes the movement of water in the Bay. On the other hand, the pollution model analyzes the movement of the pollutant substances in the water body. Movement of the pollutant substances is determined firstly by tidal currents in the water body. Diffusion caused by tidal currents and assimilation by natural process of the certain substances, such as organic materials indicated in COD, are to be considered for the model. Equations and calculation methods of the mathematical models are described in the following sections. Computer programs of these models have been developed by the consultant and proved to be very useful for evaluation of pollution characteristics of various water bodies in Japan.

2. Mathematical Models

2.1 Hydraulic Model

Equations describing the movement of the water are in general presented by two forms of differential equations, continuity and motion. These are written as follows. Figure E.1 depicts the axis of coordinates for equations.

Equation of continuity :
$$\frac{\partial u}{\partial x} + \frac{\partial v}{\partial y} + \frac{\partial w}{\partial z} = 0 \quad \cdots\cdots (1)$$

Equation of motion :
$$\frac{\partial u}{\partial t} + u\frac{\partial u}{\partial x} + v\frac{\partial u}{\partial y} + w\frac{\partial u}{\partial z}$$
$$= f \cdot v - g\frac{\partial \xi}{\partial x} + \nu\frac{\partial^2 u}{\partial z^2}$$
$$+ L\left(\frac{\partial^2 u}{\partial x^2} + \frac{\partial^2 u}{\partial y^2}\right) \quad \cdots\cdots (2)$$

$$\frac{\partial u}{\partial t} + u\frac{\partial u}{\partial x} + v\frac{\partial u}{\partial y} + w\frac{\partial u}{\partial z}$$
$$= -f \cdot u - g\frac{\partial \xi}{\partial y} + \nu\frac{\partial^2 v}{\partial z^2}$$
$$+ L\left(\frac{\partial^2 v}{\partial x^2} + \frac{\partial^2 v}{\partial y^2}\right) \quad \cdots\cdots (3)$$

where;

- u, v, w : flow velocities in x, y, z directions
- ξ : depth of water from mean sea level
- ν, L : vertical and horizontal eddy viscosity coefficient
- f : coefficient of Coriolis force
- g : acceleration of gravity

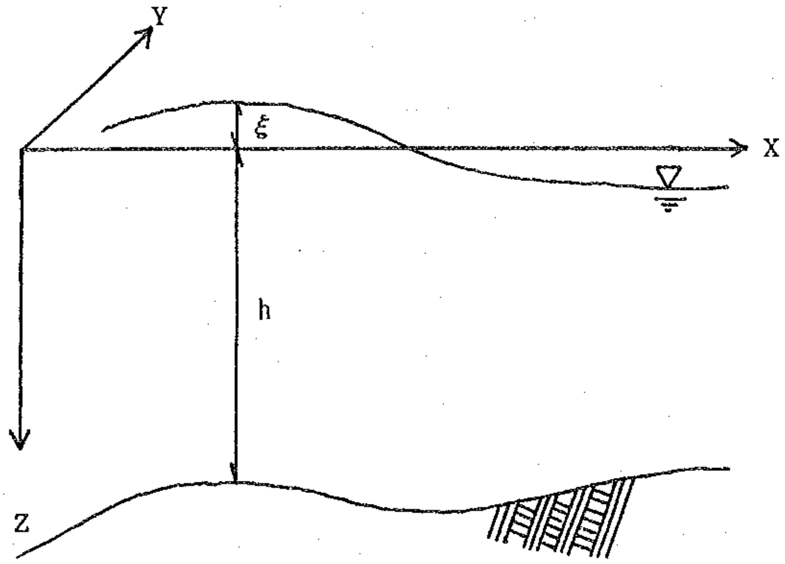

Figure E.1 The Axis of Coordinates

Then, the following equations are obtained by integration of the above equations from the bottom to the surface of the water.

$$\frac{\partial \xi}{\partial t} + \frac{\partial M}{\partial x} + \frac{\partial N}{\partial y} = 0 \qquad \cdots\cdots(4)$$

$$\frac{\partial M}{\partial t} = f \cdot N - g(h + \xi)\frac{\partial \xi}{\partial x} - r_b \cdot U \cdot \sqrt{U^2 + V^2}$$
$$- (h + \xi)\left(\frac{\partial U^2}{\partial x} + \frac{\partial UV}{\partial y}\right) + L\left(\frac{\partial^2 M}{\partial x^2} + \frac{\partial^2 N}{\partial y^2}\right) \cdots (5)$$

$$\frac{\partial N}{\partial t} = -f \cdot M - g(h + \xi)\frac{\partial \xi}{\partial y} - r_b \cdot U \cdot \sqrt{U^2 + V^2}$$
$$- (h + \xi)\left(\frac{\partial UV}{\partial x} + \frac{\partial V^2}{\partial y}\right) + L\left(\frac{\partial^2 N}{\partial x^2} + \frac{\partial^2 M}{\partial y^2}\right) \cdots (6)$$

Where;

U, V : mean flow velocities in x and y directions

M, N : flow rate in x and y directions

h : water depth from the bottom to the mean sea level

r_b : coefficient of friction of the sea bottom

$$M = \int_{-\xi}^{h} u dz = U \cdot (h + \xi)$$

$$N = \int_{-\xi}^{h} v dz = V \cdot (h + \xi)$$

Equations (4),(5) and (6) are modified into difference equations for numerical calculation by computer.

2.2 Pollution Model

Analysis of pollution in such a water body as bay where water depth is far small in dimension compared to aerial dimension and a main interest is aerial distribution of pollutants, the following two-dimension differential equations are generally used.

$$\frac{\partial (HC)}{\partial t} = - \frac{\partial (HUC)}{\partial x} - \frac{\partial (HVC)}{\partial y} + \frac{\partial}{\partial x} (HD_x \frac{\partial C}{\partial x}) + \frac{\partial}{\partial y} (HD_y \frac{\partial C}{\partial y}) + F \qquad \cdots\cdots (7)$$

where;

$H = (h + \xi)$: depth of water from sea bottom

Dx, Dy : diffusion coefficient in x and y directions

F : decreasing or increasing parameter

This equation defines behavior of pollutant substances in unsteady conditions. Equation (7) is modified into difference equations for numerical calculation by computer.

3 Tidal Current Analysis

3.1 Grid Model

For the numerical calculation of the tidal current, Tawahi Bay is approximated by a grid model with relevant distances taking into account the physical characteristics of the Bay. Most of the perimeters of the Bay except for Aden Harbor and Little Aden are surrounded by shallow sandy coast. An average difference of water level in high and low tides is approximately 0.8 m. Considering these factors, boundaries of water body are considered to be 0.5 m bathymetric line for the convenience of calculation.

Distance of the grid is set at 500 m taking into account time intervals of step calculation. Depth of water at each node of grid is read from a bathymetric map. Boundary of the Tawahi Bay to the Gulf of Aden, which also forms boundary of calculation is set at the line from western tip of Aden Peninsula to the eastern tip of Little Aden. The study team was advised by the officer of YPA to enlarge calculation area to include the coastal areas around Elephant Trunk in Tawahi and neighboring areas of Little Aden Refinery. Grid system covering the Tawahi Bay thus expanded and depths of water are illustrated on Figure E.3.

3.2 Tide Level

Tide levels should be given at boundary. A representative tidal wave such as M2 tidal constituent with an average amplitude and cycle is usually considered for water pollution analysis, since the analysis is mostly based on long time basis, i.e. more than several days. M2 tidal constituent is, however, not available. An attempt was made to figure out representative tidal wave in the Tawahi Bay. High and low tides for seven days from 1st to 7th December, 1988 at Aden Habor given by tide table are shown in Table E.1 and Figure E.2. As shown in the figure, amplitude and cycle does not seem to be uniform. It is very difficult to give such waves of long time period to the model, and it is not necessary to make the calculation so complicated. Therefore, average amplitude and cycle for the month of December, 1988 are calculated to be used for calculation. It was confirmed with the officer of YPA that this wave is representative at Aden Harbor. These are as follows.

Average amplitude	0.41 m	December, 1988
Average cycle	12.35 hrs.	

3.3 Other Factors

Other factors to be considered include the following.

(1) Friction Coefficient of Sea Bottom

This coefficient varies depending on the shape and materials which form the bottom. A uniform value for sandy bottom obtained in Japan of 0.0026 was used for all the meshes.

(2) Coriolis Force

This parameter is calculated by the following formula

$$f = 2 \times \omega \times \sin\phi$$

where;

ω = 7,292,115 10^{-11} (rad/s)

ϕ = latitude (12° 47" N)

(3) Horizontal Eddy Viscosity Coefficient

This coefficient represents decline of motion caused by differences of velocities in viscosity fluid. It is assumed to be 1.47 x 10^5 cm2/sec.

(4) Time Interval for Calculation

There is a certain limitation regarding time interval and distance of grid nodes. Time interval should satisfy the following formula.

$$\Delta t \leqq \Delta s / \sqrt{2gh_{max}}$$

where,

ΔS : distance of grid node

g : acceleration of gravity (9.8 m/sec2)

H_{max} : maximum water depth

With 500 m distance, time interval is calculated to be $\leqq$ 29.0 sec. Hence, time interval of 20.0 sec. is used.

3.4 Results of the Calculation

Results of the calculation are presented on Figures E.4, E.5 and E.6. Figure E.4 shows the mean flow velocity of each mesh for one tidal cycle (12.35 hrs.). Mean flow velocity of each mesh is calculated by integratation of velocity vectors at every steps of calculation with a certain time interval. Therefore, mean velocity represents a constant flow direction of a certain mesh for one cycle of tide (from highest level of water to the next highest). It is observed from the Figure E.4 that mean velocities are in anticlockwise in wide area of Tawahi Bay, especially, high velocities are observed west side of the embankment, also clockwise direction is shown near Little Aden.

Figure E.5 shows tidal currents at ebb tide when decreasing of water level is at maximum rate (6 hrs. after starting of calculation viz. half cycle).

Figure E.6 shows tidal current at flood tide when increasing of water level is at maximum rate (12 hrs. after starting of calculation viz. one cycle).

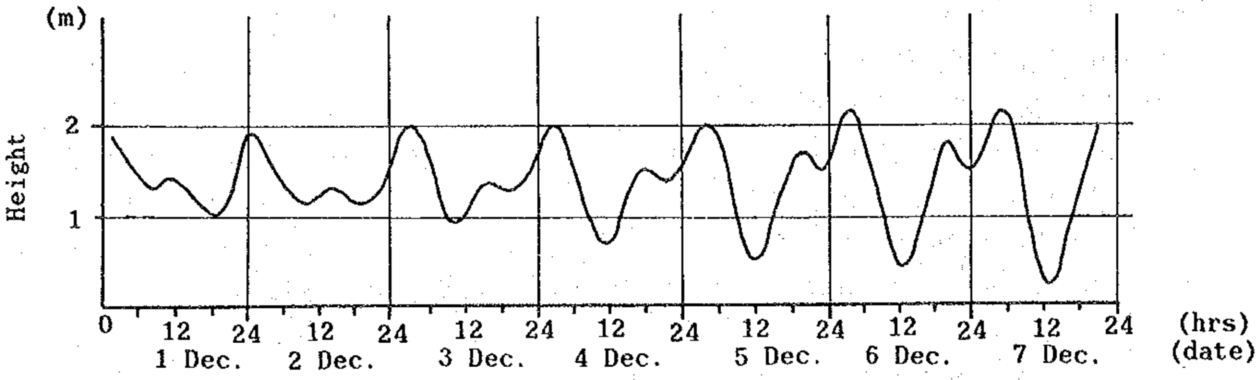

Figure E.2 Tide Levels at Aden Harbor

Table E.1 Tide Table at Aden Harbor

December, 1988

Date	Time hr min	Height m	Date	Time hr min	Height m	Date	Time hr min	Height m	Date	Time hr min	Height m
1	01:52 09:09 11:58 18:35	1.9 1.3 1.4 1.0	9	00:58 06:09 13:45 21:13	1.5 2.2 0.0 2.0	17	01:58 09:24 15:37 19:52	2.2 0.8 1.4 1.2	25	02:56 07:56 15:11 22:24	1.5 2.0 0.2 2.0
2	02:33 10:07 14:58 19:26	1.9 1.1 1.3 1.1	10	01:39 06:50 12:24 21:48	1.5 2.2 0.0 2.0	18	02:45 10:22 17:37 21:13	2.3 0.5 1.6 1.4	26	03:37 08:35 15:41 22:52	1.4 1.9 0.3 2.0
3	03:05 10:43 17:00 20:35	2.0 0.9 1.4 1.3	11	02:22 07:35 15:05 22:26	1.5 2.2 0.0 2.0	19	03:30 11:13 18:54 22:31	2.3 0.3 1.8 1.6	27	04:22 09:15 16:09 23:18	1.4 1.8 0.5 2.0
4	03:33 11:11 18:11 21:45	2.0 0.7 1.5 1.4	12	03:13 08:24 15:46 23:03	1.5 2.1 0.1 2.0	20	04:15 12:00 19:43 23:41	2.3 0.1 1.9 1.6	28	05:11 09:52 16:33 23:45	1.4 1.7 0.6 2.0
5	04:00 11:37 18:56 22:43	2.0 0.5 1.7 1.5	13	04:13 09:16 16:30 23:45	1.5 2.0 0.2 2.1	21	05:00 12:41 20:20	2.2 0.0 2.0	29	06:11 10:33 16:54	1.3 1.6 0.8
6	04:30 12:05 19:33 23:31	2.1 0.4 1.8 1.5	14	05:22 10:15 17:13	1.4 1.8 0.4	22	00:41 05:45 13:22 20:54	1.6 2.2 0.0 2.0	30	00:11 07:20 11:28 17:09	2.0 1.2 1.4 1.0
7	05:00 12:35 20:05	2.1 0.2 1.9	15	00:28 06:43 11:26 17:58	2.1 1.3 1.6 0.7	23	01:30 06:30 14:00 21:26	1.6 2.2 0.0 2.0	31	00:39 08:31 13:07 17:16	2.1 1.1 1.3 1.2
8	00:16 05:33 13:07 20:39	1.5 2.2 0.1 1.9	16	01:11 08:09 13:11 18:48	2.2 1.0 1.5 0.9	24	02:15 07:15 14:37 21:54	1.5 2.1 0.1 2.0			

Latitude: 12 47' N Longitude: 44 59' E
Datum: 1.34 m below mean sea level
Source: Proudman Oceanographic Laboratory, UK

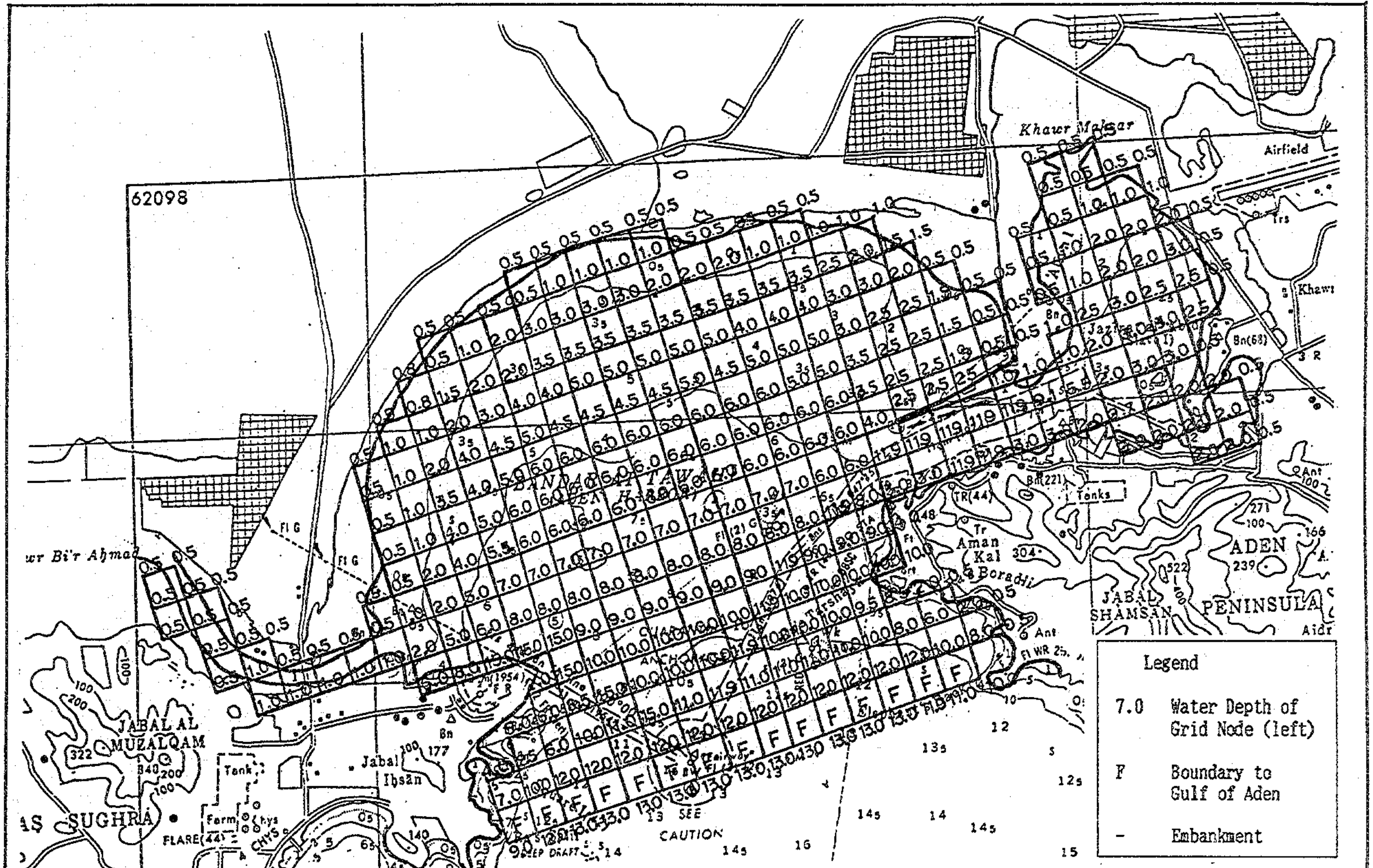

Fig. E.3 Grid Model of Tawahi Bay

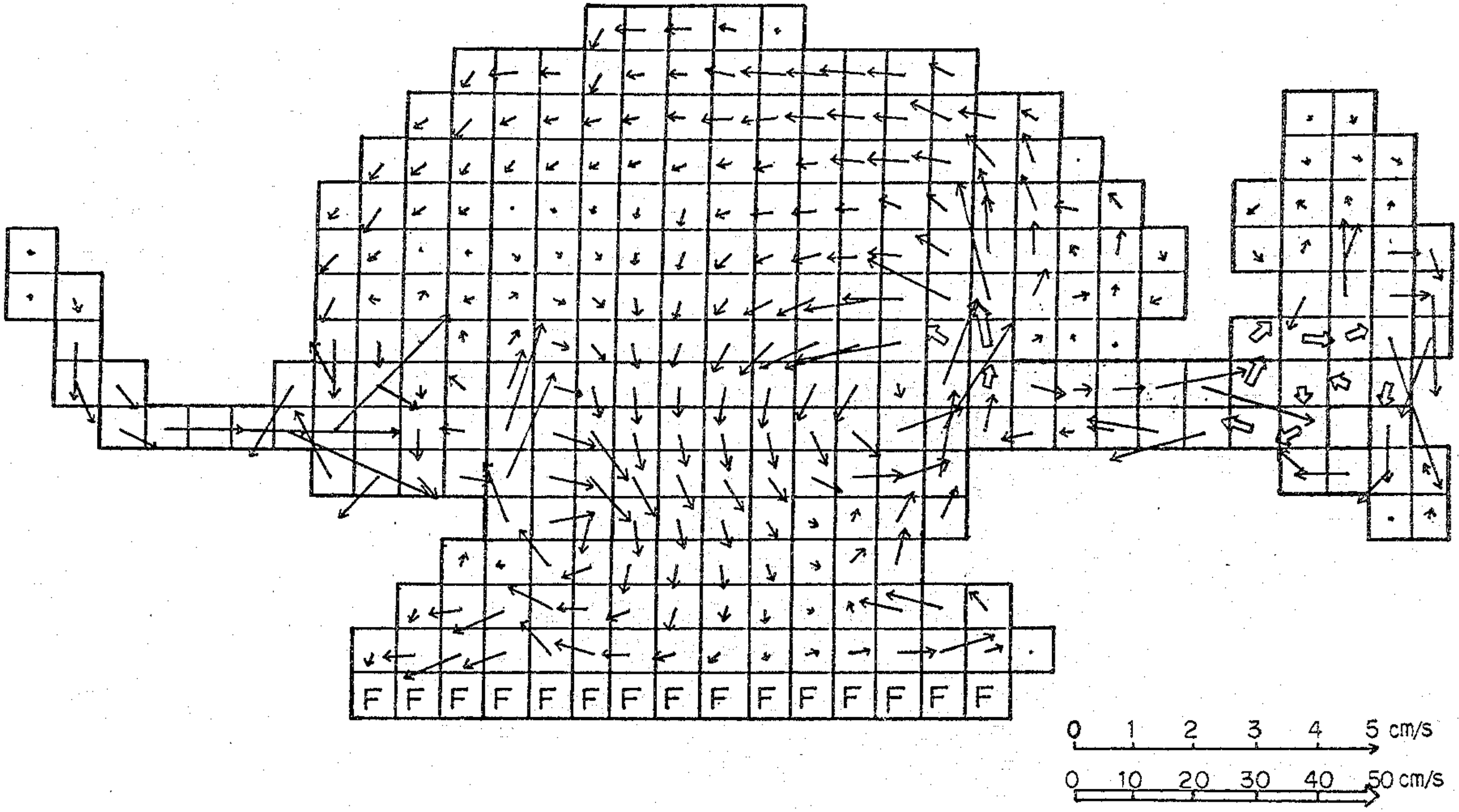

Fig. E.4 Mean Flow Velocity

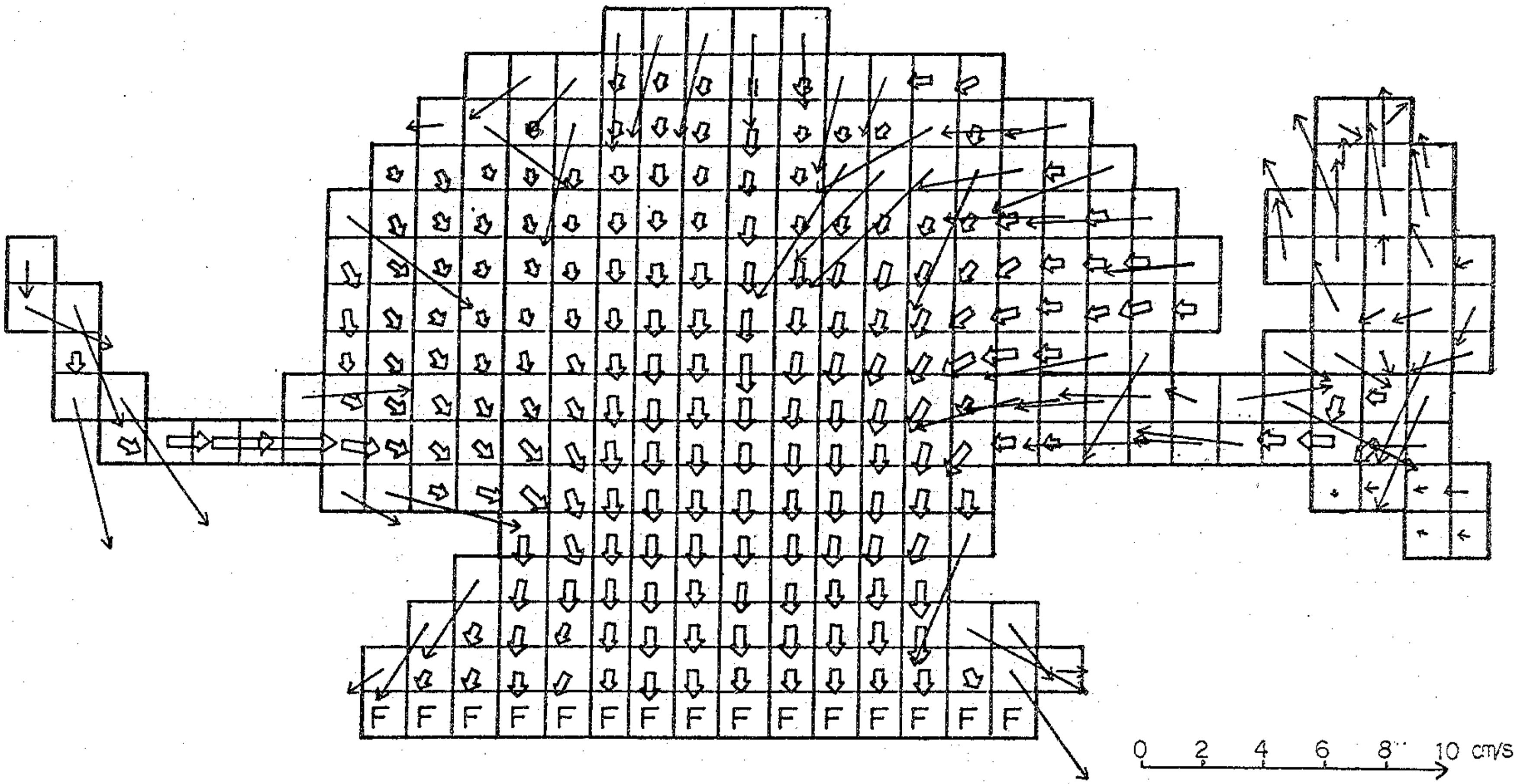

Fig. E.5 Flow Velocity at Ebb Tide

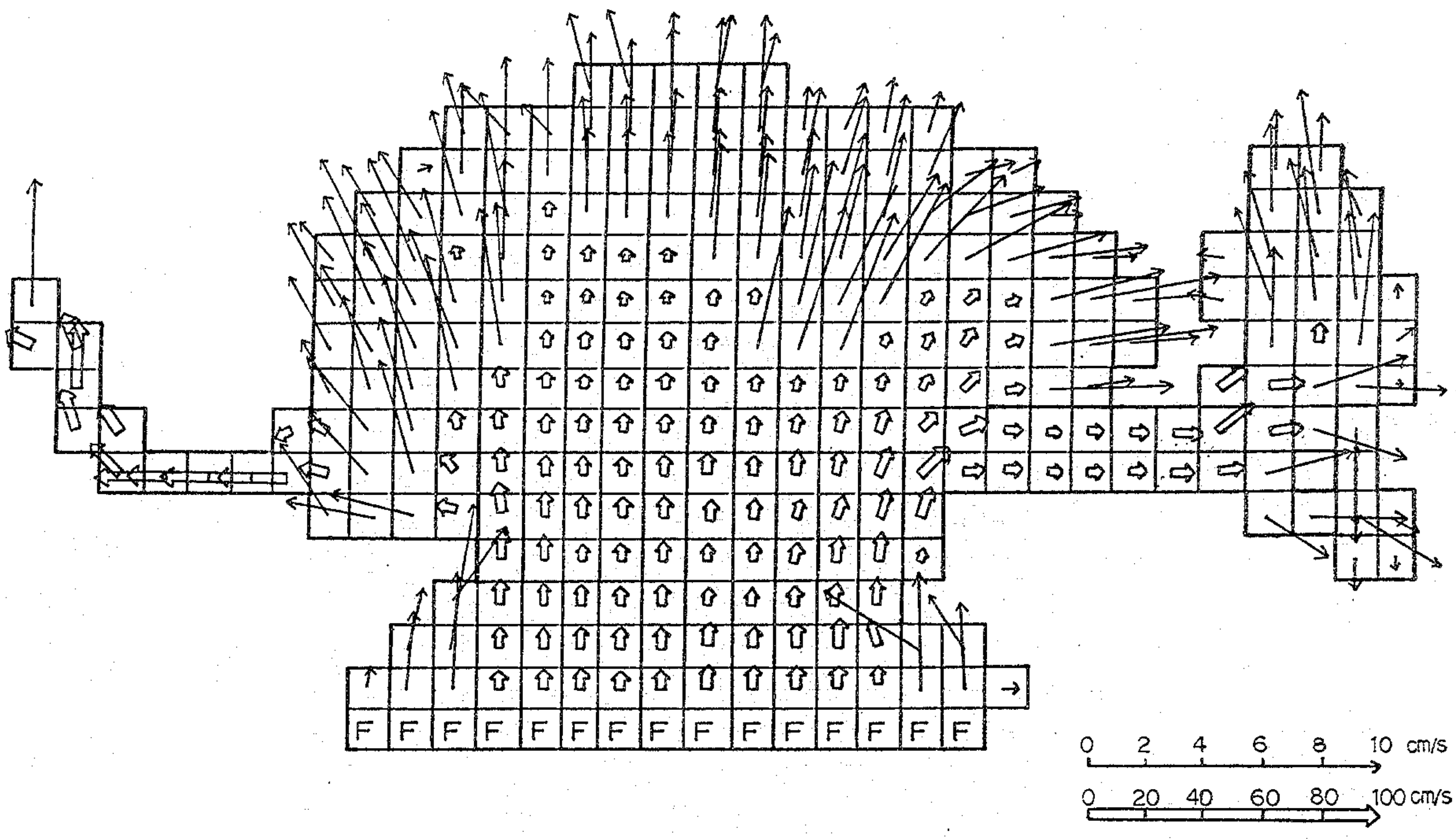

Fig. E.6 Flow Velocity at Flood Tide

4. Pollution Analysis

4.1 Prerequisites for Analysis

(1) Grid Model

Same grid model is used here as used for tidal current analysis which is described in section 3.1.

(2) Tide

Tide for the pollution calculation is taken from the result of tidal simulation. Mean flow velocity is adopted here to describe pollution model.

(3) Pollution Index

COD_{mn} is adopted here according to the experience in Japan. BOD is not a good index for the sea water. COD_{mn} is common in Japan than COD_{cr}.

(4) Boundary Condition

Boundaries of calculation area are set at the same lines as for the tidal simulation. COD_{mn} concentration of 1.0 mg/l is given at boundaries taking into account the results of water quality analysis in Tawahi Bay.

4.2 Parameters for Mathematical Model

The equation of pollution model is described in section 2.3. Equation (7) (non-ergodic equation), which is changed into difference form for the numerical calculation with the method developed by Public Works Research Institute in Japan, was used.

The explanation of each parameter is as follows.

(1) Diffusion Coefficient

Elder type coefficient is used for calculation, parameters are set as follows.

$$D_x = \alpha(a|U| + b|V|)H$$
$$D_y = \alpha(a|V| + b|U|)H$$

where,

D_x, D_y : diffusion coefficient in x and y directions

U,V : mean flow velocity in x and y directions

H =(h+): depth of water from sea bottom

a = 0.623

b = 0.03

= 100.

4.3 Simulation of Present Situation

(1) Discharge Points

Three discharge points (see Figure E.7) are selected, viz. Ma'alla, Tawahi and Al-Shaab taking into account the present sources of organic pollutants. They are the major discharging points of sewerage system in Aden.

COD_{mn} loads discharged were calculated from the present daily average flow of sewage and their average concentrations obtained from water quality analysis.

Table E.2 Flow at Discharge Points

Point	COD_{mn} (mg/l)	Flow (m^3/day)	Loading (kg/day)
Al-shaab	85	10,376	882
Ma'alla	138	6,313	871
Tawahi*1	224	4,480	1,003
Total		21,169	2,756

Note:

*1) There are four discharge points in Tawahi district. One representative discharge point is selected for the convenience of calculation. This flow is the sum of those four points.

To trace the effect of tidal current to COD concentration distribution in the bay with sewage discharging flow from three points, initial COD concentration are set at 1.0 mg/l at all meshes. This may cause the discrepancy between simulated COD concentrations and actual concentrations of sampling points, which are presented in Figure E.7.

(2) COD Decreasing Coefficient

In general, this coefficient is determined with consideration of field data. Due to the lacking of such data, the following relation is assumed according to the experiments in Japan. COD decreasing coefficient is proportion to the second power of COD concentration.

$K = k \cdot C^2$

where;

K : COD decreasing coefficient (1/day)

k : coefficient $(1/day/(mg/l)^2)$

C : present COD concentration (mg/l)

The grid model covers about a area of 7.125×10^7 m^2 and average depth is about 3 m. Then volume of water is 2.1375×10^8 m^3. As is evident from Table E.2, total loading is 2,756 kg/day. Based on the volume of water and COD loading, the COD concentration increases per day by added load is 0.012. If it is assumed that this increase is canceled by self purification of the water body, coefficient k is calculated to be 0.01. The results of the calculation with this coefficient in Figure E.8. Simulated COD concentrations at sampling points S1, S2, and S3 are lower than measured values. To overcome this discrepancy, many calculations were carried out by changing various coefficients. However, better results could not be obtained, and combination of coefficients became so complicated. It was considered to be of no use to continue calculations with uncertain coefficients. Therefore, for the simulation of future pollution in Tawahi Bay, above mentioned model is used.

4.4 Simulation of Future Situation

From the results described above, the future pollution condition in Tawahi Bay in the year 2010 was calculated. As all sewage from Ma'alla and Tawahi districts is proposed to be treated at proposed treatment plant at that time, the following two cases are considered for simulation.

Case 1 - Sewage from Ma'alla and Tawahi districts is discharging into Tawahi Bay as it is now. Flows and COD_{mn} loading are estimated as shown in Table E.3. This case represents the condition without sewerage project.

Table E.3 Discharge Flow in the Year 2010

Point	COD_{mn} (mg/l)	Flow 2010 (m^3/day)	Loading (kg/day)
Al-shaab	64	20,752	1,328
Ma'alla	110	13,464	1,481
Tawahi	178	6,912	1,230

Case 2 - Sewage from Ma'alla and Tawahi districts is treated at the treatment plant, and is not discharged to Tawahi Bay. In this case, only Al-shaab discharge is considered. This case represents the condition after implementation of sewerage project.

Results are shown in Figure E.9 and E.10. Obviously, it is far better not to discharge Ma'alla and Tawahi sewage to Tawahi Bay. Although it is not observed that the adjacent meshes of the discharging points do not increase the COD concentration because of fairly large scale of the distance of grid (500m), COD concentrations at the discharging points increase by 1.3 times at Ma'alla and Al-shaab discharge points.

5. Considerations

Since 1970s, marine pollution caused by human activities has widely become aware of everywhere all over the world. Since then, various attempts were made to analyze pollution problems. Analysis of physical phenomena by mathematical models has rapidly progressed thanks to advance of computer and techniques.

Analytical method presented here is an outcome of the progress. However, it should be reminded that many factors which affect actual tidal current are not necessarily taken into account at present. Although dominant driving force of currents is tide, other factors, such as densimetric flow and wind can not be neglected.

It should also be noted that there are much uncertain factors in the pollution model. COD concentration at the specific point is the compound phenomena caused by incoming loading from the land, diffusion by currents, sedimentation and self purification in the water body and so on. Data and information are decisively lacking at present to evaluate these phenomena. Thus, the pollution model presented here is to be elaborated further to evaluate various alternative sewerage systems.

It can be said from the results of the water quality analysis that pollution in Tawahi Bay caused by discharging of raw sewage is limited in localized area at present. This is due to high reduction rate of organic pollutant. Active biochemical reaction due to high water temperature throughout a year contribute to high reduction rate. This tendency was simulated by the models to a certain extent.

Due to the advanced microcomputer techniques, many models can be used easily for analysis of pollution. However, it will take significant time and efforts to accumulate the data and information. The importance of continual monitoring of Tawahi Bay concerned with water pollution should be emphasized in view of solving mechanism and preventing pollution.

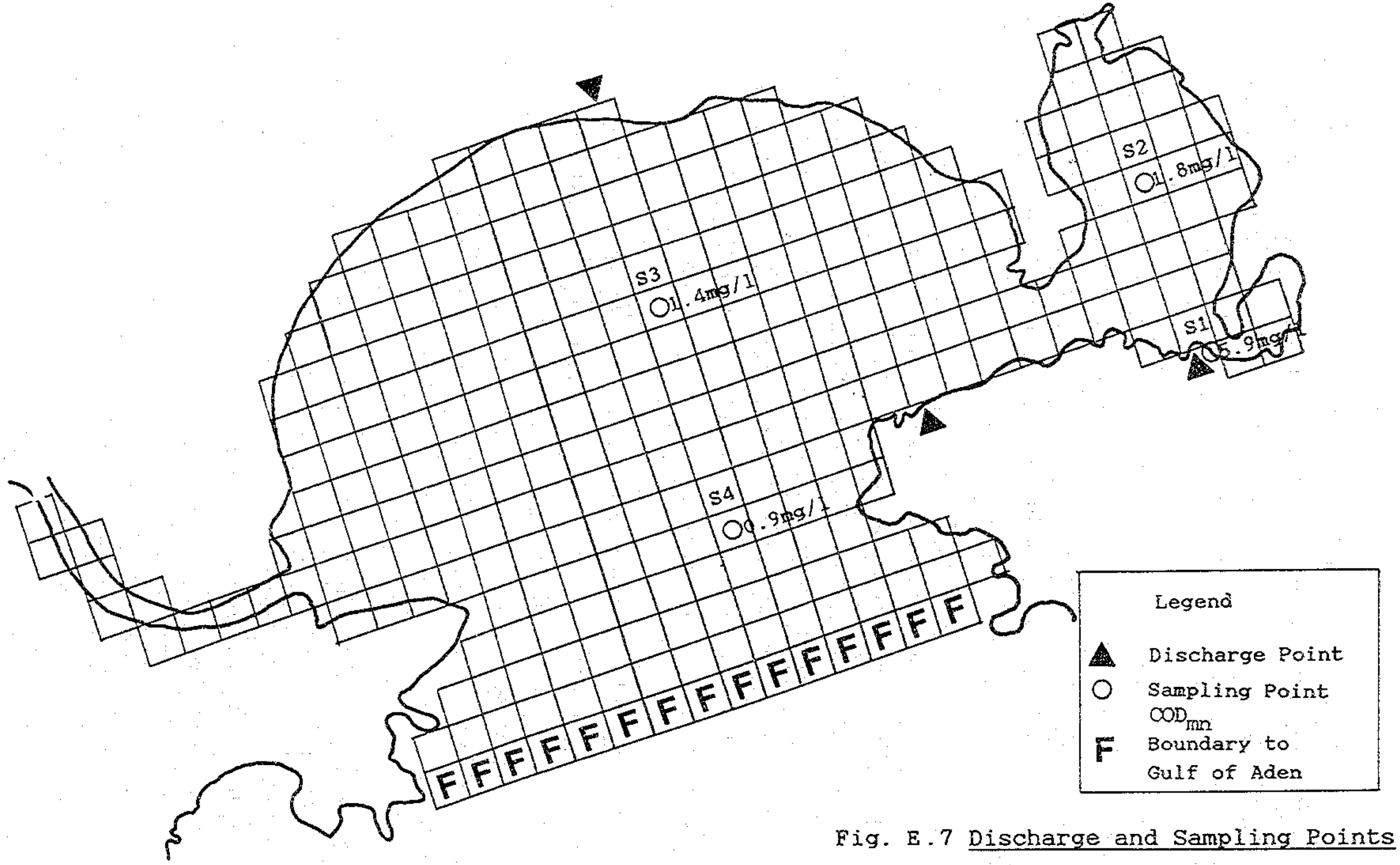

Fig. E.7 Discharge and Sampling Points

** Discharge Points **

Q(19, 2)= 882.0 KG/DAY
Q(32,12)= 871.0 KG/DAY
Q(25,11)= 1003.0 KG/DAY

** Coefficients **

Time Interval 600.00SEC
Grid Distance 500.0 M
Decreasing Coefficient .010 1/DAY

Water Quality CODmn (MG/L)

	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34
2															1.0	1.0	1.0	1.0	2.0															
3												1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0											
4											1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0						1.0	1.0		
5										1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0					1.0	1.0	1.0	
6									1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0			1.0	1.0	1.0	1.0	
7		1.0							1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0		1.0	1.0	1.0	1.0	1.0
8		1.0	1.0						1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0			1.0	1.0	1.0	1.0
9			1.0						1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0			1.0	1.0	1.0	1.0	1.0
10			1.0	1.0				.9	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	.9	1.0	1.0	1.0	
11				1.0	1.0	1.0	1.0	1.1	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.2	1.0	1.0	1.0	1.0	1.0	1.1		1.0	
12									1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0								1.0	1.8	1.0	1.0
13													1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0										1.0	1.0
14												1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0												
15											1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0										
16										1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0									
17										1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0										

⇧ Discharging points

Fig. E.8 Simulation of COD Distribution in 1989

** Discharge Points **

Q(19, 2)= 1328.1 KG/DAY
Q(32,12)= 1481.1 KG/DAY
Q(25,11)= 1230.3 KG/DAY

** Coefficients **

Time Interval 600.00SEC
Grid Distance 500.0 M
Decreasing Coefficient .010 1/DAY

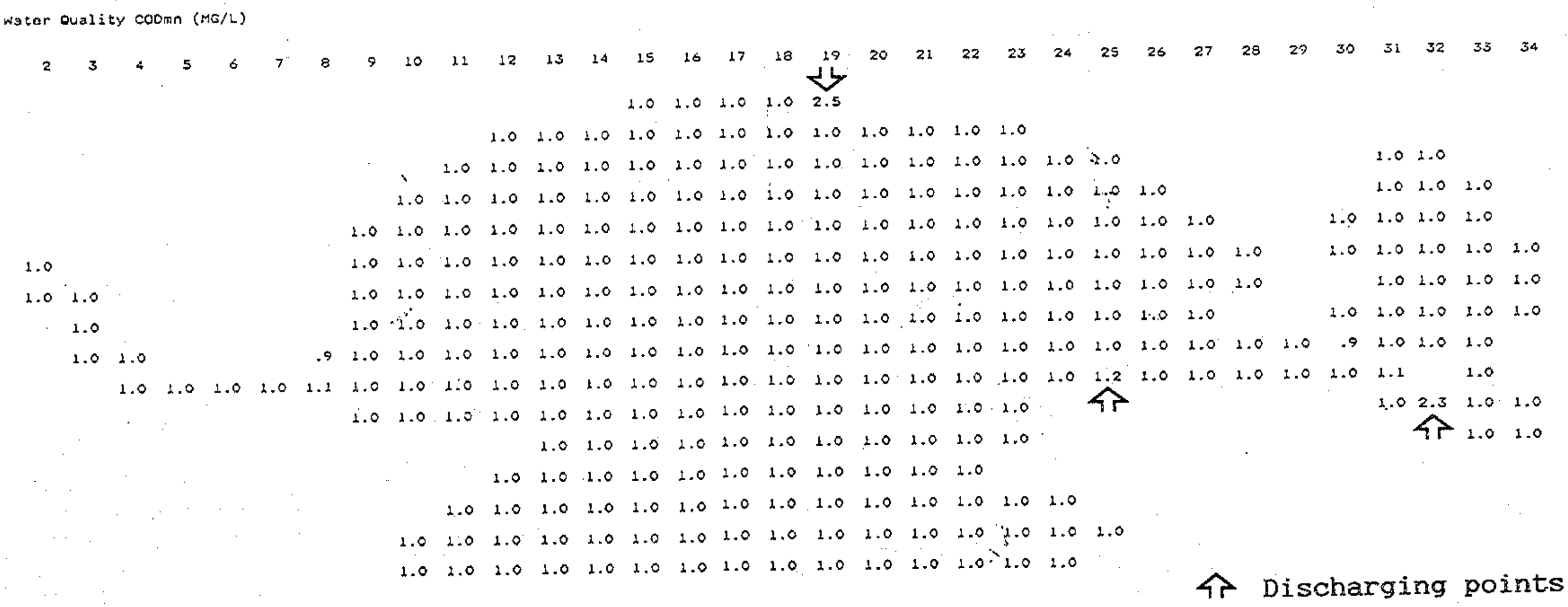

Fig. E.9 COD Distribution in 2010, Case-1

** Discharge Points **

Q(19, 2)= 1328.1 KG/DAY

** Coefficients **

Time Interval 600.00SEC
Grid Distance 500.0 M
Decreasing Coefficient .010 1/DAY

Water Quality CODmn (MG/L)

	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34
2															1.0	1.0	1.0	1.0	2.5															
3												1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0											
4											1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0						1.0	1.0		
5										1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0					1.0	1.0	1.0	
6									1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0			1.0	1.0	1.0	1.0	
7		1.0							1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0		1.0	1.0	1.0	1.0	1.0
8		1.0	1.0						1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0			1.0	1.0	1.0	1.0
9			1.0						1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0			1.0	1.0	1.0	1.0	1.0
10			1.0	1.0				.9	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	.9	1.0	1.0	1.0	
11				1.0	1.0	1.0	1.0	1.1	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.1		1.0	
12									1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0								1.0	1.0	1.0	1.0
13													1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0										1.0	1.0
14												1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0												
15											1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0										
16										1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0									
17										1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0										

⇧ Discharging point

Fig. E.10 COD Distribution in 2010, Case-2

APPENDIX F

HYDRAULIC CALCULATION FOR EXISTING AND PROPOSED SEWERS

APPENDIX F

HYDRAULIC CALCULATION FOR EXISTING AND PROPOSED SEWERS

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APPENDIX F

HYDRAULIC CALCULATION FOR EXISTING AND PROPOSED SEWERS

1. General

Presently, four districts, viz. Tawahi, Ma'alla, Khormaksar and Crater, in Aden have sewerage systems, but there is no sewage treatment plant. In this Appendix, hydraulic calculations for the existing sewer system in the four districts have been carried out in order to check the sewer capacity for the flow rates in the target year of 2010. Further, hydraulic calculation of proposed trunk sewers are presented in this Appendix. It is recommended that the existing sewers of which capacities are proved to be adequate to gravitate flows in 2010 are to be used for the project. If sewer capacity is not enough, replacement or diversion of certain sections are considered.

Manning formula is used for hydraulic calculations with a roughness coefficient (n) of 0.013. Data necessary for the calculation, such as diameter and gradient of a pipe, are obtained from the results of the field survey and/or available drawings. Flow rate to be used for design of sewer is peek flow which is estimated in Section 3.4 of the Main Report. In order to simulate the most probable conditions in the future, Tawahi, Ma'alla and Crater were divided into three categories of areal sewage flow densities depending on population density and story of buildings. Results of hydraulic calculation of each district are as follows.

2. Tawahi District

2.1 Check of Existing Sewer System

Basic data for calculation is as follows.

	Area(ha)	Flow Rate(m^3/day)
Planning Area	87.0	12,024
Extra Catchment Area (Military Area)	119.0	1,800
Total	206.0	13,824

Tawahi district was divided into three categories and flow rates per area of each category was calculated as follows.

Category	Area(ha)	Flow(m^3/day)	Flow(m^3/s/ha)
a	10.7	1,938	0.00210
b	49.6	7,185	0.00168
c	26.7	2,901	0.00126
Total	87.0	12,024	

Presently, Tawahi district is divided into five drainage areas, named A, B, C, D and E from east to west, identified by outlets to Tawahi Bay. Western area, area E, has no sewer system. Sewage from extra catchment area (military area) which is located in south part of Tawahi flows into area A. Its flow is 1,800 m^3/day. Hydraulic calculation sheet is shown in Table F.1. Downstream sections of the existing sewers are calculated to determine adequacy of capacity. These sections are shown in Figure F.1.

Table F.1 Hydraulic Calculation Sheet of Existing Sewer System in Tawahi

No.of Sewer	Sewerage Area				Existing Sewer			
	Area	Total Area	Sewer Length	Design Flow	Sewer Dia.	Slope	Velocity	Sewer Capacity
	ha	ha	m	m3/s	mm		m/s	m3/s
Flow from Military Area (0.0208m3/s)								
A1	0.5	0.5	54	0.0211	150	46.6	1.86	0.0329
A2	0.6	1.1	37	0.0221	150	28.4	1.45	0.0257
A3	0.4	20.5	28	0.0550	225	15.3	1.40	0.0555
A4	0.5	21.0	35	0.0557	225	15.3	1.40	0.0555
C1	21.5	21.5	18	0.0353	150	4.2	0.56	0.0099
C2	2.9	24.4	24	0.0409	225	10.4	1.15	0.0458
C3	1.7	26.1	44	0.0441	225	10.9	1.18	0.0469
C4	10.1	36.2	21	0.0612	225	13.9	1.33	0.0529
C5	0.2	36.4	42	0.0615	225	5.5	0.84	0.0333
C6		36.4	46	0.0615	225	5.5	0.84	0.0333
C7		36.4	26	0.0615	225	5.5	0.84	0.0333
C8	2.7	39.1	41	0.0656	225	5.4	0.83	0.0330
C9	0.9	40.0	47	0.0667	225	5.4	0.83	0.0330
C10	0.9	40.9	48	0.0679	225	5.4	0.83	0.0330
C11	0.5	41.4	20	0.0687	225	5.4	0.83	0.0330
C12	0.4	41.8	23	0.0695	225	3.4	0.66	0.0262
D6	6.3	6.3	24	0.0104	225	8.3	1.03	0.0409
D7	0.6	6.9	54	0.0114	225	4.8	0.78	0.0311

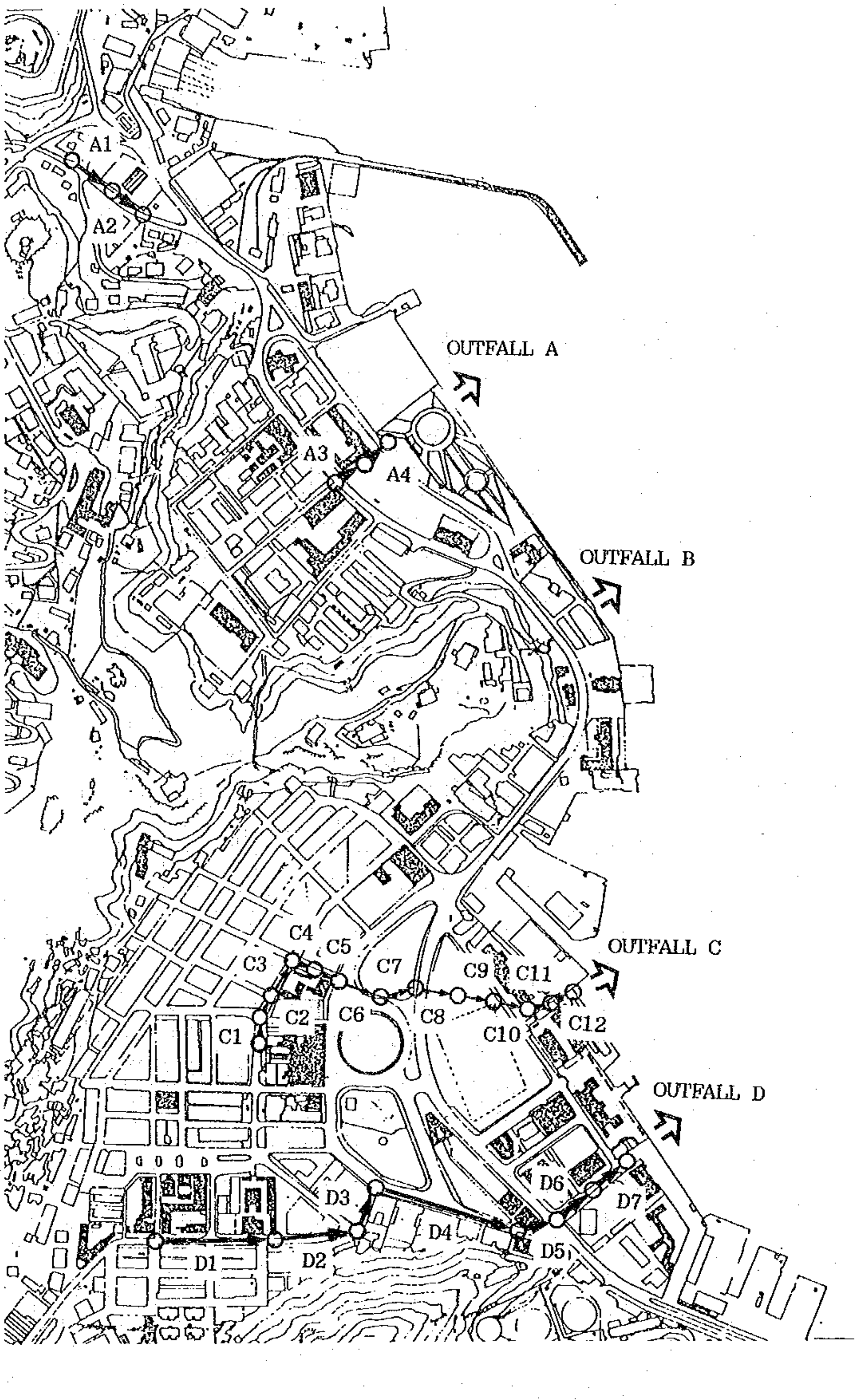

Figure F.1 Checked Sewers in Tawahi District

Capacities of some sewers in area C are found to be smaller than design flows. However, these will be usable if southeast side of this area is diverted to area D. Hydraulic calculation sheet of C and D areas after diversion is shown in Table F.2.

Table F.2 Hydraulic Calculation Sheet of Existing Sewers in C and D areas

No.of Sewer	Sewerage Area		Sewer Length	Design Flow	Existing Sewer			
	Area	Total Area			Sewer Dia.	Slope	Velocity	Sewer Capacity
	ha	ha	m	m3/s	mm		m/s	m3/s
C1	5.1	5.1	18	0.0094	150	4.2	0.56	0.0099
C2	2.9	8.0	24	0.0149	225	10.4	1.15	0.0458
C3	1.7	9.7	44	0.0182	225	10.9	1.18	0.0469
C4	10.1	19.8	21	0.0352	225	13.9	1.33	0.0529
C5	0.2	20.0	42	0.0355	225	5.5	0.84	0.0333
								(0.0358)
C6		20.0	46	0.0355	225	5.5	0.84	0.0333
								(0.0358)
C7		20.0	26	0.0355	225	5.5	0.84	0.0333
								(0.0358)
D1	13.5	13.5	130	0.0211	200	14.1	1.24	0.0389
D2	2.9	16.4	85	0.0259	200	17.4	1.38	0.0433
D3	1.0	17.4	45	0.0275	225	17.9	1.51	0.0601
D4	2.4	19.8	165	0.0315	225	5.3	0.82	0.0327
D5	0.9	20.7	45	0.0334	225	5.3	0.82	0.0327
								(0.0352)

Note: ( ) indicates maximum capacity of sewer.

Two sections of new sewer line are necessary for diversion. As the result of calculation, it is possible to use existing sewers until 2010, since every downstream sections of sewer lines were proved to have sufficient capacities.

2.2 <u>Proposed Trunk Sewers</u>

All sewage in Tawahi district will be collected at new pumping station placed at YPA in Tawahi district. New collecting trunk sewers are proposed for this purpose. Proposed trunk sewers are shown in Figure F.2, which shows a place of new pumping station. The existing ocean outfalls and sections of sewers which are located at seaside from the proposed trunk sewers are to be abandoned after construction of new trunk sewers and Tawahi P/S. Instead of the existing sewers in this area, new lateral and branch sewers to connect to trunk sewers will have to be planned. Area E is isolated from area D and its sewage flow is so small. Sewer pipes in area E is not designed, but its flow is included in the design of trunk sewers in area D. Hydraulic calculation sheet of proposed trunk sewers is shown in Table F.3. In conclusion, three lines of gravity sewers are proposed in Tawahi district.

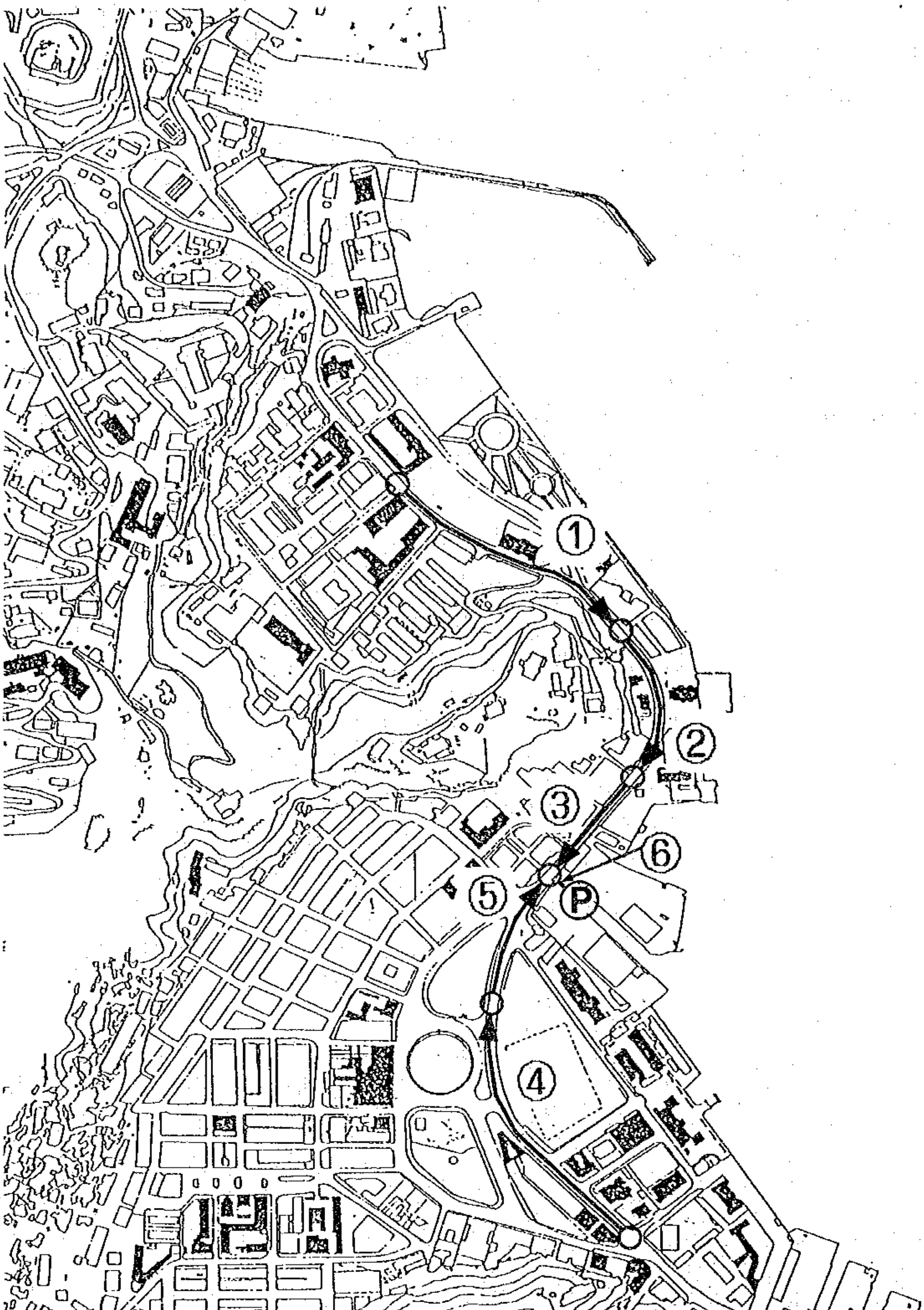

Figure F.2 Proposed Trunk Sewers in Tawahi District

Table F.3 Hydraulic Calculation Sheet of Proposed Trunk Sewers in Tawahi

No. of Sewer	Flow to	Sewerage Area: Area	Sewerage Area: Total Area	Sewer Length: Length	Sewer Length: Total Length	Design Flow	Designed Sewer: Sewer Dia.	Designed Sewer: Slope	Designed Sewer: Velocity	Designed Sewer: Sewer Capacity	Sewer Invert Elevation: Begin	Sewer Invert Elevation: End	Ground Elevation: Begin	Ground Elevation: End	Earth Cover: Begin	Earth Cover: End
		ha	ha	m	m	m3/s	mm		m/s	m3/s	m	m	m	m	m	m
1		23.5	142.5	270	270	0.0589	300	4.0	0.87	0.0612	0.774	-0.446	3.589	3.820	2.478	3.929
2		2.9	145.4	150	420	0.0634	350	3.5	0.90	0.0863	-0.496	-1.081	3.820	3.190	3.926	3.881
3	6	0.3	145.7	135	555	0.0640	350	3.5	0.90	0.0863	-1.101	-1.614	3.190	3.437	3.901	4.661
4		32.7	32.7	350	350	0.0500	300	4.0	0.87	0.0612	1.432	-0.008	3.378	3.500	1.609	3.171
5		27.6	60.3	120	470	0.0960	400	3.0	0.91	0.1141	-0.108	-0.508	3.500	3.437	3.163	3.500
6		-	206.0	15	570	0.1600	500	2.0	0.86	0.1689	-1.764	-1.834	3.437	3.500	4.647	4.780

3. Ma'alla District

3.1 Check of Existing Sewer System

Data for hydraulic calculation are as follows.

Planning Area	270.0 ha
Flow Rate	26,928 m^3/day

Ma'alla district was divided into three categories depending on population density,story of buildings and land use plan. Flow rate in each category is as follows.

Category	Area(ha)	Flow(m^3/day)	Flow(m^3/s/ha)
a	84.7	14,707	0.00201
b	92.1	7,996	0.00100
c	73.0	4,226	0.00067
Tank area	20.2	-	-
Total	270.0	26,928	

Tank area was excluded from the planning area because of no sewage. At present, there are four pumping stations in Ma'alla district, viz. Pepsi, Dakka, Dolphin and Hedjuff. Ma'alla district was divided into five areas for convenience of calculation viz. Pepsi, Wharf, Main pipe, Dolphin and Class-C areas from west to east. In the existing sewerage system, sewages from Pepsi, Wharf, Main pipe and Dolphin areas flows into the main line along the Ma'alla main road to Hedjuff pumping station. All sewage in Ma'alla district discharge to Tawahi Bay through Hedjuff pumping station. In sewerage planning, this pumping station is proposed to be replaced by a new pumping station located at the same place which convey sewage to the proposed sewage treatment plant.

Forty eight sections of sewers as illustrated in Figure F.3 were checked and its hydraulic calculation sheet is shown in Table F.4.

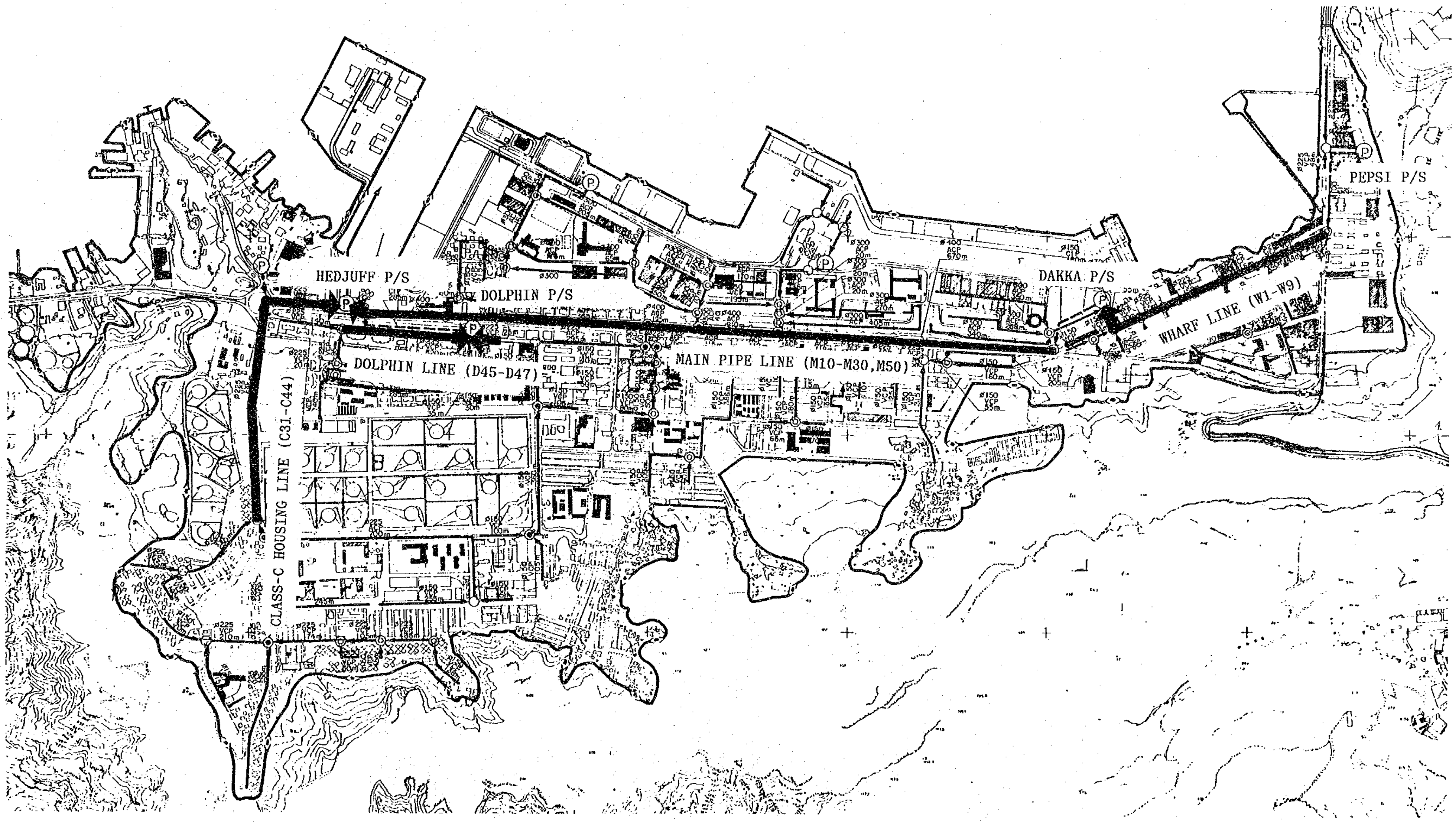

Figure F.3 Checked Sewer Lines in Ma'alla District

Table F.4 Hydraulic Calculation Sheet of Existing Sewer in Ma'alla District

No of Sewer	Flow to	Area	Flow	Sewer Length		Design Flow	Existing Sewer			
				Length	Total Length		Sewer Dia.	Slope	Velocity	Sewer Capacity
		ha	m3/s	m	m	m3/s	mm		m/s	m3/s
W1		9.9	0.0099	81.1	81.1	0.0099	300	1.80	0.58	0.0410
W2		3.2	0.0032	44.5	125.6	0.0132	300	1.78	0.58	0.0407
W3		0.9	0.0009	44.2	169.8	0.0141	300	1.79	0.58	0.0409
W4		2.4	0.0024	118.9	288.7	0.0165	300	1.80	0.58	0.0410
W5		4.8	0.0048	77.1	365.8	0.0213	300	1.82	0.58	0.0412
W6		1.3	0.0013	77.7	443.5	0.0226	300	1.88	0.59	0.0419
W7		2.8	0.0028	60.7	504.2	0.0254	300	1.86	0.59	0.0417
W8		0.7	0.0007	77.1	581.3	0.0261	300	1.85	0.59	0.0416
W9	P/S DA	1.8	0.0018	61.0	642.3	0.0279	300	2.51	0.69	0.0484
M10		13.4	0.0269	61.0	61.0	0.0549	375	1.61	0.64	0.0703
M11		0.8	0.0016	313.9	374.9	0.0565	375	1.94	0.70	0.0773
M12		0.3	0.0006	99.5	474.4	0.0571	375	1.69	0.65	0.0720
M13		16.8	0.0198	41.0	515.4	0.0769	375	1.56	0.63	0.0693
M14		0.2	0.0004	42.5	557.9	0.0773	375	1.65	0.64	0.0712
M15		0.3	0.0006	61.0	618.9	0.0779	375	1.48	0.61	0.0673
M16		0.3	0.0006	61.0	679.9	0.0785	375	1.82	0.68	0.0748
M17		6.7	0.0045	61.0	740.9	0.0830	375	1.64	0.64	0.0710
M18		0.4	0.0008	69.5	810.4	0.0838	375	1.63	0.64	0.0707
M19		65.0	0.1140	67.7	878.1	0.1979	375	1.67	0.65	0.0716
M20		0.7	0.0014	76.2	954.3	0.1993	375	1.68	0.65	0.0719
M21		0.6	0.0012	61.0	1015.3	0.2005	375	1.70	0.66	0.0724
M22		0.6	0.0012	61.0	1076.3	0.2017	375	1.69	0.65	0.0720
M23		0.6	0.0012	61.0	1137.3	0.2029	375	1.75	0.66	0.0734
M24		0.4	0.0008	30.5	1167.8	0.2037	375	1.70	0.66	0.0724
M25		12.1	0.0081	30.5	1198.3	0.2118	375	1.80	0.67	0.0744
M26		0.3	0.0006	61.0	1259.3	0.2124	375	1.69	0.65	0.0720
M27		0.5	0.0010	61.0	1320.3	0.2134	375	1.66	0.65	0.0713
M28		9.0	0.0060	61.0	1381.3	0.2194	375	1.64	0.64	0.0710
M29		1.0	0.0010	91.4	1472.7	0.2204	375	1.67	0.65	0.0717
M30		0.5	0.0005	41.5	1514.2	0.2209	375	1.69	0.65	0.0720
M50	P/S MA	0.7	0.0007	50.0	1564.2	0.2425	600	1.64	0.88	0.2486
C31		35.1	0.0353	54.9	54.9	0.0353	225	4.99	0.80	0.0317
C32		0.5	0.0005	61.0	115.9	0.0358	225	5.00	0.80	0.0317
C33		0.4	0.0004	61.0	176.9	0.0362	225	4.49	0.76	0.0301
C34		1.0	0.0010	61.0	237.9	0.0372	225	5.02	0.80	0.0318
C35		1.0	0.0010	61.0	298.9	0.0382	300	4.10	0.88	0.0619
C36		3.0	0.0030	48.8	347.7	0.0412	300	4.92	0.96	0.0678
C37		1.4	0.0014	48.8	396.5	0.0426	300	7.38	1.17	0.0830
C38		1.8	0.0018	68.6	465.1	0.0444	300	5.09	0.98	0.0690
C39		1.0	0.0010	68.6	533.7	0.0454	300	4.97	0.96	0.0682
C40		0.4	0.0004	51.7	585.4	0.0458	300	2.94	0.74	0.0524
C41		25.4	0.0170	61.0	646.4	0.0628	300	2.90	0.74	0.0521
C42		1.7	0.0011	61.0	707.4	0.0640	300	2.90	0.74	0.0521
C43		7.7	0.0052	61.0	768.4	0.0691	300	2.90	0.74	0.0521
C44	P/S MA			11	779.4	0.0691	300	7.18	1.16	0.0819
D45	D47	1.9	0.0038	79.8	79.8	0.0038	225	20.80	1.63	0.0648
D46	D47	8.2	0.0165	335.3	335.3	0.0165	225	3.37	0.66	0.0261
D47	P/S DO	0.3	0.0006	25.0		0.0209	225	6.80	0.93	0.0370

3.2 Proposed Trunk Sewer

As the result of hydraulic calculation, some sections of sewers need to be replaced. Further, hydraulic gradients of insufficient capacity pipes were checked whether rises of water level at upstream manholes are within an allowable limit. If rises exceed the limit, replacement by larger sized pipes is proposed. This is the case for main lines. Sewers in Class-C Housing area were found to be inadequate to gravitate design flow, but rises of water level are within the limit. In conclusion, sections of main line from M17 to P/S are to be replaced as illustrated in Figure F.4, and no replacement for Class-C lines. Hydraulic calculation sheet for the new line from M17 to P/S is shown in Table F.5.

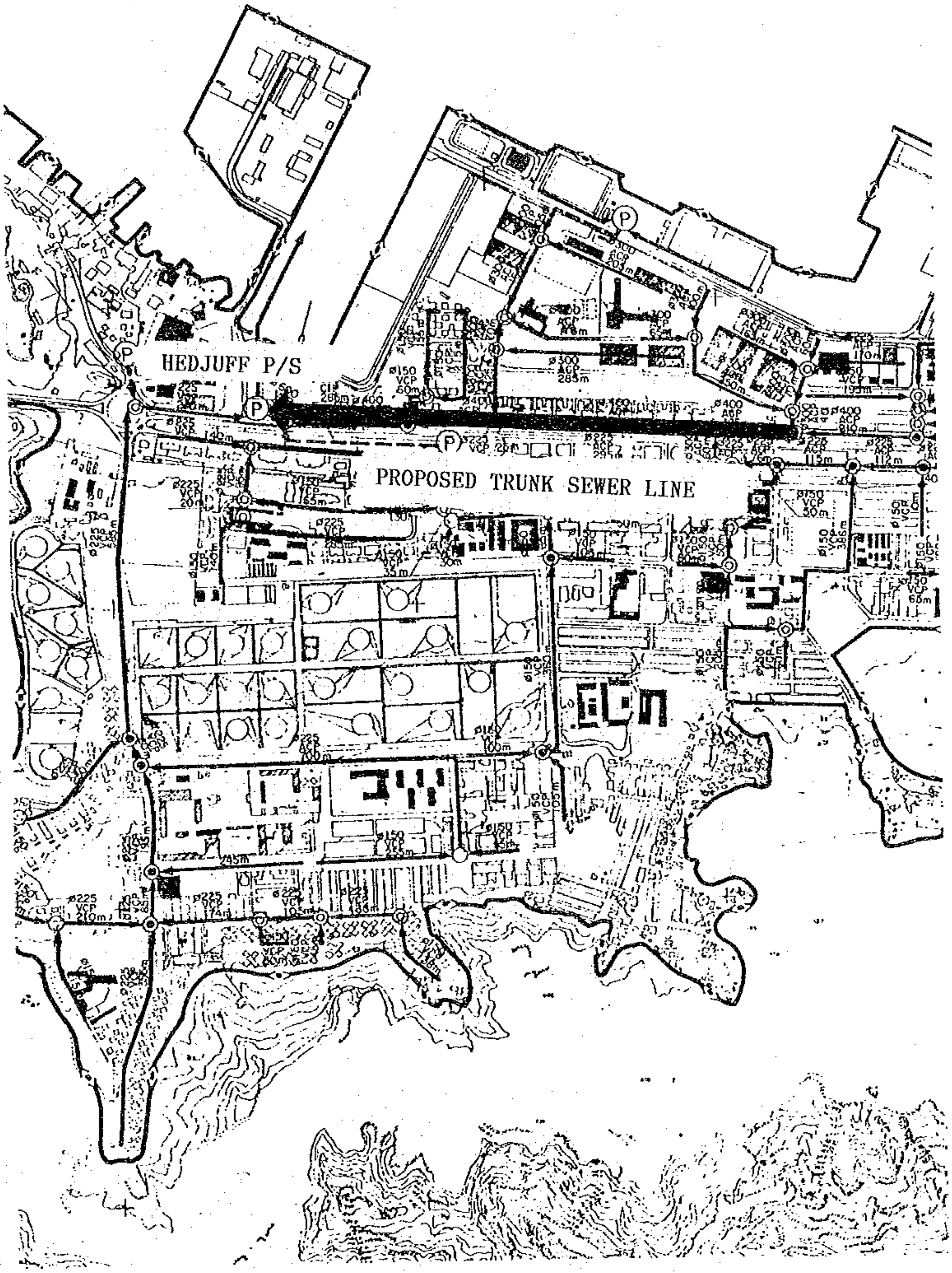

Figure F.4 Proposed Trunk Sewer Line in Ma'alla District

Table F.5 Hydraulic Calculation Sheet of Proposed Trunk Sewers in Ma'alla District

DESIGNED SEWER														
No of Sewer	Flow to	Area			Sewer Invert Elevation		Earth Cover			Sewer			Ground Elevation	
			Dia.	Slope	Begin	End	Begin	End	Vel.	Cap.		length	Begin	End
		ha	mm		m	m	m	m	m/s	m3/s	%	m	m	m
1		6.7	600	2	1.564	1.442	2.084	2.346	0.971	0.275	80.2	61	4.309	4.449
2		0.4	600	2	1.422	1.283	2.366	2.152	0.971	0.275	80.5	69.5	4.449	4.096
3		65.0	600	2	1.263	1.128	2.179	2.417	0.971	0.275	72.0	67.7	4.096	4.199
4		0.7	600	2	1.108	0.955	2.437	1.886	0.971	0.275	72.5	76.2	4.199	3.495
5		0.6	600	2	0.935	0.813	1.906	1.960	0.971	0.275	73.0	61	3.495	3.427
6		0.6	600	2	0.793	0.671	1.980	2.083	0.971	0.275	73.4	61	3.427	3.408
7		0.6	600	2	0.651	0.529	2.103	2.058	0.971	0.275	73.8	61	3.408	3.241
8		0.4	600	2	0.509	0.448	2.078	2.147	0.971	0.275	74.1	30.5	3.241	3.249
9		12.1	600	2	0.428	0.367	2.167	3.013	0.971	0.275	77.1	30.5	3.249	4.034
10		0.3	600	2	0.347	0.225	3.033	3.196	0.971	0.275	77.3	61	4.034	4.075
11		0.5	600	2	0.205	0.083	3.216	2.865	0.971	0.275	77.7	61	4.075	3.602
12		9.0	600	2	0.063	-0.059	2.885	2.655	0.971	0.275	79.9	61	3.602	3.25
13		1.0	600	2	-0.079	-0.262	2.675	3.083	0.971	0.275	80.2	91.4	3.25	3.475
14		0.5	600	2	-0.282	-0.365	3.103	3.218	0.971	0.275	80.4	41.5	3.475	3.507
15	P/S M	0.7	600	2	-0.385	-0.485	3.238	2.989	0.971	0.275	88.3	50	3.507	3.158