4. Khormaksar District

At present there are 19 pumping station and one comminutor station in this district. All sewage is collected to comminutor station and thereby is discharged to Gulf of Aden. Existing sewerage system is shown in Drawing Nos. 6 to 8. Total planning area and flow rate are 806 ha and 24,373 m^3/day, respectively. Data for hydraulic calculation are as follows.

	Area(ha)	Flow Rate (m^3/day)
Tareq Camp	41	2,447
Bader Camp	88	4,028
Seaside Area	39	-
Labor Island	12	293
Residential and Public Zones	488	17,234
Ithmas Camp	118	234
Ministry of Interior and Airport Junction Area	20	137
Total	806	24,373

Hydraulic calculation have been carried out only for existing sewered areas. Residential and public zones have ten sewered area. Each area has one pumping station to transmit sewage to the comminutor station or another area. The sewerage system in the district is illustrated schematically in Figure F.5. Hydraulic calculation sheets of each area are shown in Tables F.6 to F.15.

As the result of the calculation, almost all sewers can be used for planning.

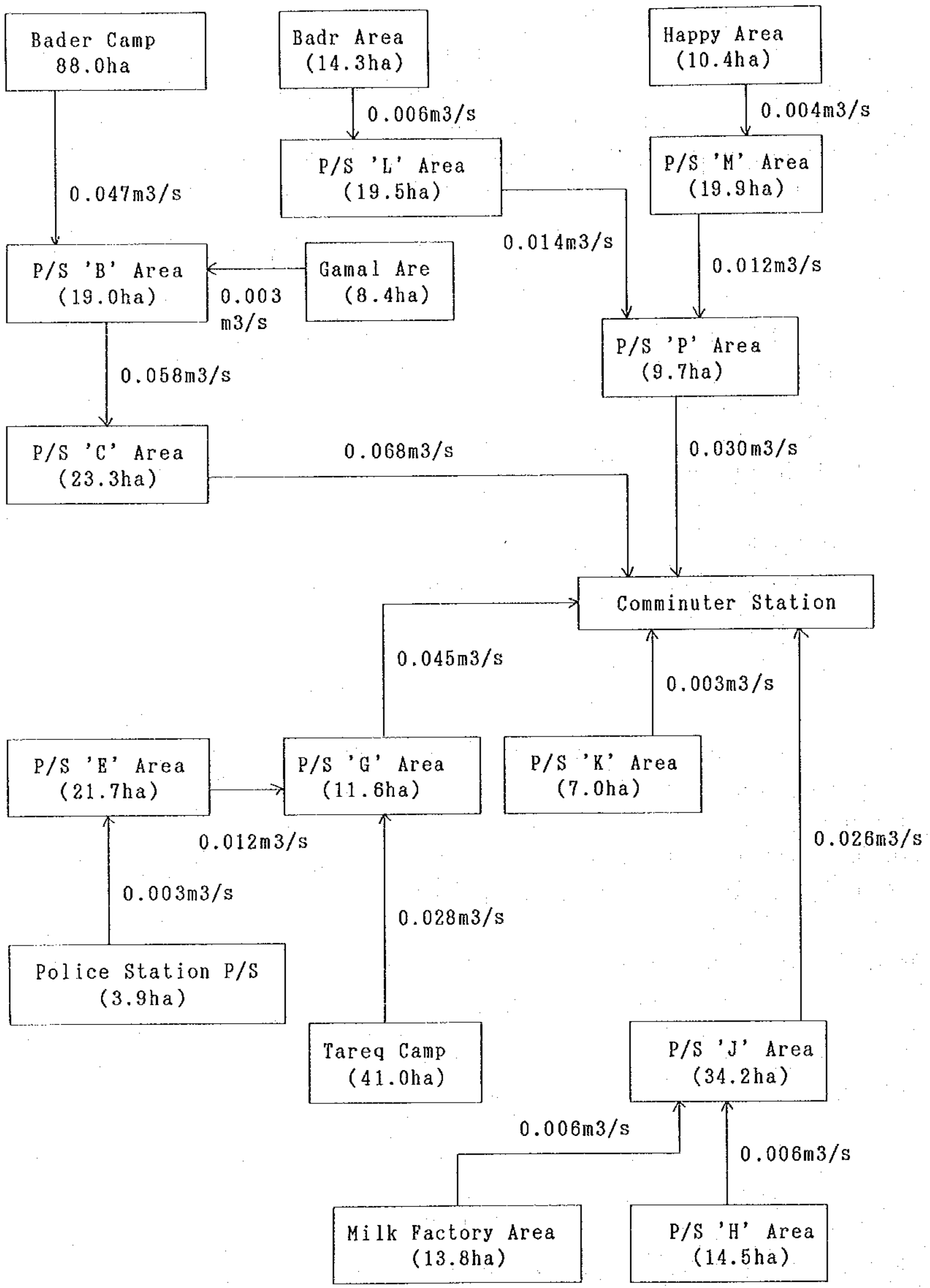

Figure F.5 Sewerage System in Khormaksar District

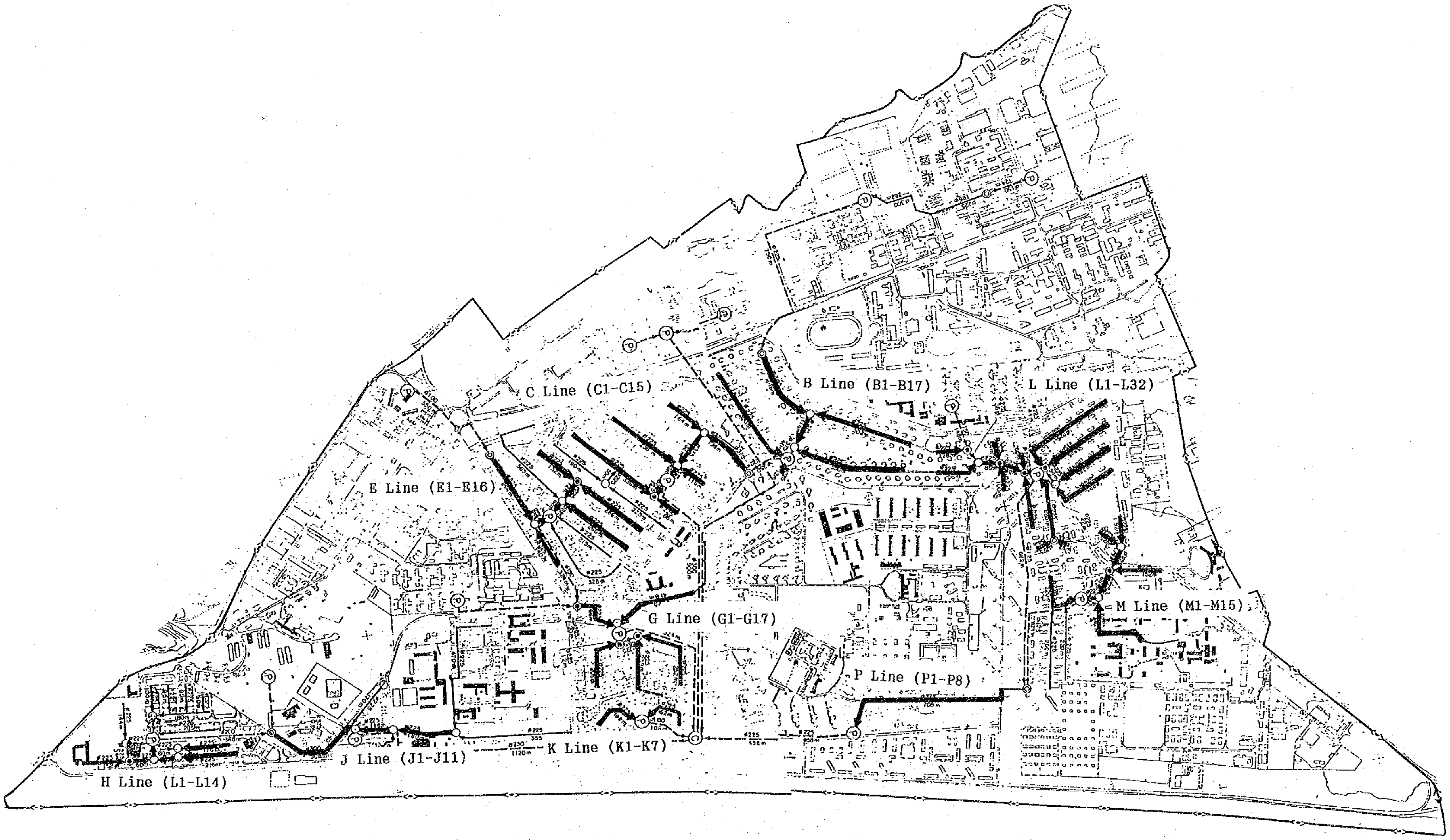

Figure F.6 Checked Sewer Lines in Khormaksar District

Table F.6 Hydraulic Calculation Sheet for L Area in Khormaksar

No.of Sewer	Flow to	Sewerage Area: Area	Sewerage Area: Total Area	Sewer Length: Length	Sewer Length: Total Length	Design Flow	Existing Sewer: Sewer Dia.	Existing Sewer: Slope	Existing Sewer: Velocity	Existing Sewer: Sewer Capacity
		ha	ha	m	m	m3/s	mm		m/s	m3/s
L1		0.5	0.5	41	41	0.000	225	5.6	0.84	0.033
L2	L3	0.7	1.2	87	128	0.000	225	5.8	0.86	0.034
		Flow from Badr Area (0.006m3/s)								
L3	L6	0.7	1.9	37	165	0.007	225	5.6	0.84	0.034
L4		0.9	0.9	110	110	0.000	225	5.6	0.84	0.033
L5		0.4	1.3	46	155	0.001	225	5.8	0.86	0.034
L6	L9	0.4	3.6	79	244	0.007	225	5.6	0.84	0.034
L7	L9	1.5	1.5	101	101	0.001	225	5.8	0.86	0.034
L8		0.7	0.7	55	55	0.000	225	5.6	0.84	0.033
L9	L13	0.4	6.2	82	326	0.009	225	5.6	0.84	0.033
L10		0.7	0.7	88	88	0.000	225	5.5	0.84	0.033
L11	L13	0.7	1.4	88	177	0.001	225	5.5	0.84	0.033
L12		0.5	0.5	61	61	0.000	225	5.5	0.84	0.033
L13		0.2	8.3	37	363	0.009	225	5.6	0.84	0.034
L14	P/S		8.3	8	370	0.009	225	5.6	0.85	0.034
L15		1.0	1.0	110	110	0.000	225	5.6	0.84	0.033
L16		1.0	2.0	101	210	0.001	225	5.5	0.84	0.033
L17		1.1	3.1	82	293	0.001	225	5.6	0.85	0.034
L18	L27	0.2	3.3	76	369	0.001	225	5.8	0.86	0.034
L19		0.7	0.7	79	79	0.000	225	5.6	0.84	0.034
L20		0.8	1.5	82	162	0.001	225	5.5	0.84	0.033
L21	L24	0.2	1.7	64	226	0.001	225	8.9	1.06	0.042
L22		0.7	0.7	110	110	0.000	225	5.6	0.84	0.033
L23		0.6	1.3	94	204	0.001	225	6.8	0.93	0.037
L24	L27	0.2	3.2	55	280	0.001	225	13.3	1.30	0.052
L25		1.0	1.0	104	104	0.000	225	5.6	0.84	0.034
L26		0.6	1.6	107	210	0.001	225	5.7	0.85	0.034
L27	P/S		8.1	21	390	0.003	225	4.3	0.74	0.029
L28	L30	0.8	0.8	34	34	0.000	225	5.5	0.83	0.033
L29		1.0	1.0	27	27	0.000	225	5.6	0.84	0.033
L30		0.6	2.4	101	134	0.001	225	5.5	0.84	0.033
L31		0.6	3.0	101	235	0.001	225	5.7	0.85	0.034
L32	P/S		3.0	18	253	0.001	225	5.0	0.80	0.032
		Flow to P1 Sewer (0.014m3/s)								

Table F.7 Hydraulic Calculation Sheet for M Area in Khormaksar

No.of Sewer	Flow to	Sewerage Area: Area	Sewerage Area: Total Area	Sewer Length: Length	Sewer Length: Total Length	Design Flow	Existing Sewer: Sewer Dia.	Existing Sewer: Slope	Existing Sewer: Velocity	Existing Sewer: Sewer Capacity
		ha	ha	m	m	m3/s	mm		m/s	m3/s
M1		5.8	5.8	43	43	0.002	225	5.6	0.84	0.034
M2		0.4	6.2	34	76	0.003	225	5.6	0.85	0.034
M3		0.3	6.5	82	158	0.003	225	5.9	0.87	0.035
M4	M10	0.6	7.1	107	265	0.003	225	5.7	0.85	0.034
M5	M7	1.7	1.7	91	91	0.001	225	5.3	0.82	0.033
M6		1.0	1.0	91	91	0.000	225	5.3	0.82	0.033
M7	M9	0.8	3.5	73	165	0.001	225	5.8	0.86	0.034
		Flow from Happy Area (0.004m3/s)								
M8		3.0	3.0	152	152	0.005	225	5.6	0.85	0.034
M9		1.3	7.8	88	253	0.007	225	5.5	0.84	0.033
M10	M15	0.5	15.4	52	317	0.010	225	5.9	0.87	0.034
M11		1.7	1.7	110	110	0.001	225	5.6	0.84	0.033
M12		1.7	3.4	40	149	0.001	225	4.0	0.71	0.028
M13		0.4	3.8	34	183	0.002	225	7.1	0.95	0.038
M14		0.7	4.5	37	219	0.002	225	5.8	0.86	0.034
M15	P/S		19.9	3	320	0.008	225	6.0	0.87	0.035
		Flow to P1 Sewer (0.012m3/s)								

Table F.8 Hydraulic Calculation Sheet for P Area in Khormaksar

No.of Sewer	Flow to	Sewerage Area: Area	Sewerage Area: Total Area	Sewer Length: Length	Sewer Length: Total Length	Design Flow	Existing Sewer: Sewer Dia.	Existing Sewer: Slope	Existing Sewer: Velocity	Existing Sewer: Sewer Capacity
		ha	ha	m	m	m3/s	mm		m/s	m3/s
		Flow from P/S 'L & M' (0.026m3/s)								
P1		0.5	0.5	59	59	0.026	375	4.6	1.07	0.118
P2		0.6	1.1	44	104	0.026	375	4.2	1.03	0.114
P3		0.9	2.0	62	166	0.027	375	3.2	0.90	0.099
P4		1.6	3.6	107	273	0.027	375	3.3	0.91	0.100
P5		1.4	5.0	93	366	0.028	375	3.1	0.88	0.097
P6		1.9	6.9	120	486	0.029	375	3.2	0.90	0.099
P7		1.6	8.5	30	517	0.029	375	3.0	0.87	0.096
P8		1.2	9.7	107	623	0.030	375	2.7	0.82	0.091
P9	P/S		9.7	5	628	0.030	375	6.7	1.30	0.143
		Flow to Comminuter Station (0.030m3/s)								

Table F.9 Hydraulic Calculation Sheet for B Area in Khormaksar

No.of Sewer	Flow to	Sewerage Area: Area	Sewerage Area: Total Area	Sewer Length: Length	Sewer Length: Total Length	Design Flow	Existing Sewer: Sewer Dia.	Existing Sewer: Slope	Existing Sewer: Velocity	Existing Sewer: Sewer Capacity
		ha	ha	m	m	m3/s	mm		m/s	m3/s
B1		1.6	1.6	101	101	0.001	225	5.5	0.84	0.033
B2		1.3	2.9	98	198	0.001	225	5.5	0.84	0.033
B3	B7	1.4	4.3	104	302	0.002	225	5.6	0.84	0.034
		Flow from Badr Camp (0.047m3/s)								
B4		2.6	2.6	107	107	0.048	300	4.2	0.89	0.063
B5		0.7	3.3	61	168	0.048	300	6.4	1.09	0.077
B6		1.0	4.3	78	245	0.049	300	6.4	1.10	0.078
B7	B12	1.0	9.6	116	418	0.051	375	2.7	0.82	0.091
B8		0.9	0.9	70	70	0.000	225	5.6	0.84	0.033
B9		0.9	1.8	67	137	0.001	225	5.5	0.84	0.033
B10		1.1	2.9	110	247	0.001	225	5.6	0.84	0.033
B11		1.1	4.0	104	351	0.002	225	5.6	0.84	0.034
B12	P/S	0.2	13.8	40	457	0.053	375	2.7	0.82	0.091
B13		2.0	2.0	107	107	0.001	225	5.7	0.85	0.034
B14		1.1	3.1	104	210	0.001	225	5.4	0.83	0.033
B15		1.1	4.2	88	299	0.002	225	5.6	0.85	0.034
B16	P/S	0.5	4.7	52	351	0.002	225	5.6	0.84	0.034
		Flow from Gamal Area (0.003m3/s)								
B17	P/S	0.5	0.5	12	12	0.003	225	5.5	0.84	0.033
		Flow to C1 Sewer (0.058m3/s)								

Table F.10 Hydraulic Calculation Sheet for C Area in Khormaksar

No.of Sewer	Flow to	Sewerage Area: Area	Sewerage Area: Total Area	Sewer Length: Length	Sewer Length: Total Length	Design Flow	Existing Sewer: Sewer Dia.	Existing Sewer: Slope	Existing Sewer: Velocity	Existing Sewer: Sewer Capacity
		ha	ha	m	m	m3/s	mm		m/s	m3/s
		Flow from P/S 'B' (0.058m3/s)								
C1		1.2	1.2	89	89	0.058	375	4.8	1.10	0.121
C2	C5	1.7	2.9	108	197	0.059	375	4.8	1.10	0.121
C3		3.5	3.5	82	82	0.001	225	5.6	0.84	0.033
C4		1.6	5.1	82	165	0.002	225	5.6	0.84	0.033
C5		0.3	8.3	72	269	0.061	375	2.7	0.82	0.091
C6	C10	0.4	8.7	59	328	0.062	375	2.9	0.86	0.095
C7		3.0	3.0	105	105	0.001	225	5.5	0.84	0.033
C8	C10	1.3	4.3	102	207	0.002	225	5.7	0.85	0.034
C9		2.5	2.5	116	116	0.001	225	5.8	0.86	0.034
C10	P/S	0.4	15.9	43	371	0.064	375	2.7	0.83	0.091
C11		3.1	3.1	85	85	0.001	225	5.6	0.85	0.034
C12		1.0	4.1	85	171	0.002	225	5.6	0.85	0.034
C13	C15	1.1	5.2	85	256	0.002	225	5.6	0.84	0.034
C14		1.8	1.8	85	85	0.001	225	5.4	0.83	0.033
C15	P/S	0.4	7.4	73	329	0.003	225	5.4	0.83	0.033
		Flow to Comminuter Station (0.068m3/s)								

Table F.11 Hydraulic Calculation Sheet for E Area in Khormaksar

No. of Sewer	Flow to	Sewerage Area: Area	Sewerage Area: Total Area	Sewer Length: Length	Sewer Length: Total Length	Design Flow	Existing Sewer: Sewer Dia.	Existing Sewer: Slope	Existing Sewer: Velocity	Existing Sewer: Sewer Capacity
		ha	ha	m	m	m3/s	mm		m/s	m3/s
E1		2.4	2.4	88	88	0.001	225	5.5	0.84	0.033
E2		0.8	3.2	88	177	0.001	225	5.5	0.84	0.033
E3	E6	0.9	4.1	88	265	0.002	225	5.5	0.84	0.033
E4		2.1	2.1	76	76	0.001	225	5.6	0.85	0.034
E5		0.9	3.0	76	152	0.001	225	5.6	0.85	0.034
E6	E10	0.7	7.8	72	337	0.003	225	5.1	0.81	0.032
E7		3.0	3.0	88	88	0.001	225	5.5	0.84	0.033
E8		1.1	4.1	88	177	0.002	225	5.7	0.85	0.034
E9		0.9	5.0	88	265	0.002	225	6.0	0.88	0.035
E10	P/S	0.9	13.7	62	399	0.006	225	6.1	0.88	0.035
		Flow from Police Station P/S (0.002m3/s)								
E11		1.9	5.8	79	79	0.002	225	5.9	0.87	0.034
E12		1.2	7.0	79	157	0.003	225	5.9	0.87	0.034
E13	E16	0.9	7.9	107	264	0.003	225	5.9	0.87	0.034
E14		2.6	2.6	76	76	0.001	225	5.6	0.85	0.034
E15		1.0	3.6	64	140	0.001	225	5.6	0.85	0.034
E16	P/S	0.4	11.9	68	332	0.005	225	5.9	0.87	0.034
		Flow to G14 Sewer (0.012m3/d)								

Table F.12 Hydraulic Calculation Sheet for G Area in Khormaksar

No. of Sewer	Flow to	Sewerage Area: Area	Sewerage Area: Total Area	Sewer Length: Length	Sewer Length: Total Length	Design Flow	Existing Sewer: Sewer Dia.	Existing Sewer: Slope	Existing Sewer: Velocity	Existing Sewer: Sewer Capacity
		ha	ha	m	m	m3/s	mm		m/s	m3/s
G1		1.3	1.3	61	61	0.001	225	5.8	0.86	0.034
G2		0.6	1.9	58	119	0.001	225	5.3	0.82	0.033
G3		1.0	2.9	67	186	0.001	225	6.1	0.88	0.035
G4	G7	0.5	3.4	67	253	0.001	225	5.3	0.82	0.033
G5		1.2	1.2	62	62	0.001	225	5.6	0.84	0.033
G6		0.5	1.7	62	125	0.001	225	5.7	0.85	0.034
G7	G11	0.3	5.4	49	302	0.002	225	5.5	0.84	0.033
G8		1.7	1.7	82	82	0.001	225	5.6	0.84	0.033
G9		0.7	2.4	43	125	0.001	225	5.7	0.85	0.034
G10		0.6	3.0	82	207	0.001	225	5.6	0.84	0.033
G11	P/S		8.4	12	314	0.004	225	5.5	0.84	0.033
G12		1.1	1.1	110	110	0.000	225	5.6	0.84	0.033
G13	G17	0.7	1.8	88	198	0.001	225	5.5	0.84	0.033
		Flow from P/S 'E' and Tareq Camp (0.040m3/s)								
G14		0.3	0.3	55	55	0.040	300	5.6	1.02	0.072
G15		0.4	0.7	40	94	0.040	300	5.5	1.02	0.072
G16		0.7	1.4	72	166	0.041	300	5.6	1.02	0.072
G17	P/S		3.2	21	219	0.041	300	7.4	1.17	0.083
		Flow to Comminuter Station (0.045m3/s)								

Table F.13 Hydraulic Calculation Sheet for K Area in Khormaksar

No.of Sewer	Flow to	Sewerage Area: Area	Sewerage Area: Total Area	Sewer Length: Length	Sewer Length: Total Length	Design Flow	Existing Sewer: Sewer Dia.	Existing Sewer: Slope	Existing Sewer: Velocity	Existing Sewer: Sewer Capacity
		ha	ha	m	m	m3/s	mm		m/s	m3/s
K1		1.3	1.3	52	52	0.001	225	5.6	0.84	0.034
K2		0.5	1.8	43	94	0.001	225	5.5	0.84	0.033
K3		0.9	2.7	34	128	0.001	225	5.5	0.84	0.033
K4	P/S	0.9	3.6	34	162	0.001	225	6.1	0.88	0.035
K5		1.4	1.4	40	40	0.001	225	5.5	0.84	0.033
K6		0.9	2.3	61	101	0.001	225	5.6	0.84	0.033
K7	P/S	1.1	3.4	61	162	0.001	225	5.9	0.86	0.034
		Flow to Comminuter Station (0.003m3/s)								

Table F.14 Hydraulic Calculation Sheet for H Area in Khormaksar

No.of Sewer	Flow to	Sewerage Area: Area	Sewerage Area: Total Area	Sewer Length: Length	Sewer Length: Total Length	Design Flow	Existing Sewer: Sewer Dia.	Existing Sewer: Slope	Existing Sewer: Velocity	Existing Sewer: Sewer Capacity
		ha	ha	m	m	m3/s	mm		m/s	m3/s
H1		0.5	0.5	91	91	0.000	225	5.6	0.84	0.033
H2		0.7	1.2	91	183	0.000	225	5.5	0.84	0.033
H3	H6	0.1	1.3	34	216	0.001	225	5.5	0.83	0.033
H4		0.3	0.3	85	85	0.000	225	5.4	0.83	0.033
H5		0.3	0.6	85	171	0.000	225	9.8	1.12	0.044
H6	H14	0.3	2.2	79	296	0.001	225	5.6	0.84	0.034
H7		1.8	1.8	76	76	0.001	225	5.6	0.85	0.034
H8	H10	0.8	2.6	76	152	0.001	225	5.6	0.85	0.034
H9		0.5	0.5	67	67	0.000	225	5.5	0.83	0.033
H10	H13	0.1	3.2	34	186	0.001	225	5.5	0.83	0.033
H11		1.1	1.1	85	85	0.000	225	5.7	0.85	0.034
H12		1.4	2.5	59	145	0.001	225	6.7	0.92	0.037
H13		0.2	5.9	76	262	0.002	225	7.6	0.98	0.039
H14	P/S		8.1	9	305	0.003	225	6.7	0.92	0.037
	Flow to J7 Sewer (0.006m3/s)									

Table F.15 Hydraulic Calculation Sheet for J Area in Khormaksar

No. of Sewer	Flow to	Sewerage Area: Area	Sewerage Area: Total Area	Sewer Length: Length	Sewer Length: Total Length	Design Flow	Existing Sewer: Sewer Dia.	Existing Sewer: Slope	Existing Sewer: Velocity	Existing Sewer: Sewer Capacity
		ha	ha	m	m	m3/s	mm		m/s	m3/s
J1		27.1	27.1	110	110	0.011	225	5.6	0.84	0.033
J2	J5	2.2	29.3	70	180	0.012	225	5.7	0.85	0.034
J3		0.8	0.8	15	15	0.000	225	5.6	0.85	0.034
J4		0.1	0.9	27	43	0.000	225	5.2	0.82	0.032
J5		0.6	30.8	30	210	0.013	225	6.0	0.87	0.035
J6	J11	0.5	31.3	87	297	0.013	225	5.5	0.84	0.033
		Flow from P/S 'H' and Milk Factory Area (0.012m3/s)								
J7		0.9	0.9	98	98	0.012	300	3.7	0.83	0.058
J8		0.2	1.1	61	158	0.012	300	3.6	0.82	0.058
J9		0.8	1.9	113	272	0.013	300	3.6	0.82	0.058
J10		1.0	2.9	37	308	0.013	300	8.9	1.29	0.091
J11	P/S		34.2	10	319	0.026	375	2.9	0.86	0.095
		Flow to Comminuter Station (0.030m3/s)								

5. Crater District

The whole district has already been covered by sewerage system as shown in Drawing No. 5. There are two pumping station in this district. Data for hydraulic calculation of existing sewers are as follows.

Planning Area	235.0 ha
Flow Rate	29,268 m^3/day

Crater district was divided into three categories depending on population density and story of buildings. Flow rate of each category is shown in the following.

Category	Area(ha)	Flow(m^3/day)	Flow(m^3/s/ha)
a	45.0	7,265	0.00187
b	111.5	14,399	0.00149
c	78.5	7,604	0.00112
Total	235.0	29,268	

Results of hydraulic calculation are shown in Table F.16. As shown in the table, capacities of all sewers are sufficient to gravitate design flows.

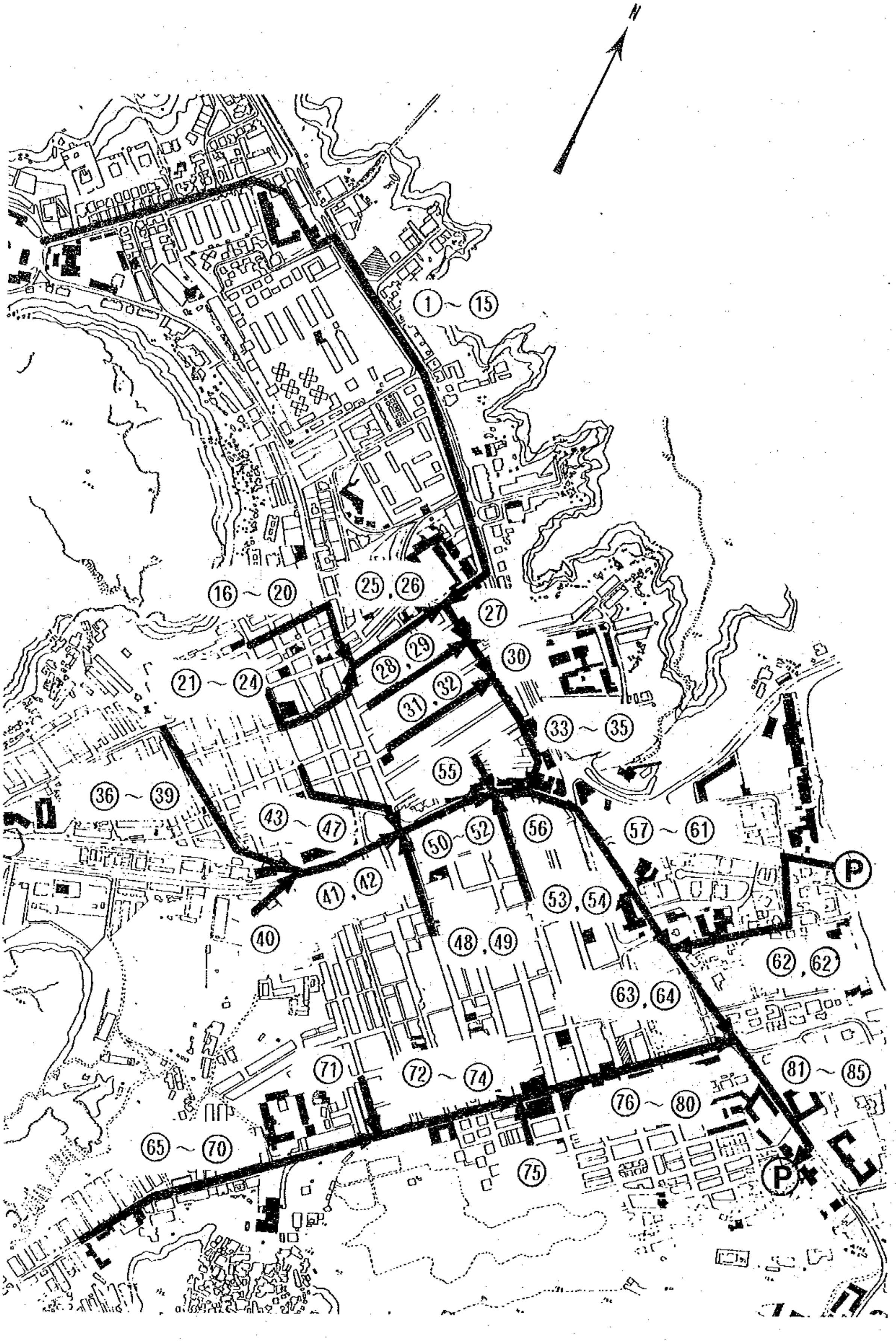

Figure F.7 Checked Sewer Lines in Crater District

Table F.16 Hydraulic Calculation Sheet for Existing Sewer System in Crater

No of Sewer	Flow to	Area	Flow	Sewer Length: Length	Sewer Length: Total Length	Design Flow	Existing Sewer: Sewer Dia.	Existing Sewer: Slope	Existing Sewer: Velocity	Existing Sewer: Sewer Capacity
		ha	m3/s	m	m	m3/s	mm		m/s	m3/s
1		9.4	0.011	112	112	0.011	225	15.6	1.41	0.056
2		0.8	0.001	118	231	0.012	225	19.6	1.58	0.063
3		3.2	0.004	109	340	0.016	225	17.9	1.51	0.060
4		3.8	0.006	80	420	0.022	225	14.0	1.34	0.053
5				91	511	0.022	225	17.7	1.50	0.060
6		4.2	0.006	18	530	0.028	225	8.3	1.03	0.041
7				67	597	0.028	225	9.1	1.08	0.043
8		4.8	0.005	110	707	0.034	225	8.3	1.03	0.041
9				110	816	0.034	225	8.9	1.07	0.042
10		1.3	0.001	48	864	0.035	225	10.9	1.18	0.047
11				91	955	0.035	225	10.0	1.13	0.045
12				54	1,009	0.035	225	10.4	1.15	0.046
13		17.4	0.026	24	1,034	0.061	300	10.6	1.41	0.100
14		3.8	0.004	105	1,138	0.065	300	8.3	1.25	0.088
15	27	1.1	0.001	71	1,210	0.067	300	8.1	1.23	0.087
16		2.0	0.004	67	67	0.004	225	15.9	1.42	0.057
17				52	119	0.004	225	11.8	1.23	0.049
18				37	155	0.004	225	16.7	1.46	0.058
19		2.0	0.004	37	192	0.007	225	8.3	1.03	0.041
20	25			34	226	0.007	225	9.1	1.08	0.043
21		3.3	0.005	45	45	0.005	225	27.0	1.86	0.074
22		1.1	0.002	43	88	0.007	225	14.9	1.38	0.055
23		0.9	0.002	37	124	0.008	225	14.9	1.38	0.055
24				104	228	0.008	225	12.2	1.25	0.050
25				100	328	0.016	225	6.1	0.88	0.035
26				104	432	0.016	225	8.8	1.06	0.042
27	30	1.7	0.003	79	1,289	0.085	400	11.6	1.78	0.224
28		0.6	0.001	86	86	0.001	225	9.6	1.11	0.044
29		5.2	0.008	97	183	0.009	225	8.3	1.03	0.041
30	33			79	1,368	0.094	400	11.6	1.78	0.224
31		0.7	0.001	85	85	0.001	225	10.8	1.17	0.047
32		1.3	0.002	103	188	0.004	225	10.8	1.17	0.047
33				79	1,447	0.097	400	11.9	1.81	0.227
34		7.9	0.009	58	1,506	0.106	400	3.9	1.03	0.130
35	57			65	1,571	0.106	400	5.6	1.24	0.156
36		6.4	0.010	76	76	0.010	300	11.3	1.45	0.103
37				73	149	0.010	300	10.8	1.42	0.100
38				110	259	0.010	300	10.0	1.37	0.097
39	41			109	367	0.010	300	16.0	1.73	0.122
40		8.3	0.009	71	71	0.009	225	37.7	2.19	0.087
41		8.2	0.012	76	443	0.031	600	7.2	1.84	0.521
42	50			110	554	0.031	600	6.9	1.80	0.510
43		2.0	0.004	62	62	0.004	225	15.7	1.41	0.056
44				73	135	0.004	225	21.7	1.66	0.066
45				53	188	0.004	225	12.7	1.27	0.051
46				29	217	0.004	225	12.7	1.27	0.051
47	50			36	253	0.004	225	12.7	1.27	0.051

48		5.6	0.010	85	85	0.010	225	5.0	0.80	0.032
49				100	185	0.010	225	4.9	0.79	0.031
50				20	574	0.045	600	5.3	1.58	0.447
51		5.7	0.011	37	611	0.056	600	5.3	1.58	0.447
52	56			98	709	0.056	600	4.2	1.41	0.398
53		4.7	0.009	90	90	0.009	225	2.7	0.59	0.023
54	56			101	192	0.009	225	2.7	0.59	0.023
55		3.7	0.007	35	35	0.007	225	14.8	1.37	0.055
56				66	775	0.072	600	2.3	1.04	0.294
57				76	1,647	0.178	750	0.8	0.71	0.315
58				26	1,673	0.178	750	0.8	0.71	0.315
59		5.0	0.009	50	1,723	0.187	750	0.8	0.71	0.315
60				114	1,837	0.187	750	0.8	0.71	0.315
61	63			91	1,928	0.187	750	1.0	0.80	0.352
62'		26.9	0.040	302	302	0.040	100	Force Main from P/S		
62				102	404	0.040	225	11.4	1.21	0.048
63				102	2,030	0.228	750	0.9	0.76	0.334
64	81			83	2,113	0.228	750	1.1	0.84	0.369
65		4.5	0.005	126	126	0.005	225	51.5	2.56	0.102
66				111	237	0.005	225	45.5	2.41	0.096
67				95	332	0.005	225	40.7	2.28	0.091
68				108	440	0.005	225	18.3	1.53	0.061
69		14.3	0.016	46	486	0.021	225	16.7	1.46	0.058
70	72			50	536	0.021	225	18.1	1.52	0.060
71		5.9	0.009	102	102	0.009	225	3.9	0.71	0.028
72		3.8	0.004	85	621	0.034	225	15.5	1.41	0.056
73				90	710	0.034	225	15.8	1.42	0.056
74	76			60	771	0.034	225	16.2	1.44	0.057
75		3.9	0.004	64	64	0.004	225	3.6	0.68	0.027
76		3.6	0.007	86	857	0.045	300	14.8	1.66	0.118
77				85	942	0.045	300	5.4	1.01	0.071
78		17.3	0.029	52	994	0.074	400	3.5	0.98	0.123
79				79	1,073	0.074	400	3.5	0.98	0.123
80				78	1,151	0.074	400	3.5	0.98	0.123
81		12.9	0.019	112	2,225	0.321	750	1.5	0.98	0.431
82				101	2,326	0.321	750	1.5	0.98	0.431
83				30	2,356	0.321	750	2.5	1.27	0.560
84				6	2,362	0.321	750	1.0	0.80	0.352
85				11	2,373	0.321	900	0.8	0.81	0.518
to Main Pumping Station										

APPENDIX G

DESIGN OF SEWAGE TREATMENT PLANT AND PUMPING STATIONS

APPENDIX G

DESIGN OF SEWAGE TREATMENT PLANT AND PUMPING STATIONS

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APPENDIX G

DESIGN OF SEWAGE TREATMENT PLANT AND PUMPING STATION

1. Design of Sewage Treatment Plant

1.1 Introduction

This section presents design of sewage treatment plant carried out on the basis of modified sewage flow and characteristics in Section 3.4 of the main report. Design criteria and standards described in the following subsections are those which are internationally accepted. Conditions in the study area which will affect the design of treatment plant have been carefully examined and determined. These are also described below.

Treatment capacity is the sewage flow rate in 2010, which is expected to be produced in the four districts, viz. Ma'alla, Tawahi, Crater and Khormaksar. Staged construction up to 2010 to keep pace with the implementation program is considered. As a result, the major treatment works, i.e. stabilization pond system are divided into three process trains, the first one for the first phase program up to 2000, and the remaining two trains for the second phase program up to 2010.

Treatment goals are set taking into account environmental protection for the receiving water, i.e. Khormaksar beach, and future reuse of effluent for growing greenery. Permissible levels of BOD concentration and coliform number of the effluent are set at 60 mg/l and 100/100ml respectively. In order to achieve this goal and minimize the construction cost, process train consisting of an anaerobic pond, a facultative pond and two maturation ponds in series is adopted. However, for the first phase construction, anaerobic pond is divided into two ponds taking into maintenance work, i.e. periodical desludging of the pond.

Ancillary works necessary for operation and maintenance of the treatment plant have also been designed and described briefly. These include administration building comprising offices and a laboratory and inlet works for flow measurement.

Water levels in each treatment facility and levels of embankment have been determined considering many factors, such as tide levels at discharging point, present ground level at the site and minimum volume of earth work for embankment. Hydraulic calculation of the water levels is described in detail in this section.

Finally, configuration of embankment has been designed to satisfy engineering requirements and minimize construction cost. Since soils at site can not be used and soils for embankment should be transported from other places, volume of embankment will affect the construction cost significantly. Configuration of embankment has been designed carefully in this regard. Volume of earth work inclusive of embankment has been calculated.

1.2 Design Basis

Flow rate (Daily average in 2010)		48,769 m^3/day
Influent BOD		250 mg/l
Effluent BOD		60 mg/l
Number of coliform groups (Fc)	Influent	1 X 10^7 /100ml
	Effluent	100 /100ml
Temperature	25°C (Minimum monthly average, January)	

Treatment plant has three process trains. Flow rate for one train is;

48,769 / 3 = 16,256 m^3/day.

Flow rate of the first phase program which is produced in Ma'alla and Tawahi in 2000 is;

10,500 + 5,835 = 16,335 m^3/day

1.3 Anaerobic Pond

In order to reduce the total area for treatment and ensure efficient removal of organic pollutant, anaerobic pond is provided ahead of facultative pond.

Design criteria and calculation are described below. Figures in parentheses are those for the first phase program (facultative and maturation ponds design is presented in the same manner).

Retention time	1.5 day
Effective water depth	3.0 m
Area for need	16,256 X 1.5 / 3.0 = 8,128 (8,168) m^2
Size	Width 80 m X Length 140 m X Depth 3.5 m
Area	80 X 140 = 11,200 m^2
Volume	11,200 X 3.0 = 33,600 m^3
Retention time	33,600 / 16,256 = 2.07 (2.06) day
Population	231,300 / 3 = 77,100 (102,900)
Accumulation for 5 years	0.04 X 77,100 X 5 = 15,420 (20,580) m^3
Accumulation depth	15,420 / 11,200 = 1.38 (1.84) m
BOD loading	16,256 X 250 X 10^{-3} = 4,064 (4,084) kg/day
BOD areal loading	4,064 / 1.12 = 3,629 (3,646) kg/ha/day (1,000 - 6,000)
BOD volumetric loading	4,064 / 33,600 = 0.12 (0.12) kg/m^3/day (0.1 - 0.4)
BOD removal rate	60 %
Effluent BOD	250 X (1.0 - 0.6) = 100 mg/l

Effluent number of coliform group

$$N_e = N_i / (1 + K_b \times t)$$

where, N_e : effluent number of coliform group

N_i : influent number of coliform group

K_b : first order rate constant

$K_b = 2.6\ (1.19)^{T-20} = 6.2$ at 25°C

t : retention time

$$N_e = 1 \times 10^7 / (1 + 6.2 \times 2.07)$$
$$= 7.24 \times 10^5\ (7.27 \times 10^5)\ /100ml$$

Minimum requirement for retention time is considered to be 1.5 days, but in determining the dimensions of pond, sludge accumulation is critical. Therefore area and volume of pond are determined by allowable depth of sludge accumulation, resulting in longer retention time.

1.4 Facultative Pond

BOD loading	100 X 16,256 X 10^{-3} = 1,626 (1,634) kg/day
Minimum retention time	5 days
Effective water depth	1.5 - 2.0 m
Area for need	3.69 (3.71) ha

From McGarry and Pescod's experimental formula, maximum BOD areal loading is;

$S_{max} = 11.2 \times (1.054)^T$

$= 643$ (kg/ha/day) at T = 77° F (= 25° C)

Standard BOD areal loading by the simple formula recommended by World Bank manual is;

$S = 20\ T - 60$,

and at T = 25° C, BOD loading is;

$S = 440$ kg/ha/day

In comparison of these two figures, the latter is adopted to allow safety factor. Thus, area for need is ;

$A = 1{,}626 / 440 = 3.69$ (3.71) ha

Size	Width 140 m X Length 360 m X Depth 1.65 m
Area	50,400 m^2
Volume	83,160 m^3
BOD areal loading	1,626 / 5.04 = 323 (324) kg/ha/day
Retention time	5.12 (5.09) day

For the design of facultative pond, following factors have been considered.

- BOD areal loading is less than 440 kg/ha/day.
- Retention time is more than 5 days.

Retention time is more critical in this case. Thus area is determined by the retention time of 5 days with effective water depth of 1.65 m (effective water depth, see subsection 1.8).

Effluent BOD 34 (34) mg/l

BOD removal is assumed to be first-order reaction. Equation of effluent BOD is;

$L_e = L_i / (1 + K_1 \times t)$,

where: L_e = Effluent BOD (mg/l)

L_i = Influent BOD (mg/l)

K_1 = First order rate constant

$K_1 = 0.3\ (1.05)^{T-20}$

$K_1 = 0.38$ at T = 25° C

t = retention time.

Therefore,

L_e = 100 / (1 + 0.38 X 5.12) = 34 (34) mg/l < 60 mg/l, OK

Eff. No. of coliform group $Ne = 7.24 \times 10^5 / (1 + 6.2 \times 5.12)$

$= 2.21 \times 10^4\ (2.23 \times 10^4)$ / 100ml

1.5 Maturation Pond

Since the reduction process of coliform group number follows first order kinetic model, multiple number of maturation pond is more efficient than single pond of the same capacity. Two serial maturation ponds are considered.

Retention time $N_e = N_i / (1 + K_b \times t)^2$

where, N_i : influent number of coliform group

N_e : effluent number of coliform group (100 /100ml)

K_b : 6.2

t : Retention time

Therefore,

$$t = \{(N_i / N_e)^{0.5} - 1\}(1 / K_b)$$

$$t = \{(2.21 \times 10^4 / 1.0 \times 10^2)^{0.5} - 1\}(1/6.20)$$

$$= 2.24\ (2.25)\ \text{day}$$

For the design of maturation pond, total retention time should be more than 5 days. In case of two serial ponds, each retention time is more than 2.5 days.

Depth		1.0 - 1.5 m
Size	First	Width 140 m X Length 200 m X Depth 1.5 m
	Second	Width 140 m X Length 220 m X Depth 1.35 m
Area	First	28,000 m^2
	Second	30,800 m^2
Volume	First	42,000 m^3
	Second	41,580 m^3
Retention time	First	42,000 / 16,256 = 2.58 (2.57) day
	Second	41,580 / 16,256 = 2.56 (2.55) day
Effluent number of coliform group		
	First	1,300 (1,320) /100ml
	Second	77 (79) /100ml < 100/100ml, OK

Total retention time of one train is 12.32 (12.26) days. Dimension of each pond are so determined as to arrange all the ponds in a long rectangle to minimize length of pond connection which in turn minimizes head losses. Arrangement of ponds is shown in Figure G.1.

1.6 Evaporation Loss

Water loss by evaporation can not be neglected since the pond system has a huge water surface area and the effluent is expected to be reused for irrigation purpose. Evaporation losses in the ponds are as follows. The concentrations of inorganic components will be condensed 1.08 of the raw sewage.

Daily average evaporation rate 9.7 mm/d

Water losses		
Anaerobic pond	$11{,}200 \times 9.7 \times 10^{-3}$	= 109 m^3/d
Facultative pond	$50{,}400 \times 9.7 \times 10^{-3}$	= 489 m^3/d
Maturation pond	$58{,}800 \times 9.7 \times 10^{-3}$	= 570 m^3/d
Total		1,168 m^3/d

Condensing factor $16{,}256 / (16{,}256 - 1{,}168) = 1.08$

1.7 Other Facilities

(1) Administration building

The administration building has been designed based on the following assumptions and requirements.

a. Total staff number stationed in the building including administrative, laboratory and operation are 12 persons at full operation of the plant.

b. Administration building consists of following rooms.

- administrative office
- laboratory
- workshop

- store
- worker's room
- small kitchen and toilet

c. Space requirement for administrative office is 10 m^2 per person.

d. Laboratory space of 50 m2 has been provided considering minimum requirement for day to day analysis work and some allowances for extra works.

As a result, spaces to accommodate above mentioned requirements total about 256 m^2. All the building and equipment will be provided in the first phase program.

(1) Inlet works

Inlet works has the following facilities.

- Receiving well
- Manually operated bar screens
- Parshall flumes for flow measurement
- Outlet chamber for even distribution
- Ancillary works

The inlet works is divided into two parallel units, i.e. two sets of bar screens and Parshall flumes. All the civil and architectural works will be completed under the first phase program. Mechanical and electrical equipment will be installed only for one unit.

(3) Inlet and outlet of pond

Inlet and outlet of each pond are provided diagonally to minimize short circuit of flow. Each pond has a outlet chamber for keeping water level by adjustable weirs.

(4) Connection pipe

Pond connection is provided by ductile cast iron pipes with suitable diameters calculated from velocities (see subsection 1.8).

(5) Distribution well

A distribution well is provided for the first phase construction, since two anaerobic ponds are constructed and isolation of one of the ponds is required periodically.

(6) Discharging channel

Discharging channel for effluent disposal to sea will be provided by an open channel. At the crossing of Abyan Road, box culvert will be constructed.

1.8 Hydraulic Calculation

In order to determine the water levels in the ponds, hydraulic calculation has been carried out. Head losses are calculated along the critical passes of flow, i.e. the longest route from the inlet works to the discharging point. Water levels are determined based on the head loss calculation and present ground level of the site as well. The basis and elements of the calculation are described below together with the results.

(1) Sewage Flow Rate

Hydraulic calculation is done based on the peak flow for critical water levels and the daily average flow for representative water levels.

Total	Peak Flow	94,393 m^3/day or 1.093 m^3/sec
	Daily Average	48,769 m^3/day or 0.564 m^3/sec
Per Train	Peak flow	0.364 m^3/sec
	Daily Average	0.188 m^3/sec

(2) Sea Water Level

Water levels at discharging point are determined as follows referring to tide levels at Steamer Point. Highest high water level is used for hydraulic calculation to ensure smooth flow at any time.

Highest High	+ 2.59 m
Mean High	+ 1.93 m

Mean Low + 0.82 m

Lowest Low - 0.25 m

(3) Equations

For the calculation of friction losses and over flow depths at weirs, the following equations are used.

a. Velocity Equation

Manning Formula $$v = \frac{1}{n} R^{2/3} \cdot I^{1/2}$$

where, v : Velocity
n : Roughness coefficient
R : Hydraulic mean depth
I : Hydraulic gradient

b. Critical Depth of Trapezoid Open Channel

$$h_c = \sqrt[3]{\frac{B\,Q^2}{g\,(B - 3h_c)^3}}$$

where, h_c : Critical depth
B : Water surface width
Q : Flow rate
g : Gravity accelation speed

c. Overflow water Depth

Francis formula $$h = \left(\frac{Q}{1.84\,B}\right)^{2/3}$$

where, h : Overflow water depth
Q : Flow rate
B : Length of overflow section

(4) Calculation of Water Level

Results of calculation is shown in Table G.1 and water levels at major points of treatment plant are illustrated in Figure G.2.

1.9 Embankment

The slope gradient of embankment is 1 : 3 on the wet side and 1 : 1.5 on the dry side. Freeboard is more than 0.5 m. Slope protection on the wet side against erosion caused by wind-induced waves is placed from top of the slope to 0.5 m below water surface. Width of the top of embankment is 3 m. Typical cross sections of the embankment are shown Figure G.3. Material of embankment will be transported from other place, since the sandy soil at the site can not be used because of its high permeability. Replacement of pond bottom by clay soil is also designed, which is shown in the same figure.

1.10 Bottom Lining

Facultative and maturation ponds should have the bottom lining for prevention of penetration of pond water into the permeable sandy soils presently exist at the site. Grading of particles sizes are poor, and it is not suitable for construction of the embankment. Lining will be provided by replacing the existing soils with clay soil which is the same material as used for embankment. It is not necessary to provide bottom lining for anaerobic pond, because sludge contained in the raw sewage is easily settled and accumulated on the pond bottom.

If the bottom lining is not provided, penetration of the water may cause adverse effect to the treatment and to the embankment. Penetration of the water into the soil will decrease as the bottom becomes impermeable by solids contained in the water and by biological activities which will take place in the soils. However, cease of the penetration can not be predicted at present. Taking into consideration these factors, bottom lining of 50 cm thickness by clay soil is considered in the design of the facultative and maturation ponds as illustrated in Figure G.3. Bottom lining should be further discussed in the detailed designing.

1.11 Volume of Earth Work

Volume of earth work is calculated according to the classification of nature of works described below.

(1) Land grading work

The existing land will be graded to an uniform level of + 2.9 m for construction, which is equal to the average existing ground level.

(2) Excavation work

Excavation of the present ground is required only for anaerobic pond area because of its depth.

(3) Bottom lining work

Bottom lining will be provided on the bottoms of the facultative and the maturation ponds by the same clay soils as used for embankment.

(4) Embankment work

All materials of embankment will be brought from other place.

(5) Slope grading work

The slope gradient of embankment will be shaped to 1 : 3 on the wet side and to 1 :1.5 on the dry side.

(6) Slope protection work

To prevent erosion caused by wind-induced waves, slope protection should be provided on the wet side of the embankment.

(7) Summary of each earth-volume

Volumes of earth works classified by the above mentioned unit works are summarized below.

Table G.2 Earth-volume

Classification	First Phase	Long term Plan
Land grading (m^2)	134,374	385,389
Excavation (m^3)	25,923	60,897
Bottom lining & Embankment (m^3)	154,938	383,885
Slope grading (m^2)	37,472	89,131
Slope protection (m^2)	11,340	32,718

Table G.1 Hydraulic Calculation (1/2)

Facilities	Items	Unit	Peak	D.Average
Sewage Flow	Total	m3/day	94,393	48,769
	(Three trains)	m3/sec	1.093	0.564
	One train	m3/day	31,464	16,256
		m3/sec	0.364	0.188
Discharge Channel	Water level	m	2.68	2.68
Outlet Chamber of Second Maturation Pond	Pipe diameter	mm	600	600
	Pipe length	m	30	30
	Flow rate	m3/sec	0.364	0.188
	Area	m2	0.283	0.283
	Velocity	m/sec	1.288	0.665
	Friction loss	m	0.106	0.028
	Outlet loss	m	0.085	0.023
	Inlet loss	m	0.042	0.011
	Water level	m	2.91	2.74
Second Maturation Pond	Weir level	m	4.65	4.65
	Weir length	m	6	6
	Flow rate	m3/sec	0.364	0.188
	Overflow W.D.	m	0.103	0.066
	Water level	m	4.75	4.72
Outlet Chamber of First Maturation Pond	Pipe diameter	mm	900	900
	Pipe length	m	20	20
	Flow rate	m3/sec	0.364	0.188
	Area	m2	0.636	0.636
	Velocity	m/sec	0.572	0.296
	Friction loss	m	0.008	0.002
	Outlet loss	m	0.017	0.004
	Inlet loss	m	0.008	0.002
	Water level	m	4.79	4.73
First Maturation Pond	Weir level	m	4.80	4.80
	Weir length	m	6	6
	Flow rate	m3/sec	0.364	0.188
	Overflow W.D.	m	0.103	0.066
	Water level	m	4.90	4.87
Outlet Chamber of Facultative Pond	Pipe diameter	mm	900	900
	Pipe length	m	20	20
	Flow rate	m3/sec	0.364	0.188
	Area	m2	0.636	0.636
	Velocity	m/sec	0.572	0.296
	Friction loss	m	0.008	0.002
	Outlet loss	m	0.017	0.004
	Inlet loss	m	0.008	0.002
	Water level	m	4.94	4.88
Facultative Pond	Weir level	m	4.95	4.95
	Weir length	m	6	6
	Flow rate	m3/sec	0.364	0.188
	Overflow W.D.	m	0.103	0.066
	Water level	m	5.05	5.02
Outlet Chamber of Anaerobic pond	Pipe diameter	mm	900	900
	Pipe length	m	20	20
	Flow rate	m3/sec	0.364	0.188
	Area	m2	0.636	0.636
	Velocity	m/sec	0.572	0.296
	Friction loss	m	0.008	0.002
	Outlet loss	m	0.017	0.004
	Inlet loss	m	0.008	0.002
	Water level	m	5.09	5.03

Table G.1 Hydraulic Calculation (2/2)

Facilities	Items	Unit	Peak	D.Average
Anaerobic Pond	Weir level	m	5.60	5.60
	Weir length	m	6	6
	Flow rate	m3/sec	0.364	0.188
	Overflow W.D.	m	0.103	0.066
	Water level	m	5.70	5.67
Outlet well of Inlet Works	Pipe diameter	mm	900	900
	Pipe length	m	350	350
	Flow rate	m3/sec	0.364	0.188
	Area	m2	0.636	0.636
	Velocity	m/sec	0.572	0.296
	Friction loss	m	0.142	0.038
	90° bend loss	m	0.050	0.013
	Outlet loss	m	0.017	0.004
	Inlet loss	m	0.008	0.002
	Water level	m	5.92	5.72
Outlet of Parchal Flume	Weir level	m	5.95	5.95
	Weir length	m	1.5	1.5
	Flow rate	m3/sec	0.364	0.188
	Overflow W.D.	m	0.259	0.167
	Water level	m	6.21	6.12
Inlet of Parchal Flume	W	cm	45.72	45.72
	Wc	cm	167.6	167.6
	Flow	l/sec	546	282
	K		1.319	1.319
	n		1.539	1.539
	Ha	cm	50.23	32.70
	Water level	m	6.36	6.21
Screen	Velocity	m/sec	0.45	0.45
	Flow rate	m3/sec	0.546	0.282
	Height	m	0.60	0.46
	Width for need	m	2.023	1.378
	Width	m	2	2
	Velocity	m/sec	0.455	0.310
	Loss	m	0.011	0.005
	Water level	m	6.37	6.22
Receiving well	Gate Height	m	0.6	0.6
	Gate Width	m	0.6	0.6
	Gate Area	m2	0.36	0.36
	Velocity	m/sec	1.517	0.784
	Loss	m	0.117	0.031
	Water level	m	6.49	6.25

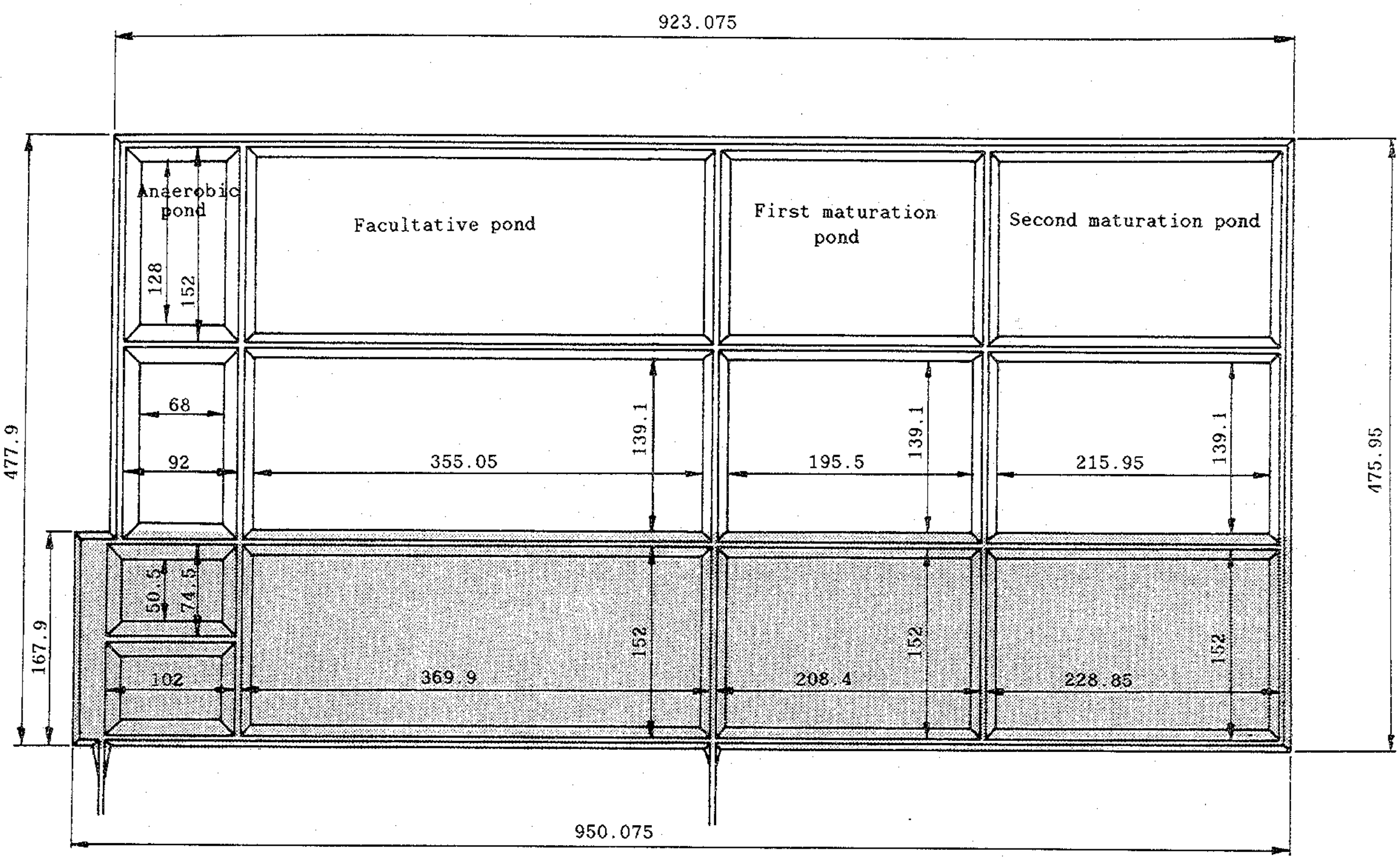


Figure G . 1 Stabilization Pond Layout Plan

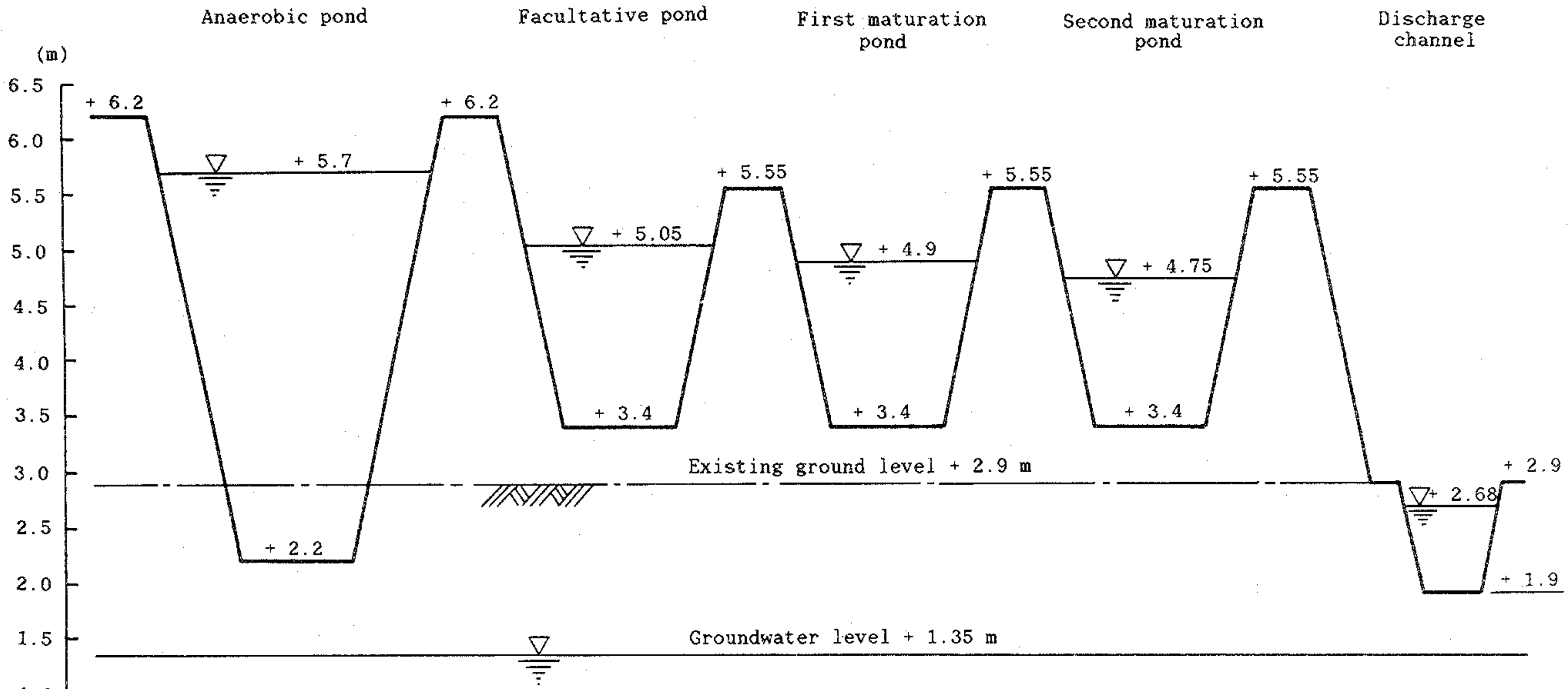
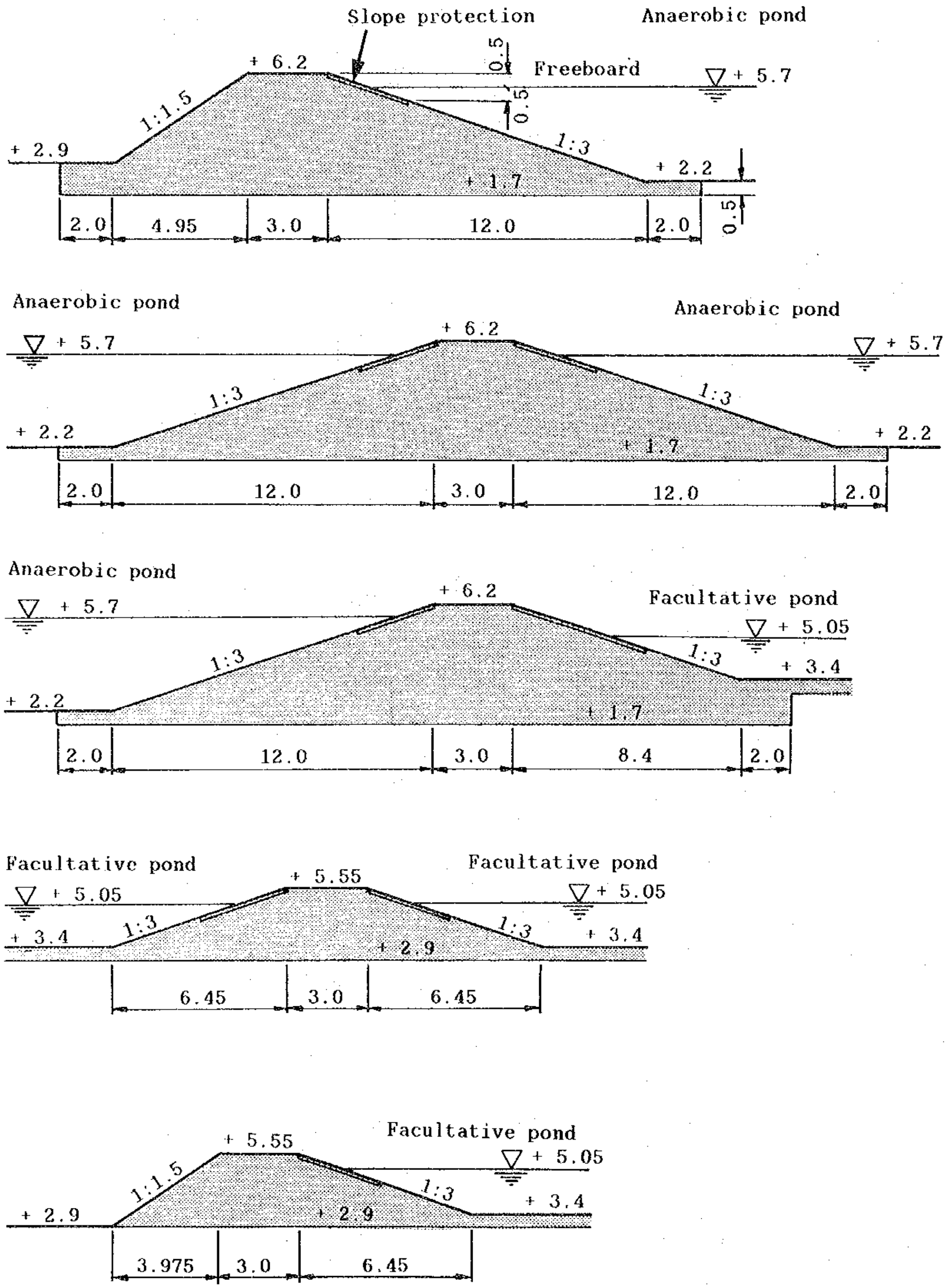


Figure G.2 Water Levels

Figure G.3 Typical Cross Section of The Embankment



2. Design of Major Pumping Stations

Four alternative sewerage systems are considered to select the most appropriate system for the study area as described in Chapter Four of the main report. Design outlines of the force mains and major pumping stations were carried out to estimate construction costs of all alternatives. Main design items such as pump capacities, total head, motor powers, and diameters and lengths of force mains are shown in Tables G.3 to G.6. Calculations of these items were made on an assumption that multi-continuous transfer system is adopted in each alternative.

After the selection of proposed Alternative 3, further consideration was given to compare the two system of pump and force main, viz. single line vs. double line and intermediate system vs. multi-continuous transfer system. Design outlines of pumping stations and force mains of four cases for the comparison of the two systems are shown in Table G.7 to G.10.

Number of alternatives and cases indicated in Tables G.3 through G.10 correspond to those in the main report.

Table G.3 Design Outline of Major Pumping Stations, Alternative 1A

PUMPING STATION		(UNIT)	TAWAHI	MA'ALLA	CRATER	KHORMAKSAR
1) In-flow Sewarage Volume						
In-flow	q=	(m3/s)	0.160	0.312	0.339	0.282
Pump delivery Capacity	q=	(m3/s)	0.160	0.312	0.339	0.282
2) Pipeline Data						
Total Flow Rate	Q=	(m3/s)	0.472	0.312	0.339	0.621
(Khormaksar Branch)	Q=	(m3/s)				(0.288)
Velocity	V=	(m/s)	1.674	1.592	1.202	1.620
(Khormaksar Branch)	V=	(m/s)				(1.785)
Diameter of Pipeline	D=	(mm)	600	500	600	700
(Khormaksar Branch)	D=	(mm)				(450)
Pipe Sectional Area	A=	(m2)	0.282	0.196	0.282	0.384
(Khormaksar Branch)	A=	(m2)				(0.154)
Pipeline Length	L=	(m)	2,000	2,130	4,445	5,195
(Khormaksar Branch)	L=	(m)				(105)
3) Head Loss						
Actual Loss	H1=	(m)	6.44	11.62	5.01	7.41
Loss Head at P/S	H2=	(m)	3.00	3.00	3.00	3.00
Pipeline Loss Head	H3=	(m)	10.71	12.88	12.90	21.81
(Khormaksar Branch)	H3=	(m)				(0.88)
Pressure Head	H4=	(m)	-	10.71	21.81	-
Total Head	H=	(m)	20.15	38.21	42.72	33.10
4) Pump & Motor Data						
Pump No.(1 is stand by)	N=	(set)	3	3	3	3
Required KW / each		(KW)	35	115	145	90
Total KW / P. Station		(KW)	90	250	300	190
5) Diesel Generator Data						
Required KVA		(KVA)	140	280	350	220

Table G.4 Design Outline of Major Pumping Stations, Alternative 1B

	PUMPING STATION	(UNIT)	TAWAHI	MA'ALLA	CRATER	KHORMAKSAR
1)	In-flow Sewarage Volume					
	In-flow	q= (m3/s)	0.160		0.339	0.282
	Pump delivery Capacity	q= (m3/s)	0.160		0.339	0.282
2)	Pipeline Data					
	Total Flow Rate	Q= (m3/s)	0.160		0.339	0.621
	(Khormaksar Branch)	Q= (m3/s)				(0.282)
	Velocity	V= (m/s)	1.667		1.202	1.617
	(Khormaksar Branch)	V= (m/s)				(1.785)
	Diameter of Pipeline	D= (mm)	350		600	700
	(Khormaksar Branch)	D= (mm)				(450)
	Pipe Sectional Area	A= (m2)	0.096		0.282	0.384
	(Khormaksar Branch)	A= (m2)				(0.158)
	Pipeline Length	L= (m)	2,130		4,445	5,195
	(Khormaksar Branch)	L= (m)				(105)
3)	Head Loss					
	Actual Loss	H1= (m)	12.7		5.01	7.41
	Loss Head at P/S	H2= (m)	3.00		3.00	3.00
	Pipeline Loss Head	H3= (m)	21.27		12.90	21.81
	(Khormaksar Branch)	H3= (m)				(0.88)
	Pressure Head	H4= (m)	-		21.81	-
	Total Head	H= (m)	36.99		42.72	33.10
4)	Pump & Motor Data					
	Pump No.(1 is stand by)	N= (set)	3		3	3
	Required KW / each	(KW)	65		145	90
	Total KW / P. Station	(KW)	150		300	220
5)	Diesel Generator Data					
	Required KVA	(KVA)	220		350	230

Table G.5 Design Outline of Major Pumping Stations, Alternative 2

	PUMPING STATION	(UNIT)	TAWAHI	MA'ALLA	CRATER	KHORMAKSAR
1)	In-flow Sewarage Volume					
	In-flow	q= (m3/s)	0.160	0.312	0.339	0.282
	Pump delivery Capacity	q= (m3/s)	0.160	0.312	0.339	0.282
2)	Pipeline Data					
	Total Flow Rate	Q= (m3/s)	0.160	0.472	0.339	0.621
	(Khormaksar Branch)	Q= (m3/s)				(0.282)
	Velocity	V= (m/s)	1.006	1.229	1.202	1.617
	(Khormaksar Branch)	V= (m/s)				(1.785)
	Diameter of Pipeline	D= (mm)	450	700	600	700
	(Khormaksar Branch)	D= (mm)				(450)
	Pipe Sectional Area	A= (m2)	0.159	0.384	0.282	0.384
	(Khormaksar Branch)	A= (m2)				(0.158)
	Pipeline Length	L= (m)	2,130	15,800	4,445	5,195
	(Khormaksar Branch)	L= (m)				(105)
3)	Head Loss					
	Actual Loss	H1= (m)	14.9	13.8	5.01	7.41
	Loss Head at P/S	H2= (m)	3.00	3.00	3.00	3.00
	Pipeline Loss Head	H3= (m)	6.26	39.93	12.90	21.81
	(Khormaksar Branch)	H3= (m)				(0.88)
	Pressure Head	H4= (m)	39.93	-	21.81	-
	Total Head	H= (m)	64.09	56.73	42.72	33.1
4)	Pump & Motor Data					
	Pump No.(1 is stand by)	N= (set)	3	3	3	3
	Required KW / each	(KW)	100	175	145	90
	Total KW / P. Station	(KW)	210	360	300	200
5)	Diesel Generator Data					
	Required KVA	(KVA)	250	410	350	230

Table G.6 Design Outline of Major Pumping Stations, Alternative 3

PUMPING STATION		(UNIT)	TAWAHI	MA'ALLA	CRATER	KHORMAKSAR
1) In-flow Sewarage Volume						
In-flow	q=	(m3/s)	0.160	0.312	0.339	0.282
Pump delivery Capacity	q=	(m3/s)	0.160	0.312	0.339	0.282
2) Pipeline Data						
Total Flow Rate	Q=	(m3/s)	0.160	0.472	0.339	0.621
(Khormaksar Branch)	Q=	(m3/s)				(0.282)
Velocity	V=	(m/s)	1.270	1.230	1.200	1.620
(Khormaksar Branch)	V=	(m/s)				(1.780)
Diameter of Pipeline	D=	(mm)	400	700	600	700
(Khormaksar Branch)	D=	(mm)				(450)
Pipe Sectional Area	A=	(m2)	0.126	0.384	0.282	0.384
(Khormaksar Branch)	A=	(m2)				(0..158)
Pipeline Length	L=	(m)	2,130	10,960	4,445	5,195
(Khormaksar Branch)	L=	(m)				(105)
3) Head Loss						
Actual Loss	H1=	(m)	10.51	9.41	5.01	7.41
Loss Head at P/S	H2=	(m)	3.00	3.00	3.00	3.00
Pipeline Loss Head	H3=	(m)	11.10	27.70	12.90	21.81
(Khormaksar Branch)	H3=	(m)				(0.88)
Pressure Head	H4=	(m)	27.70	-	21.81	-
Total Head	H=	(m)	52.31	40.11	42.72	33.10
4) Pump & Motor Data						
Pump No.(1 is stand by)	N=	(set)	3	3	3	3
Required KW / each		(KW)	90	120	140	90
Total KW / P. Station		(KW)	210	250	300	210
5) Diesel Generator Data						
Required KVA		(KVA)	300	360	430	300

Table G.7 Design Outline of Pump and Force Main System, Case-1

	PUMPING STATION	(UNIT)	TAWAHI	MA'ALLA	CRATER	KHORMAKSAR
1)	In-flow Sewarage Volume					
	In-flow	q= (m3/s)	0.160	0.312	0.339	0.282
	Pump delivery Capacity	q= (m3/s)	0.160	0.312	0.339	0.282
2)	Pipeline Data					
	Total Flow Rate	Q= (m3/s)	0.160	0.472	0.339	1.093
	(Khormaksar Branch)	Q= (m3/s)				(0.282)
	Velocity	V= (m/s)	1.031	1.229	1.202	1.721
	(Khormaksar Branch)	V= (m/s)				(1.785)
	Diameter of Pipeline	D= (mm)	450	700	600	900
	(Khormaksar Branch)	D= (mm)				(450)
	Pipe Sectional Area	A= (m2)	0.158	0.384	0.282	0.635
	(Khormaksar Branch)	A= (m2)				(0.158)
	Pipeline Length	L= (m)	2,130	5,765	4,445	5,195
	(Khormaksar Branch)	L= (m)				(105)
3)	Head Loss					
	Actual Loss	H1= (m)	10.51	9.41	5.01	7.41
	Loss Head at P/S	H2= (m)	3.00	3.00	3.00	3.00
	Pipeline Loss Head	H3= (m)	6.26	14.56	12.90	18.25
	(Khormaksar Branch)	H3= (m)				(0.88)
	Pressure Head	H4= (m)	32.81	18.25	18.25	-
	Total Head	H= (m)	52.58	45.22	39.16	29.54
4)	Pump & Motor Data					
	Pump No.(1 is stand by)	N= (set)	3	3	3	3
	Required KW / each	(KW)	75	210	130	85
	Total KW / P. Station	(KW)	175	430	270	180
5)	Diesel Generator Data					
	Required KVA	(KVA)	250	490	310	210

Table G.8 Design Outline of Pump and Force Main System, Case-2

	PUMPING STATION	(UNIT)	TAWAHI	MA'ALLA	CRATER	KHORMAKSAR
1)	In-flow Sewarage Volume					
	In-flow	q= (m3/s)	0.160	0.312	0.339	0.282
	Pump delivery Capacity	q= (m3/s)	0.160	0.472	0.339	1.093
2)	Pipeline Data					
	Total Flow Rate	Q= (m3/s)	0.160	0.472	0.339	1.093
	(Khormaksar Branch)	Q= (m3/s)				
	Velocity	V= (m/s)	1.667	1.674	1.730	1.721
	(Khormaksar Branch)	V= (m/s)				
	Diameter of Pipeline	D= (mm)	350	600	500	900
	(Khormaksar Branch)	D= (mm)				
	Pipe Sectional Area	A= (m2)	0.096	0.282	0.196	0.635
	(Khormaksar Branch)	A= (m2)				
	Pipeline Length	L= (m)	2,130	5,765	4,445	5,195
	(Khormaksar Branch)	L= (m)				
3)	Head Loss					
	Actual Loss	H1= (m)	10.51	9.41	5.01	7.41
	Loss Head at P/S	H2= (m)	3.00	3.00	3.00	3.00
	Pipeline Loss Head	H3= (m)	21.27	30.86	31.35	18.25
	(Khormaksar Branch)	H3= (m)				
	Pressure Head	H4= (m)	-	-	-	-
	Total Head	H= (m)	34.78	43.27	39.36	28.66
4)	Pump & Motor Data					
	Pump No. (1 is stand by)	N= (set)	3	3	3	3
	Required KW / each	(KW)	65	200	130	310
	Total KW / P. Station	(KW)	150	410	170	630
5)	Diesel Generator Data					
	Required KVA	(KVA)	220	470	310	720

Table G.9 Design Outline of Pump and Force Main System, Case-3

PUMPING STATION		(UNIT)	TAWAHI	MA'ALLA	CRATER	KHORMAKSAR
1) In-flow Sewarage Volume						
In-flow	q=	(m3/s)	0.160	0.312	0.339	0.282
Pump delivery Capacity	q=	(m3/s)	0.160	0.312	0.339	0.282
2) Pipeline Data						
Total Flow Rate	Q=	(m3/s)	0.160	0.472	0.339	0.621
(Khormaksar Branch)	Q=	(m3/s)				(0.282)
Velocity	V=	(m/s)	1.270	1.230	1.200	1.620
(Khormaksar Branch)	V=	(m/s)				(1.780)
Diameter of Pipeline	D=	(mm)	400	700	600	700
(Khormaksar Branch)	D=	(mm)				(450)
Pipe Sectional Area	A=	(m2)	0.126	0.384	0.282	0.384
(Khormaksar Branch)	A=	(m2)				(0..158)
Pipeline Length	L=	(m)	2,130	10,960	4,445	5,195
(Khormaksar Branch)	L=	(m)				(105)
3) Head Loss						
Actual Loss	H1=	(m)	10.51	9.41	5.01	7.41
Loss Head at P/S	H2=	(m)	3.00	3.00	3.00	3.00
Pipeline Loss Head	H3=	(m)	11.10	27.70	12.90	21.81
(Khormaksar Branch)	H3=	(m)				(0.88)
Pressure Head	H4=	(m)	27.70	-	21.81	-
Total Head	H=	(m)	52.31	40.11	42.72	33.10
4) Pump & Motor Data						
Pump No.(1 is stand by)	N=	(set)	3	3	3	3
Required KW / each		(KW)	90	120	140	90
Total KW / P. Station		(KW)	210	250	300	210
5) Diesel Generator Data						
Required KVA		(KVA)	300	360	430	300

Table G.10 Design Outline of Pump and Force Main System, Case-4

	PUMPING STATION	(UNIT)	TAWAHI	MA'ALLA	CRATER	KHORMAKSAR
1)	In-flow Sewarage Volume					
	In-flow	q= (m3/s)	0.160	0.312	0.339	0.282
	Pump delivery Capacity	q= (m3/s)	0.160	0.472	0.339	0.621
2)	Pipeline Data					
	Total Flow Rate	Q= (m3/s)	0.160	0.472	0.339	0.621
	(Khormaksar Branch)	Q= (m3/s)				
	Velocity	V= (m/s)	1.667	1.230	1.730	1.620
	(Khormaksar Branch)	V= (m/s)				
	Diameter of Pipeline	D= (mm)	350	700	500	700
	(Khormaksar Branch)	D= (mm)				
	Pipe Sectional Area	A= (m2)	0.096	0.384	0.196	0.384
	(Khormaksar Branch)	A= (m2)				
	Pipeline Length	L= (m)	2,130	10,960	4,445	5,195
	(Khormaksar Branch)	L= (m)				
3)	Head Loss					
	Actual Loss	H1= (m)	10.51	9.41	5.01	7.41
	Loss Head at P/S	H2= (m)	3.00	3.00	3.00	3.00
	Pipeline Loss Head	H3= (m)	21.27	27.70	31.35	21.81
	(Khormaksar Branch)	H3= (m)				
	Pressure Head	H4= (m)	-	-	-	-
	Total Head	H= (m)	34.78	40.11	39.36	32.22
4)	Pump & Motor Data					
	Pump No.(1 is stand by)	N= (set)	3	3	3	3
	Required KW / each	(KW)	65	190	130	200
	Total KW / P. Station	(KW)	150	390	270	410
5)	Diesel Generator Data					
	Required KVA	(KVA)	220	440	310	470

3. Rehabilitation of the Existing Pumping Stations

A total of 27 existing small pumping stations are proposed to be rehabilitated. Pumping equipment used in these pumping stations are tabulated in Appendix C, and are classified into two types of pump, viz. centrifugal type and ejector. For the replacement of these pumps, submersible type of pump is proposed because of i) small sewage flow, ii) minimum space requirement and iii) ease of operation and maintenance. Considerations on submersible pump for the rehabilitation of the existing pumping stations are described in Appendix H.

All the pumping stations to be rehabilitated are classified into the five categories shown in Table G.11, according to sewage flows and pump heads. Table G.12 shows number of units, motor powers, capacity of pumps and categories of all the pumping station. Typical installation of the pumping station and dimensions of pump wells of each category are shown in Figure G.4.

Conditions of the buildings are also investigated and indicated in Table G.12. All the pumping stations as classified "No" should be re-built by 2010.

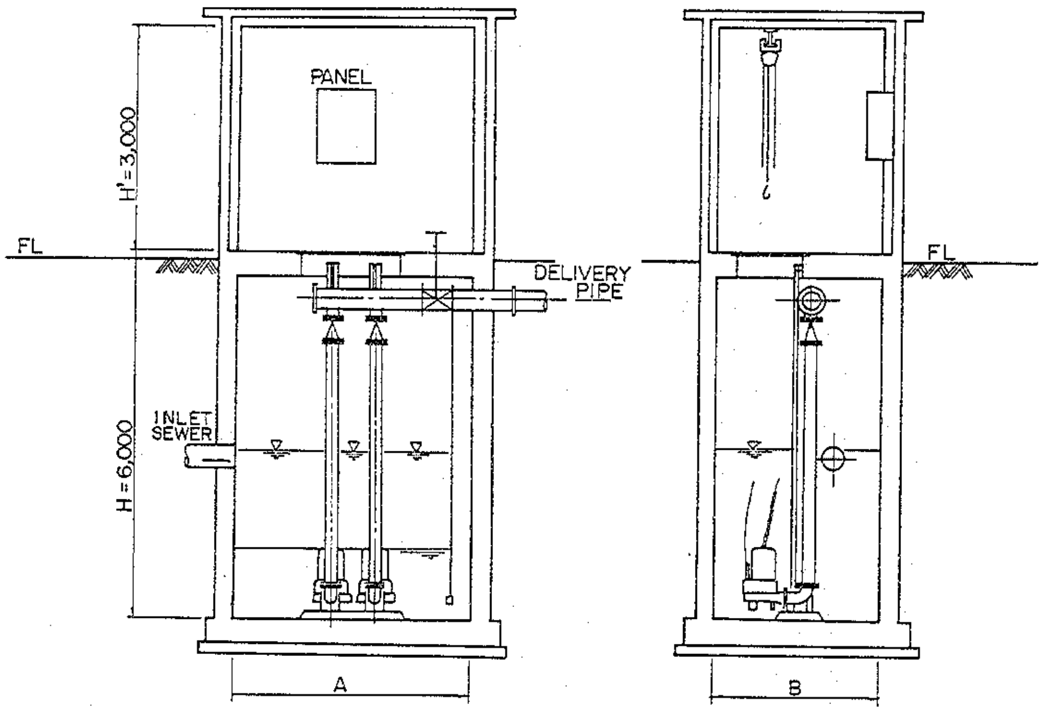
Table G.11 Category of Pumping Stations

	Type of Pump	Motor Power (kw)	Diameter (mm)
A A'	Submersible Pump	3.7	80
B B'	Submersible Pump	2.2	80
C C'	Submersible Pump	5.5	100
D	Submersible Pump	11.0	150
E'	Submersible Pump	15.0	150

Note: Number of pump units are as follows
A,B,C and D : 2 (1 Standby)
A',B',C' and E': 3 (1 standby)

Table G.12 Design Outline of Intermediate Pumping Stations to be Rehabilitated

		Planning		Condition of Old Facilities		
District	Number	Motor Power (kw X Nos.)	Capacity (dia.X m3/min)	Equipment	Building	Category
Ma'alla	102	7 X 3	150 X 3.0	NO	OK	E'
	103	7 X 2	100 X 1.0	NO	NO	C
	104	5 X 2	100 X 0.5	NO	NO	A
Tawahi	201	3.5 X 2	100 X 0.5	NO	NO	A
	202	3.5 X 2	100 X 1.0	NO	NO	C
	203	2 X 2	100 X 0.1	NO	NO	B
	204	2 X 2	100 X 0.1	NO	NO	B
	205	3.5 X 2	100 X 0.5	NO	OK	A
	206	2 X 2	100 X 0.1	NO	NO	B
	207	3.5 X 2	100 X 0.1	NO	NO	B
Crater	302	11 X 2	100 X 0.1	NO	NO	B
Khormaksar	401	5 X 3	150 X 3.0	NO	NO	E'
	402	11 X 3	150 X 0.5	NO	NO	A'
	403	11 X 2	100 X 0.1	5 years	OK	B
	404	11 X 3	100 X 0.1	NO	NO	B'
	405	7 X 3	150 X 1.0	NO	NO	C'
	406	5 X 2	150 X 2.0	NO	OK	D
	407	11 X 3	150 X 3.0	NO	OK	E'
	408	5 X 2	100 X 0.1	NO	OK	B
	409	7 X 2	150 X 0.1	NO	OK	B
	410	2 X 3	150 X 0.1	NO	NO	B'
	411	2 X 3	150 X 0.1	NO	NO	B'
	412	11 X 2	100 X 0.1	NO	OK	B
	413	11 X 2	100 X 0.1	NO	NO	B
	414	11 X 2	100 X 0.1	NO	NO	B
	416	2 X 2	100 X 1.0	NO	OK	C
	417	11 X 2	100 X 0.1	NO	NO	B



CATEGORY	WIDTH:A(m)	LENGTH:B(m)
Ⓐ	3.50	2.50
Ⓐ'	4.00	2.50
Ⓑ	3.50	2.50
Ⓑ'	4.00	2.50
Ⓒ	3.50	2.50
Ⓒ'	4.00	2.50
Ⓓ	3.50	2.50
Ⓔ'	4.00	2.50

Figure G.4 TYPICAL SUBMERSIBLE PUMPING STATION

(SCALE : NONE)

APPENDIX H

SUBMERSIBLE PUMP FOR REHABILITATION

APPENDIX H

SUBMERSIBLE PUMP FOR REHABILITATION

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APPENDIX H

SUBMERSIBLE PUMP FOR REHABILITATION

1. Introduction

As described in the Minutes of the Technical Meeting held on 2nd July, question was expressed by Aden Municipality against use of submersible pumps recommended for the rehabilitation of the existing pumping stations. The reason is that they have had many problems with this type of pump. According to their experience, submersible pumps had been damaged by many causes, such as leakage of water into motor section and corrosion of component parts, particularly impellers.

To observe actual situation, we carried out a field investigation of this type of pump on several places as suggested by the Municipality. Based on the results of the investigation, causes of damages and measures against causes of damages were analyzed. This appendix describes the situation of the submersible pumps investigated, possible measures and our recommendation.

2. Field Investigation

A field investigation was carried out at the following five places.

(1) Crater Office: Damaged pumps from military camp
(2) General Hospital P/S in Khormaksar
(3) Airport P/S
(4) Al Shaab STP, inlet work
(5) Little Aden P/S

Conditions of submersible pumps at each place are described below.

(1) Crater Office

Broken submersible pump units removed from the eight pumping stations in Khormaksar military camps were brought to this office. A total of 16 pump units, two units per P/S, were installed in 1982. All the pump units became out of order after 5 or 6 years of operation. These pump units were made in Holland.

They showed us several heavily damaged impellers taken from broken pumps. Impellars are corroded and their shapes are deformed, suggesting strong impacts by fairly large size solid materials. Three pump units which are said to be broken by water leakage into motor section are stored there. No serious corrosion nor deformation of impeller was recognized in these three pump units.

Operation and maintenance conditions of these pumps, when they were operable are not known, since these duties were under the responsibility of military authority. Aden Municipality was called by the military authority for investigation and repairing whenever a trouble happened.

(2) General Hospital P/S in Khormaksar

Initially, this pumping station was designed by JTS (John Taylor and Sons Consulting Civil Engineers), and commissioned in 1954. In those days, centrifugal type pumps installed in dry pit were used up to 1982. Then this pumping station was rehabilitated by French people and Finland made submersible pumps were installed. Operation of these pumps started in 1984, but pumps became out of order in 1989 after approximately five years of operation. Main cause of failure is said to be burn out of motor coils. The operation and maintenance of the P/S has been under the responsibility of the hospital. Aden Municipality has been called when a trouble occurred.

(3) Airport P/S

The operation started with two submersible pump units in 1984. These pump units were made in France, and diameters of pump are 100 mm. Operation of pumps is controlled automatically by water levels in pump pit. It is said that as far as this pumping station is concerned, there has been no problem except for minor failures of electric parts since operation started. Airport authority is in charge of this pumping station. When trouble happens, they would report to Aden Municipality and its staff is requested to carry out trouble-shooting.

(4) Al Shaab STP

Two submersible pump units made in Japan were installed in the head of inlet work in 1986. These pumps have been used everyday to clean screen channels in the inlet work. Initially, these pumps were designed to flush screenings from screen to comminuters. However, they stopped using comminuters because of heavy brockage problem. Screenings, mostly cans, bottles and vinyl materials are removed from screens and disposed of separately. Two pumps are in fairly good condition. Impellers of the pumps were damaged and changed twice in five years.

(5) Little Aden P/S

This pumping station started operation 35 years ago. Centrifugal type pumps installed in dry pit had been used until 1987. Then, old pumps were replaced by two new submersible pumps made in Japan. These pumps are automatically operated by water levels in the wet well. No troubles have been reported so far.

3. Causes of Damage

From the discussions with Aden Municipality engineers and technicians, and observation at sites, we consider that the followings are possible causes of breakdown.

(1) Water infiltration into motor section

Inadequate sealing or unproper handling of a pump unit causes infiltration of water into motor section, resulting in breakdown of a motor.

(2) Ingress of solid material

A number of impeller were heavily damaged and shapes were deformed. This kind of heavy damage could never be caused by anything except for strong impact by fairly large solid materials. No protective measure against ingress of such materials might have been provided.

(3) High acidity of sewage

Corrosion of impellers and other parts of pump are recognized, although it is not so serious as to cause breakdown of a pump unit. It is said that high acidity of sewage causes corrosion of materials.

(4) Insufficient maintenance

Although maintenance carried out by the authorities in charge of the pumping stations are not known, a fact that when pumps are in trouble, staff of Aden Municipality is always called on to shoot troubles, suggests their lack of knowledge and skill to maintain pumps properly.

(5) Obsolete model

Sixteen pumps used in the military area are obsolete model. It can not be lifted to the floor without a person going down to pump level to dismount pump from a suction pipe. This makes routine maintenance work very troublesome.

4. Protective Measures

Various protective measures against causes of damage mentioned above are presently available. These are described below.

(1) Corrosion resistant materials

At present anti-corrosion types of materials are used for the parts of the pump which directly contact to sewage, such as casing, impeller, and fittings. They are durable for their life time usually expected in normal sewage water. If the sewage is strongly aggressive, a careful attention should be paid for the selection of materials. Since submersible pumps are broadly used, highly anti-corrosion materials for specific parts are readily available.

(2) Water tightness

Motor section of a submersible pump recently manufactured are completely sealed against infiltration of water. Infiltration problem never occurs as long as a pump is properly operated.

(3) Screening bucket

In order to prevent ingress of solid materials into a pump, measures to remove them shall be provided. A screening bucket at the inlet point of sewer is recommended for small capacity pumping stations. This method is inexpensive, yet effective measures for removal of solid materials. Screening bucket shall be cleaned periodically, and screenings shall be disposed of properly. A typical installation of screening bucket is shown in Figure H.1.

(4) Easy dismounting

For the routine maintenance, submersible pumps are to be dismounted from the operating position and lifted to the floor. It is desirable that this can be done without a person going down to a pump. Recently most of the submersible pumps are provided with devices for this purpose. Figure H.2 shows a typical method of dismounting of a submersible pump with a guide pipe and chains.

(5) Proper maintenance

In view of the existing problems reported, the basic causes, we believe, are clearly a lack of regular monitoring and periodical maintenance services. It has no direct relation with the type of the pump. No pumps can work for their life time expected without proper maintenance. It is strongly recommended, therefore, that the regular inspection services for the existing pumping stations should be established.

5. Recommendation

One of the advantages of a submersible pump is low cost of underground structure. Construction cost of underground structure is obviously lower than that for other type of pumps, since dry pit is not required for submersible pumps. Cost for supply and installation is comparable with or slightly lower than that for other type of pump. Thus, total construction cost is lower than that of other type pumping stations.

All the existing pumping stations under the responsibility of Aden Municipality have been properly maintained by its staff. No special skill or advanced technique is required for the maintenance of submersible pump. Present staff in Aden Municipality can easily handle the submersible pumps provided that a minimum guidance is given to them.

In due consideration of the problems encountered, counter measures available at present, and current level of maintenance services, we recommend submersible pumps for the rehabilitation of the existing pumping stations in the study area.

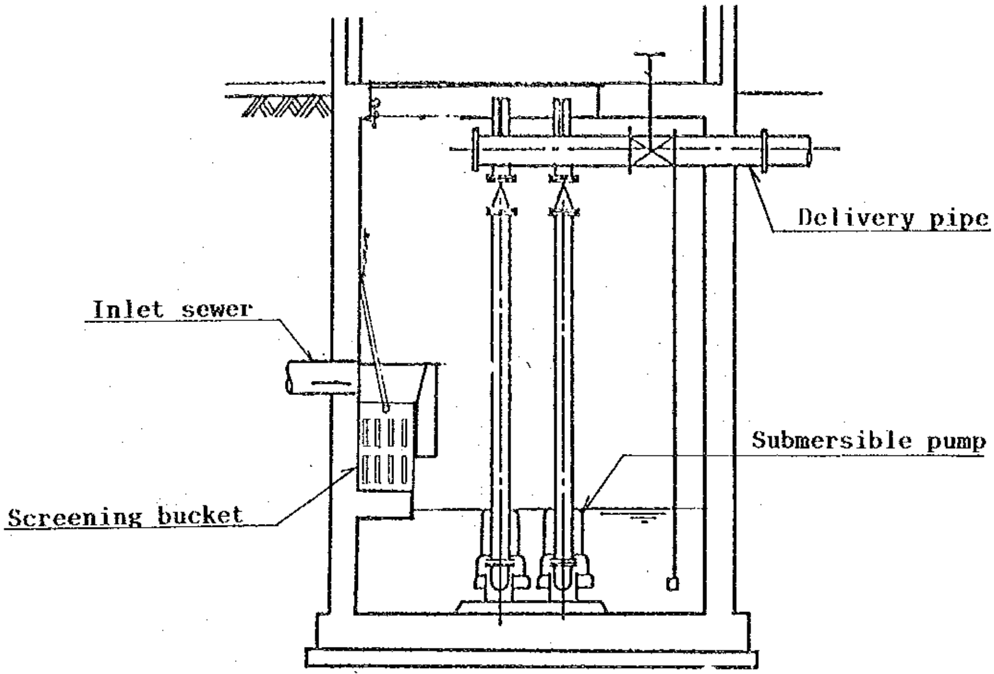

Figure H.1 Screening Bucket

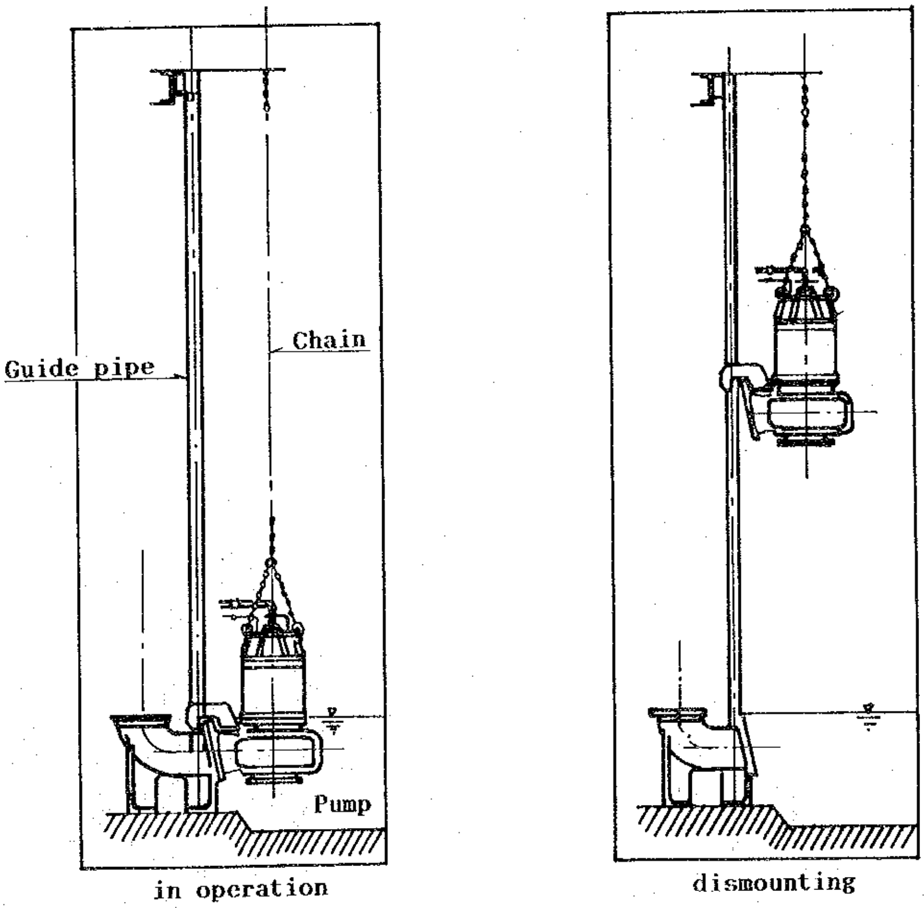

Figure H.2 Dismounting of Submersible Pump

APPENDIX I

INVESTIGATION OF THE EXISTING SYSTEM IN KHORMAKSAR

APPENDIX I

INVESTIGATION OF THE EXISTING SYSTEM IN KHORMAKSAR

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APPENDIX I

INVESTIGATION OF THE EXISTING SYSTEM IN KHORMAKSAR

1. Introduction

Khormaksar district is situated on flat and low-lying area, where the soils are sandy and groundwater levels are generally high. Because of the topographic condition of the district, sewerage service area is divided into a number of small zones each served by one or more pumping stations. Over 30 small pumping stations exist in the district.

In addition to the nature of the sewerage system in the district, high temperature and high organic substances in raw sewage provide very favorable atmosphere for hydrogen sulphide gas generation. Moreover, most of the sewer pipes in the district are made of asbestos cement. Consequently, damages caused by hydrogen sulphide gas attack are most serious which frequently results in the collapses of pipes.

The capacities of the existing sewer pipes have been found to be sufficient for the flow in 2010 by hydraulic calculation (Ref. to Appendix F). Present conditions of the pumping stations were investigated during the first on-site work (Ref. to. Appendix C).

Under the circumstances mentioned above, investigation of the existing system has been focused on the measurement of hydrogen sulphide gas on certain critical points. Based on the results of the measurement, an attempt was made to estimate the degree of damages in the future, which, in turn, necessitates the replacement of pipes.

This appendix describes the method and results of the investigation. Estimation of the necessity of pipe replacement is described in Section 5.4 of the current report.

2. Selection of Measuring Points

Structure of the sewerage system in the district which is composed by pumping stations, force mains and trunk gravity sewers, is illustrated diagrammatically in Figure I.1.

Distribution of sewer pipe collapse reported by Aden Municipality are shown in Figure I.2. As shown in the figure, collapses of pipes are concentrated in specific zones, viz. zones B, C, E, G, H, J and M. There are few reported break down of pipes in zones A, K and L.

Causes of collapse of sewer pipe with or without hydrogen sulphide gas are, in general, considered as follows.

(1) Hydrogen sulphide gas

a. Downstream of force main. Septicity of the sewage is promoted in force main, especially in case of long flowing time. Since generation of sulfuric acid requires the presence of oxygen, the collapse dose not necessarily occur at the immediate downstream of force main, but after some distance.

b. Gravity sewers in specific condition. Accumulation of sludge caused by insufficient velocity or any other reasons promotes the septicity of the sewage. Thus, the downstream section of these pipes or septic tanks is vulnerable to sulphide gas attack.

(2) Without hydrogen sulphide gas

c. Improper construction, in particular inadequate back filling.

d. Excessive load on pipes, live or dead load, in particular shallow pipes.

e. Other reasons, e.g. intrusion of root of tree. This was actually observed in the district

Taking into account the distribution and causes of the collapses, 30 points were selected for hydrogen sulphide gas measurement as shown in Figure I.3. These points were selected to include the followings.

(1) Downstream points of force main
(J0, J1, E1, E4, C2, B3, L2, M5, Comminuter Station)

(2) Near points of collapses
(H0, H1, J2, J3, E2, C1, G1, G2, K1', B1, B2, M1, M2, M3, T1)

(3) Corresponding points to (1) and (2) above in the zones where no collapse was reported.
(G3', B4, L1, M4)

3. Results of the Investigation

The results of the measurement are tabulated in Table I.1. Details of the results are described below.

Since conditions of the damages are different from zone to zone, description was made by zones from south to north. Sketches of sewerage system in all zones are shown in Figures I.4 through I.12.

(1) Zone H

Zone H is located in the southern part of the Khormaksar district. All the sewages in this zone flow into P/S H, then sent to the neighboring zone J.

Many collapses of sewer pipes were reported in the past, and these are concentrated in south-eastern part of the zone. Replacement of the broken pipes was carried out by the Municipality and some routes of the sewers were changed by the replacement.

No collapse has been reported in the north-eastern part. A few collapses have been reported in the sewer lines from northern part to P/S H, and lines from west.

Total length of sewer lines on which collapses were reported accounts for relatively high percentage of 28 % of the total length of sewers in the zone.

Hydrogen sulphide gas concentration in manhole H-1 was 40 mg/l. However, the concentration increased to 75 mg/l when sediments on the bottom of the manhole were stirred. Hydrogen sulphide gas concentration in manhole H-0 was measured to be 10 mg/l when the manhole was full of stagnated sewage. These two manholes are not functioning properly because of the collapses ahead and after these manholes. Thus concentrations might have been higher than measured before collapse.

(2) Zone J

Zone J is bounded by hospital area on the north and by a military area and a milk factory area on the west. All the sewages in the zone flow into P/S J and delivered to the comminuter station by force mains. Manhole J-1 receives the sewages from P/S H and a milk factory both of which are located outside the zone. Manhole J-3 receives wastewater from the hospital.

Many collapses were reported in this zone, and collapses are concentrated in the northern part of the zone. Collapses were also reported in the hospital area.

Total length of the sewer lines on which collapses occurred accounts for the second highest percentage of 37 % of the total sewers.

Hydrogen sulphide gas concentration in manhole J-1 was measured to be 25 mg/l. Concentration in manhole J-2 was as low as 23 m g/l. These figures are low in spite of the location of the manhole that they are located at the terminal point of the force main. The reasons for the low figures are that a large quantity of fresh wastewater from a milk factory flows frequently in this section, and pumps were not operated at the time of measurement.

A low concentration of 4 mg/l was recorded in manhole J-0. Collapses in upstream section of this point, and subsequent improper functioning of the sewer are the reasons for the low concentration.

Hydrogen sulphide gas concentration in manhole J-3 was 35 mg/l. This is considered to be low as the terminal point of the force main. There are collapses of sewers in the upstream section of this point, which may be the reason for low concentration.

In spite of the low concentrations measured at manholes, heavy corrosion was observed in the walls and the covers of each manhole suggesting high concentration of hydrogen sulphide gas in the past.

(3) Zone E

Zone E is situated in the central part of the district, and surrounded by many zones. All the sewages in the zone flow into P/S E and pumped to the E-1 manhole then gravitate to the neighboring zone G.

Sewages from outside areas, viz. Tareq Police and Traffic P/S, are delivered to manhole E-4 in the zone. Sewages from Market area also flow into this zone.

Collapses were reported on the sewer lines in the south of the zone. Total length of sewer lines of collapse accounts for 11 % of the total length of sewers.

Manhole E-1 is the terminal point of the force main from P/S E, and is located in downstream section of Tareq military P/S. Hydrogen sulphide gas concentration in manhole E-1 recorded the highest figure of 500 mg/l. This concentration would be higher if the P/S E was in operation. Sewage flow was very low, depth of water in the sewer of 30 cm diameter was about 8 cm and velocity was about 7 cm/s by observation at the time of measurement. Heavy corrosion was recognized at the inside of the manhole.

Sewage stagnated and was almost full in manhole E-2 because of the collapse of downstream sewer near P/S E. A tank lorry with a vacuum pump comes everyday to remove the sewage. Because of the present condition of sewers, hydrogen sulphide gas concentration in manhole E-2 was as low as 7 mg/l.

Manhole E-3 is not used at present, because of the change of the sewer rout resulting from the replacement of the broken pipe.

In manhole E-4, concentration of 2 mg/l by the initial measurement increased to 150 mg/l when Traffic P/S started. This figure may increase further if Tareq P/S is in operation at the same time.

Main reasons for the collapses reported in the past between E-2 and E-3 were the location of the sewers that they were situated in the downstream section of Traffic P/S and Tareq P/S. Newly replaced pipes are subject to damage by hydrogen sulphide gas attack.

High concentration of hydrogen sulphide gas due to insufficient velocity seemed to cause the collapse of pipe located immediately north to manhole E-2.

(4) Zone C

Zone C is located in the center of the district, and surrounded by many zones. All the sewages in the zone gravitate into P/S C and are pumped directly to the comminuter station. Manhole C-2 is the terminal point of the force main from the P/S B and receives the sewage from zone B.

Collapses are concentrated in the south part of the zone near manhole C-1. There is no collapses in the remaining area. Total length of sewers of collapse accounts for 14 % of the total sewer length.

Hydrogen sulphide gas concentration in manhole C-1 is as low as 4 mg/l, since most of the flow seemed to be groundwater infiltrated from broken point at immediate upstream section.

Concentration in manhole C-2 is also as low as 17 mg/l instead of the terminal point of the force main. Little flow was observed at the time of measurement, and pumps in P/S B could not be started because of the low water level in pump pit. Higher concentration should have been measured if pumps were in operation.

Manhole C-2 and the force main are new, since the old route of force main from P/S B was changed because of clogging.

(5) Zone G

Zone G is located at east side of the central part of the district, and surrounded by many zones. Sewages in the zone gravitate into P/S G, and pumped directly to the comminuter station. Manhole E-1 receives the sewages from zone E, the military area in the north and Al Ghomhoria Hospital area. Sewage from a part of Gamal area flows to manhole G-2.

Collapses of pipes are concentrated on the lines in the west part of the zone where sewages from the outside area flow in. Total length of sewers of collapse account for 22 % of the total length of sewers.

Hydrogen sulphide gas concentration in manhole G-1 was 41 mg/l, which seemed to be low taking into account the fact that this point is located in the downstream section of manhole E-1 where the highest concentration of 500 mg/l was recorded. This low concentration in manhole G-1 is due to the good ventilation through the wet well of the pumping station located immediately down to this point.

Manhole G-2 is located in the upstream section of a collapse, and sewage from a part of outside Gamal area flows into the manhole. However, hydrogen sulphide gas concentration at this point was as low as 17 mg/l. No sewage flow was observed at the time of measurement, and sewage stagnated in the manhole. Higher concentration should have been recorded if sewage flowed.

Manhole G-3 is located on the line which flow to P/S G from the east. Measurement could not be done because manhole cover could not be opened. Soils around the manhole changed its color to yellow because of the sulphur from the manhole. The same condition was recognized at several points in Ma'alla. Hydrogen sulphide gas concentrations were high and heavy corrosion was observed in these manholes. Thus, concentration in manhole G-3 is supposed to be high.

Hydrogen sulphide gas concentration in manhole G-3' was 63 mg/l. This manhole is located in the downstream section of manhole G-3. The reason for relatively low concentration is good ventilation through pump well as observed at manhole G-1.

From the measurement and observation, hydrogen sulphide gas concentration in the upstream section of manholes G-3 and G-3L is expected to be high.

(6) Zone K

Zone K is located at the east side of the central part of the district. All the sewages in the zone flow into P/S K, and pumped directly to the comminuter station. No sewage flows to this zone from outside.

Only one collapse was reported in the downstream section of manhole K-1'. Total length of the sewers of collapse accounts for 12 % of the total sewer length. Hydrogen sulphide gas concentration in manhole K-1' was very low, only 1 mg/l.

(7) Zone B

Zone B is located in the central part of the district, and surrounded by many zones. Sewages in the zone gravitate into P/S B and delivered to manhole C-2 in zone C. Manhole B-3 receives the sewage from P/S A in the military area. Sewages from Gamal and MOC areas flow into P/S B.

There are many collapses reported in the zone. All of the downstream sections of manhole B-3 which is located at the terminal point of the force main from northeast side to P/S B, are collapsed. There are also many collapses on the lines from manhole B-1. Total length of collapsed sections accounts for the highest percentage of 44 % of the total length of sewers.

High hydrogen sulphide concentration of 80 mg/l was measured in manhole B-1 where the manhole cover had been removed and a drum was put as temporary cover resulting in good ventilation. Depth of the manhole is as deep as 3.2 m, and water fall of about 2 m from a 225 mm diameter inlet sewer to 400 mm diameter outlet sewer. Because of the water fall, sewage is stirred and hydrogen sulphide gas is generated.

Low concentration of 14 mg/l was measured in manhole B-2 in the upstream section. In manhole B-3, concentration changed from 90 mg/l for the first measurement to 10 mg/l after 25 minutes. Considerable flow was observed, water depth in the 400 mm diameter sewer pipe was about 15 cm and velocity was about 30 cm/s by observation when the first measurement was made. The flow decreased and velocity was about 3 cm/s at the second measurement. In between the two measurements, pump might be stopped.

There is a collapse of force main in the power station in the upstream section of manhole B-3. Sewage overflows at that point. Thus, all sewage flow does not reach to manhole B-3. Hydrogen sulphide gas concentration would be higher than measured if all the flow reached the manhole. Heavy corrosion inside the manhole indicates high concentration.

In addition to the reasons for high concentrations in manholes B-1 and B-3 mentioned above, number of inlet pipes to manhole also affects the concentration. The larger is the number, the higher is concentration. For instance, numbers of inlet pipes to manholes B-1 and B-3 are 4 and 3 respectively, compared with 2 to manhole B-2 where the concentration was as low as 14 mg/l.

The same tendency was recognized in the other zones as described below.

Manhole No.	Number of Inlet Pipes	H_2S Concentration (mg/l)
E-1	6	500
G-1	5	41
M-1	4	66
M-2	4	107
B-4	5	180

The larger number of inlet pipes results in the greater amount of sediment in the manhole which in turn results in greater chances for hydrogen sulphide gas generation.

High concentration of 180 mg/l was recorded in manhole B-4 which receives sewage from Gamal area, although the line is gravity sewer. Pipes were replaced in March 1989. Heavy corrosion was recognized in the manhole.

(8) Zone L

Zone L is located in the northern part of the district and bounded by the airport on the north. Sewages in the zone flow into P/S L and pumped to manhole M-5 in zone M. Sewage from Badr Camp is pumped into the zone from the west. Sewage from the airport is also pumped to manhole L-2.

Only one collapse was reported in this zone. Percentage of collapsed sewer length is 3 %.

Hydrogen sulphide gas concentration was 38 mg/l in manhole L-1. This manhole is close to P/S L, and good ventilation is expected through pump well. Corrosion of manhole is not considerable.

Hydrogen sulphide gas concentration was 14 m g/l in manhole L-2 which is located at the terminal point of the force main from the airport.

(9) Zone M

Zone M is located at the southeast corner of the district. Sewages in the zone flow into P/S M and pumped to manhole M-5 which also receives sewage from zone L. Sewage from the military area surrounded by the zone M flow to manhole M-1. Sewage from Happy area also flows to manhole M-4.

Many collapses of sewer pipes were reported in the zone, although all the lines are gravity sewers. Total length of collapsed sections accounts for 14 % of the total length of sewers.

Hydrogen sulphide gas concentrations were 66 mg/l and 107 mg/l in manholes M-1 and M-2 respectively, both of which are heavily corroded. Inlet pipes to these manholes are 4 each, which contribute partly to high concentration.

Concentration in manhole M-3 was as low as 1 mg/l. This manhole was constructed 2 - 3 years ago, but full of sewage because of the collapse in the downstream section.

Although concentration was as low as 11 mg/l in manhole M-4, heavy corrosion was observed. This manhole receives sewage from Happy area.

Manhole M-5 which is located at the terminal point of the force main from zones L and M, was full of sewage because of collapse in the downstream section, and measurement could not be done. Heavy corrosion was observed on the manhole cover. Inside of the manhole is supposed to be heavily corroded.

Concentration in manhole T-1 changed from 109 mg/l at 9:30 on 16 August to 24.9 mg/l at 7:40 next day. The reason for this fluctuation of concentration is not obvious. The factory usually starts at 8:00, and wastewater flow is supposed to change at that time. There is a collapse of sewer caused by intrusion of tree roots. Sewage stagnated in this manhole, and flow could not be measured.

High concentration of hydrogen sulphide gas was observed in the downstream of the force main as expected. Out of the 30 measuring points, 7 points recorded concentrations higher than 90 mg/l. Five of the seven points of high hydrogen sulphide gas concentrations are located at downstream section of the force mains. Concentrations of more than 100 mg/l were observed at E4, E1 and T1 points, each having two pumping stations at upstream section.

Out of 10 points at the terminal points of force mains, five points recorded low concentrations. These are J0, J1, J3, C2 and L2. The reasons for low concentration are considered as follows.

J0, J1 A large quantity of fresh wastewater from a milk factory flushes the sewers periodically giving no chance for hydrogen sulphide gas generation.

J3, C2, L2 There are collapses of pumping stations or pipes in the upstream sections of these points, and sewages do not flow properly at the points. Sewage flows at the time of measurement were very small. Therefore, conditions of these points do not reflect usual conditions.

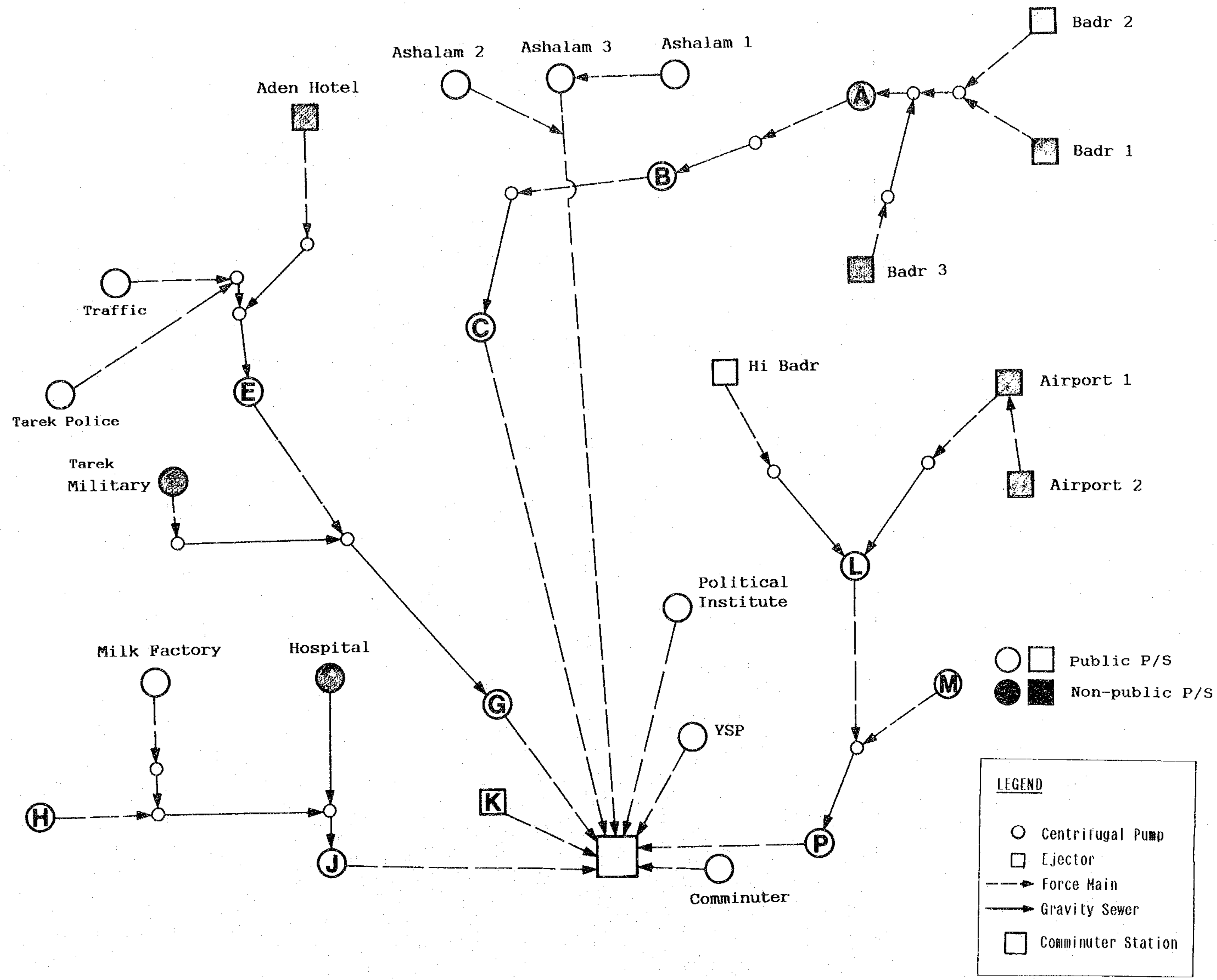

Figure I.1 Sewerage System in Khormaksar

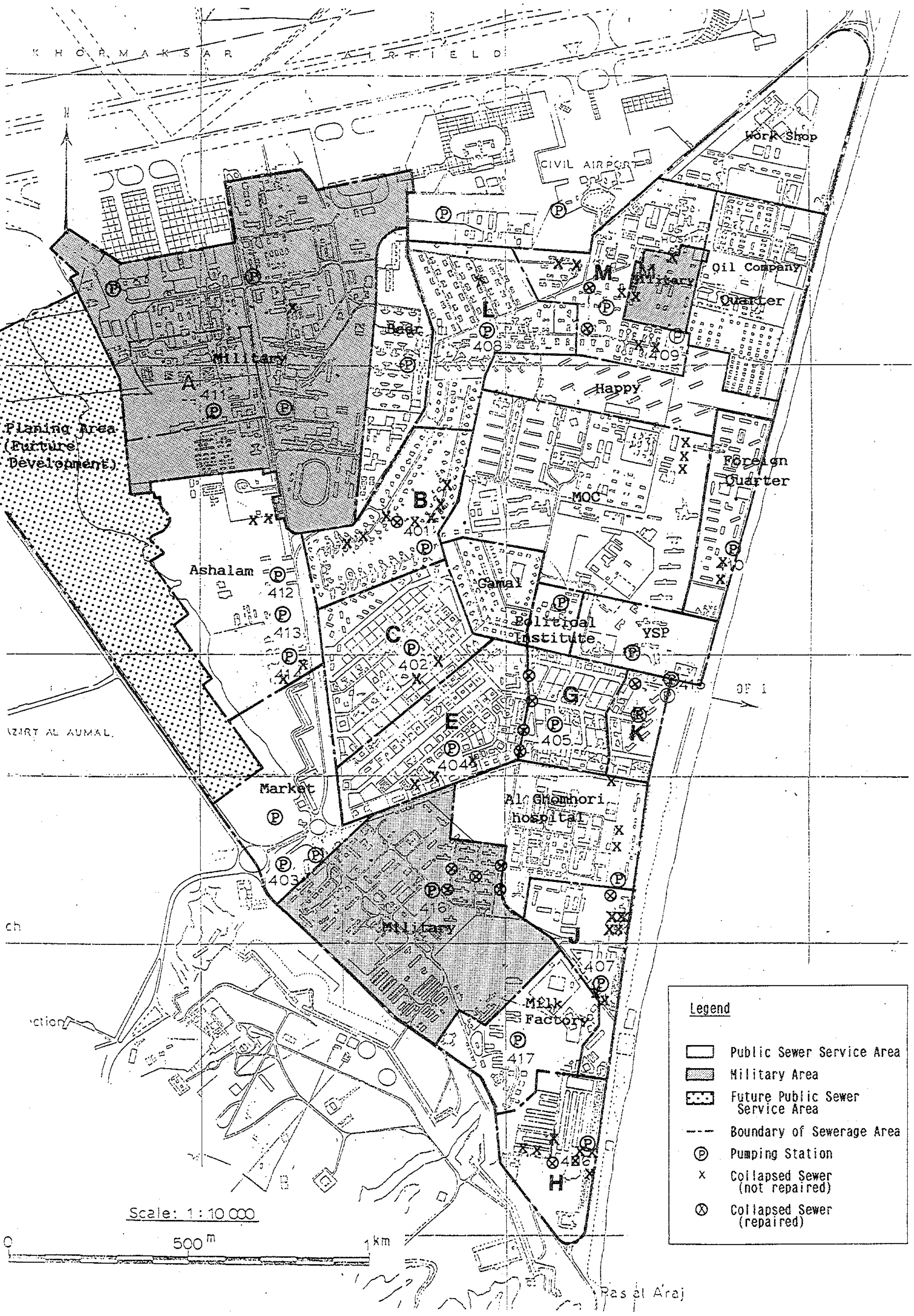

Figure I.2 Distribution of Collapsed Sewers

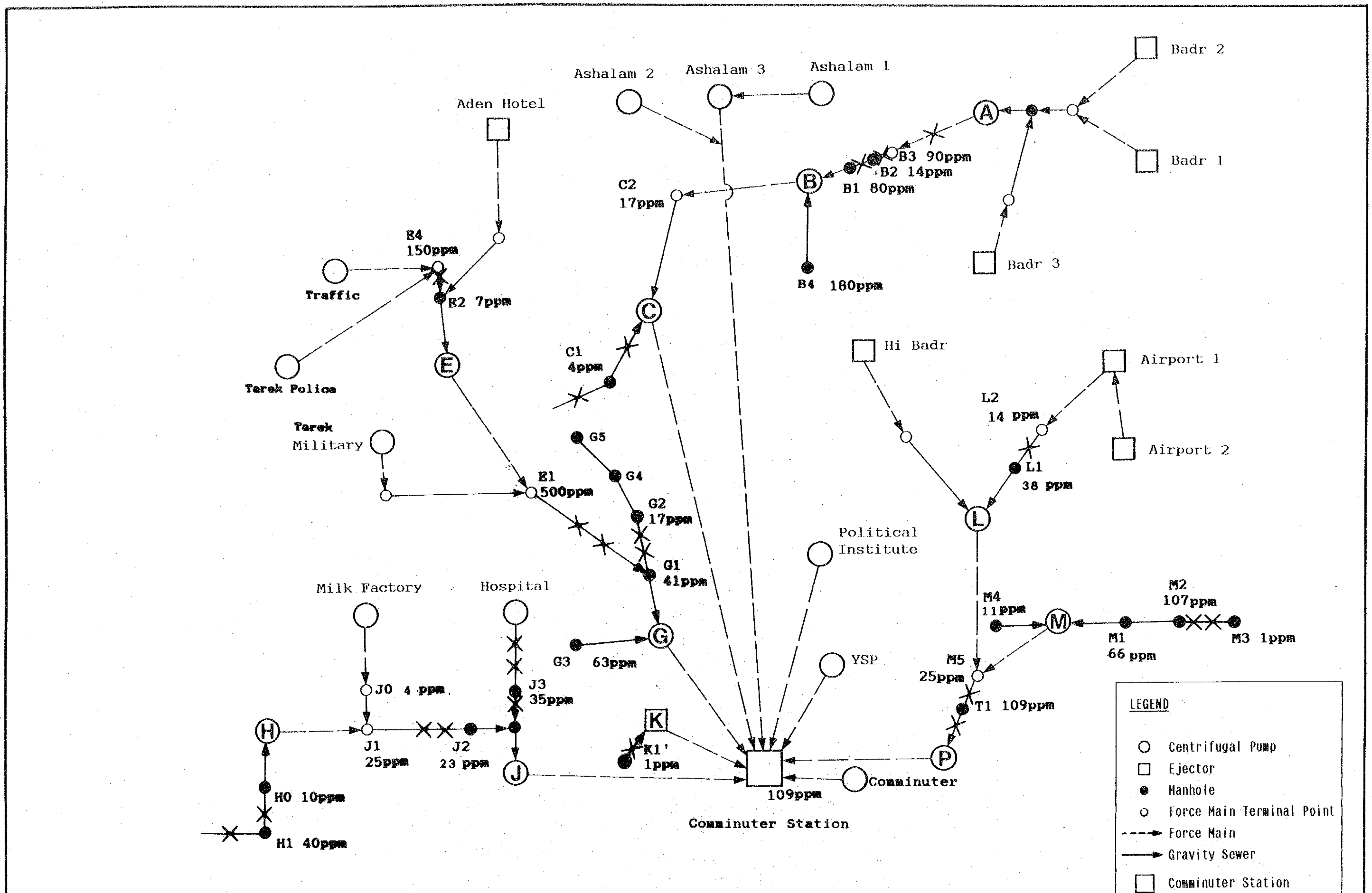

Figure I.3 H_2S gas Measuring Points

Table I.1 Investigation of H_2S Gas in Khormaksar Area

Name	Date Time	H_2S ppm	Temp Amb. Wat. (°C)	Manhole Size Dep.×W×L (m) (m)(m)	Pipe Outlet Inlet (mm)	Notes
H-0 ②	14/8 9:30	10	36 35	-		Blockage was found the upper and lower stream.
H-1 ②	10:00	40 75	36 35	-		Big amount of sludge. When stir the sludge, 75 ppm was detected.
J-0 ①	10:20	4	36 35	1.0×1.2×1.0 F.M terminal from Milk Fac.	φ300 φ300	Most sewage come from Milk factory, so sewage color is white.
J-1 ①	10:25	25	36 34	1.5×1.2×1.0 F.M terminal	φ400 φ400 φ300	Heavy corrosion.
J-2 ②	10:40	23	35 35	2.8×1.5×1.0	φ400 φ400 φ225 φ150	Heavy corrosion.
J-3 ②	10:50	35	37 35	1.35×1.5×1.0	φ225 φ225	No flow due to the block age in upstream. Not high gas density be- cause of J P/S in down- stream. Heavy corrosion.
E-1 ①	13/8 9:06	500	33 33	2.35×1.8×1.4 F.M Terminal from E P/S.	φ300 φ300 φ225×5	Flow is small. Heavy co- rrosion can be seen.
E-2 ②	8:50	7				Full of water due to the blockage in downstream. Every day take out the sewage by collected car.
E-3 ②						Destroyed already.
E-4 ①	8:40	2 150	34 32	1.4×1.4×1.8 F.M Terminal from traffic P/S.	φ250 φ250×2	The upper P/S(ADEN HO- TEL) is not operate. When P/S(Traffic office) start H_2S became 150 ppm

Note) Heavy corrosion - Concrete -almost danaged
- Cover -not proper shape

Middle corrosion- almost surface is damaged
① - End of force main
② - Collapse point
③ - Non damaged point

Name	Date Time	H_2S ppm	TEMP Amb. Wat. (°C)	Manhole Size Dep.×W×L (m) (m)(m)	PIPE Outlet Inlet (mm)	NOTES
C-1 ②	13/8 11:05	4	30 32	2.4×1.5×1.3	φ300 φ300 φ225×2 φ150	Most of sewage seemed to be ground water.
C-2 ①	11:20	17	35 32	2.4×1.5×1.3 F.M Terminal from B P/S	φ400 φ400 φ150	The upper P/S is out of order.
G-1 ②	13/8 9:25	41	34 33	3.3×1.2×1.0	φ400 φ400 φ300 φ225×3	Middle corrosion can be seen.
G-2 ②	9:45	17	35 35	2.25×1.5×1.2	φ400 φ400 φ225	Middle corrosion can be seen.
G-3 ③	10:25					MH was set up in 1963. We could not open it.
G-3' ③	10:30	63	36 32	3.5×1.0×0.8	φ300 φ225 φ300	The cover correded heavily. Concrete corroded middly.
G-4 ③	10:30	-	-	-	-	Clocked. Full of water.
G-5 ③	10:40	-	-	-	-	Clocked. Full of water.
K-1 ②						This MH is in Russian Embassy.
K-1' ②	10:45	1	37 32	1.7×1.7×1.5	φ225 φ225 φ150	

Note) Heavy corrosion - Concrete -almost damaged
- Cover -not proper shape
Middle corrosion- almost surface is damaged
① - End of force main
② - Collapse point
③ - Non damaged point

Name	Date Time	H_2S ppm	TEMP Amb. Wat. (°C)	Manhole Size Dep.×W×L (m) (m)(m)	PIPE Outlet Inlet (mm)	NOTES
B-1 ②	14/8 9:45	80	36 35	3.2×1.3×1.2	φ400 φ300 φ225 φ150 φ100	Middle corrosion. No proper MH cover but an iron plate. Gas is hight due to drop (about 2 m)
B-2 ②	8:35	14	34 32	2.9×1.6×1.3	φ400 φ300 φ225 φ150	
B-3 ①	9:00	10 90	34 32	1.5×1.5×1.0 F.M Terminal from A P/S.	φ300 φ225 φ150×2	90 ppm at pump operation Heavy corrosion.
B-4 ③	16/8 10:00	180	36 32	2.8×1.5×1.0	φ300 φ300×3 φ150×2	Heavy corrosion. Sewer is new (1989 3).
L-1 ③	15/8 9:00	38	36 32	3.4×1.8×1.2	φ225 φ225×3 φ150	
L-2 ①	9:20	14	36 32	1.2×1.5×1.2 F.M terminal	φ225 φ150×3	One old pipe converted to clay pipe (150 mm).
M-1 ②	9:30	66	36 32	2.7×1.5×1.0	φ225 φ225×3 φ150	Heavy corrosion.
M-2 ②	9:45	107	35 32	2.6×1.5×1.0	φ250 φ250 φ225×3	Heavy corrosion.
M-3 ②	10:15	1	35 32	- Full of Water	-	
M-4 ③	10:35	11	37 32	1.55×1.5×1.0	φ150 φ150×2	Middle corrosion.
M-5 ①	10:40	25	40 34	-	-	Over flow.

Note) Heavy corrosion - Concrete -almost damaged
- Cover -not proper shape
Middle corrosion- almost surface is damaged
① - End of force main
② - Collapse point
③ - Non damaged point

Name	Date Time	H_2S ppm	TEMP Amb. Wat. (°C)	Manhole Size Dep.×W×L (m) (m)(m)	PIPE Outlet Inlet (mm)	NOTES
T-1 ②	16/8 9:30	109	35 32	2.1×1.6×1.2	-	Full of water. MH is in the plastic factory. The pipe was broken by the roots of trees.
S.T IN MOC	9:45	0.2	37	1.4×2.2×1.5 1.4×2.65×5.8	-	Full of night soil. not connect to sewer.
F.M from A PS ②	14/8 9:30	14	34 32	2.9×1.6×1.3	φ225	Blockage was found under the warehouse in Power Station. It was blocked in July, 1989.
C.S ①	15/8 8:40	109	38 32			Comminuter Station
C.S ①	10:57	109	40 32			Comminuter Station
C.S ①	16/8 10:50	109	35 32	-	-	Comminuter Station

Note) P/S - Pumping Station
F.M - Force Main
C.S - Comminuter Station
S.T - Septic Tank
MOC - Ministry of Construction
① - End of force main
② - Collapse point
③ - Non damaged point

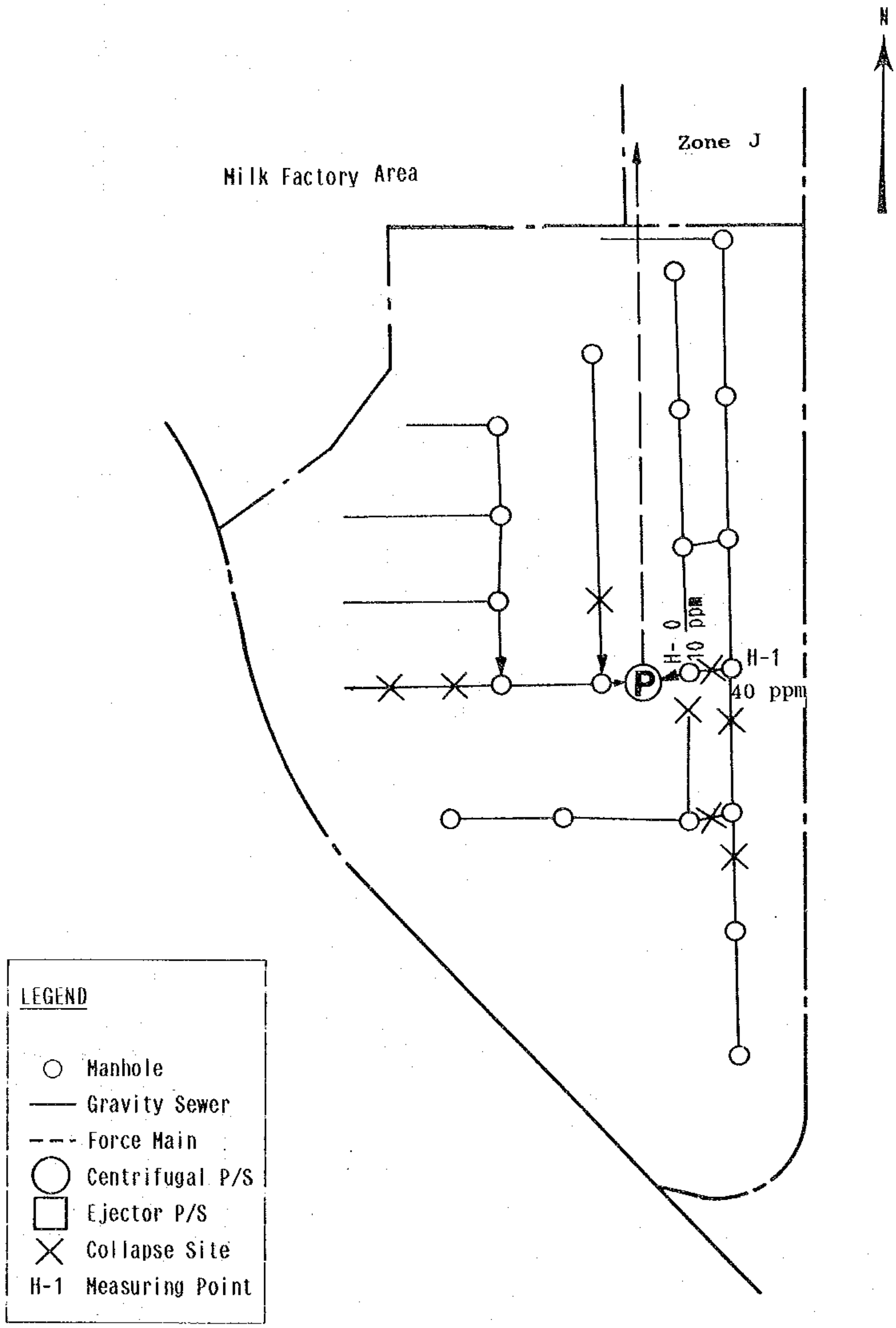

Figure I.4 Present Condition of Sewerage System in Zone H

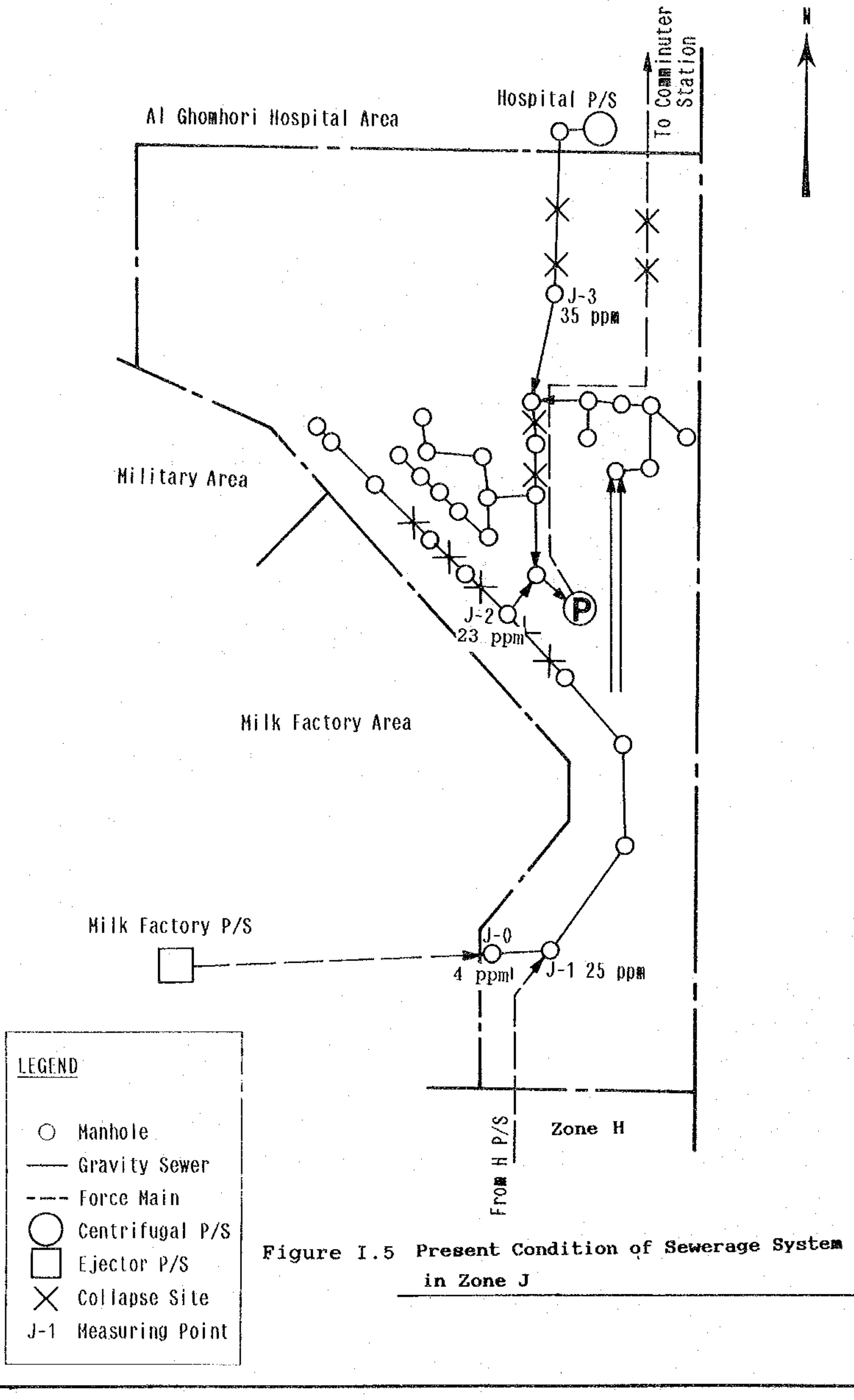

Figure I.5 Present Condition of Sewerage System in Zone J

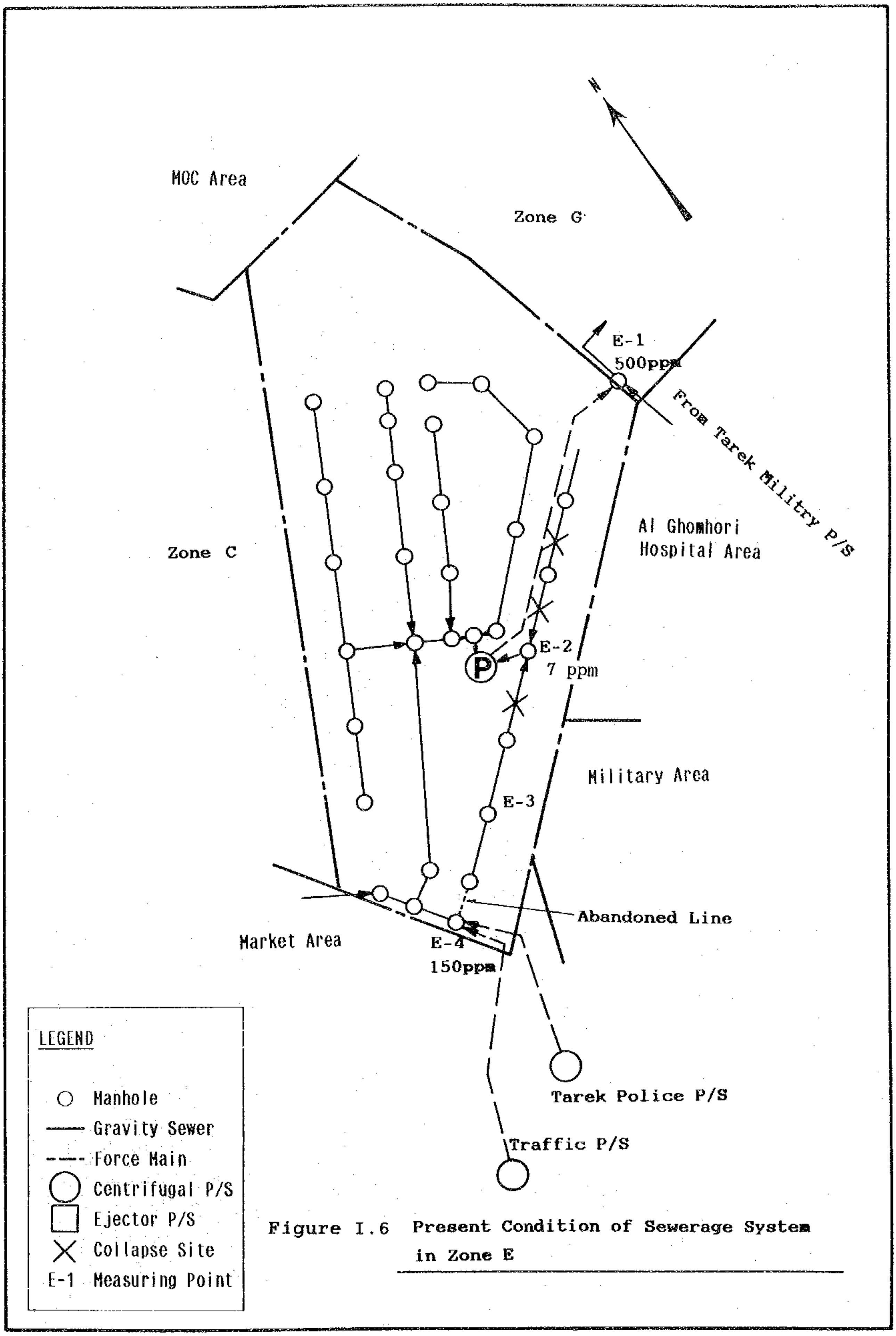

Figure I.6 Present Condition of Sewerage System in Zone E

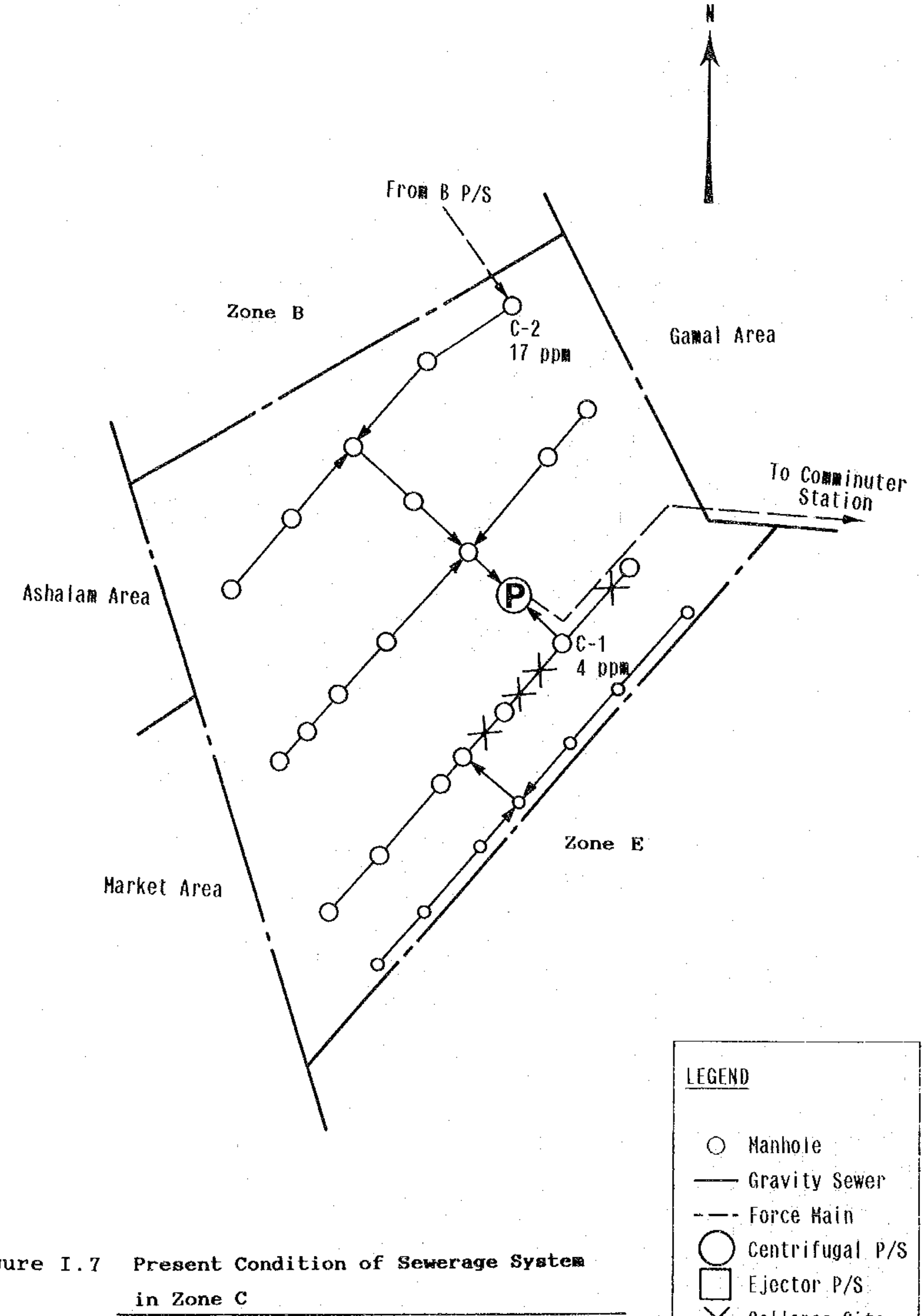

Figure I.7 Present Condition of Sewerage System in Zone C

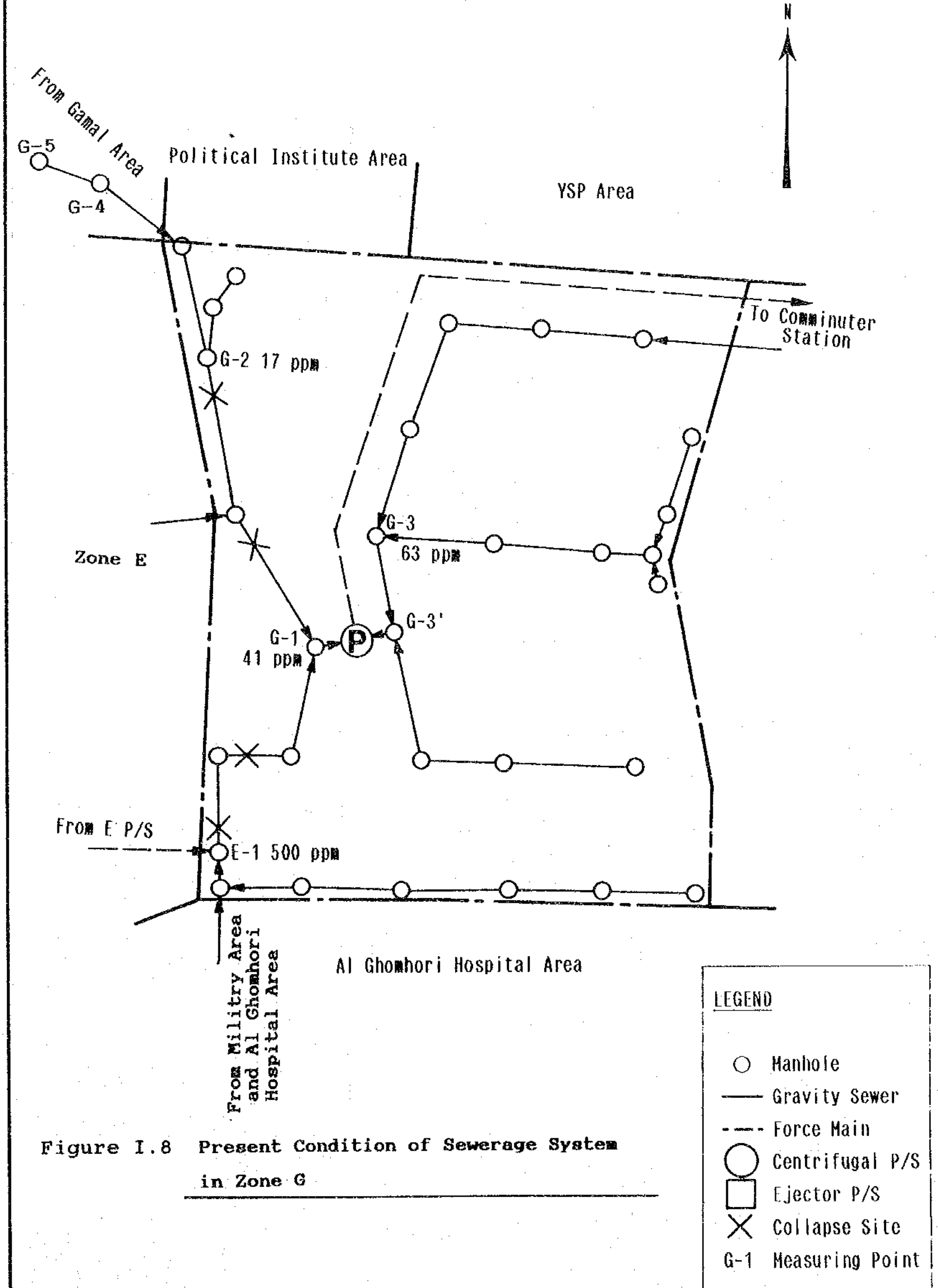

Figure I.8 Present Condition of Sewerage System in Zone G

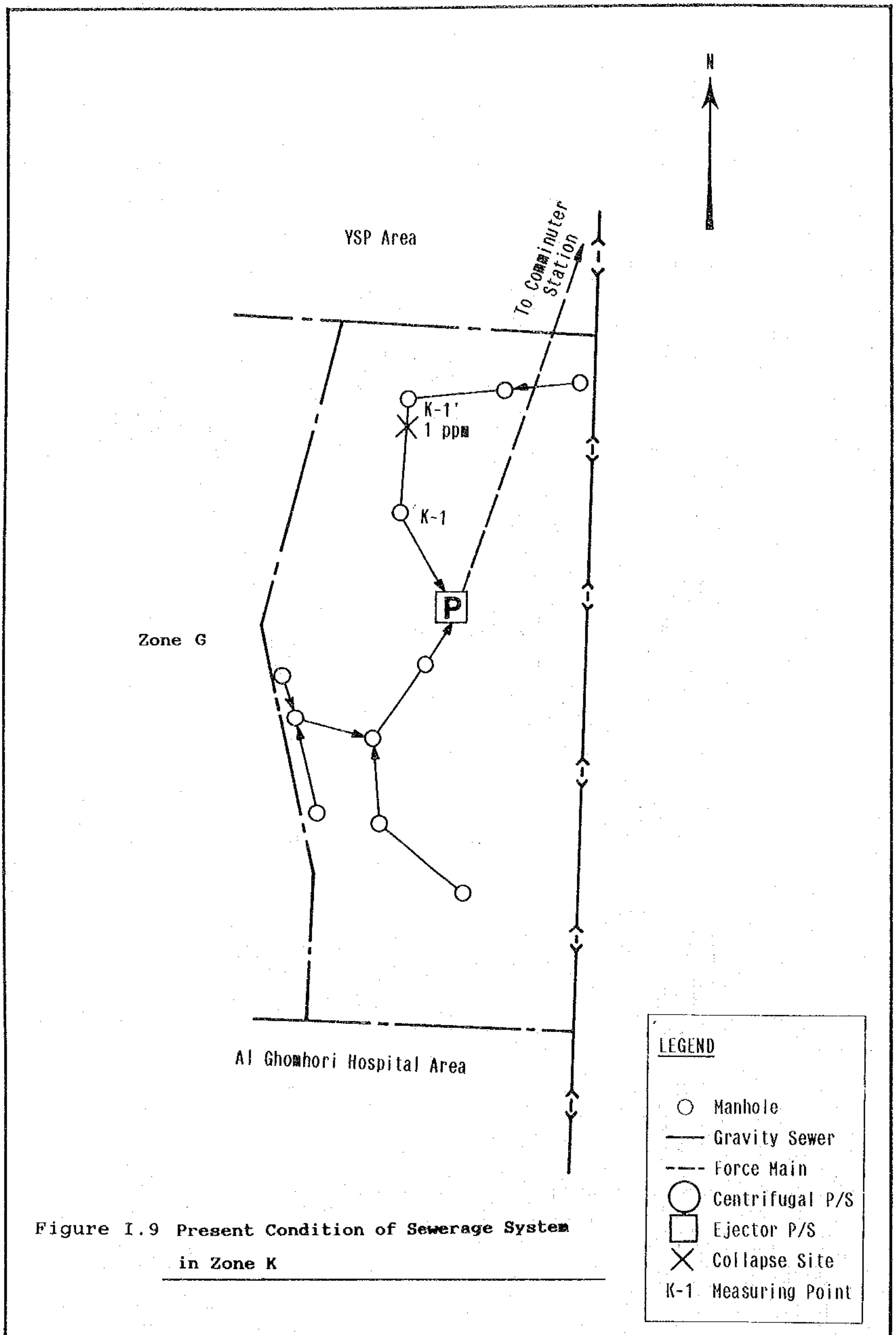

Figure I.9 Present Condition of Sewerage System in Zone K

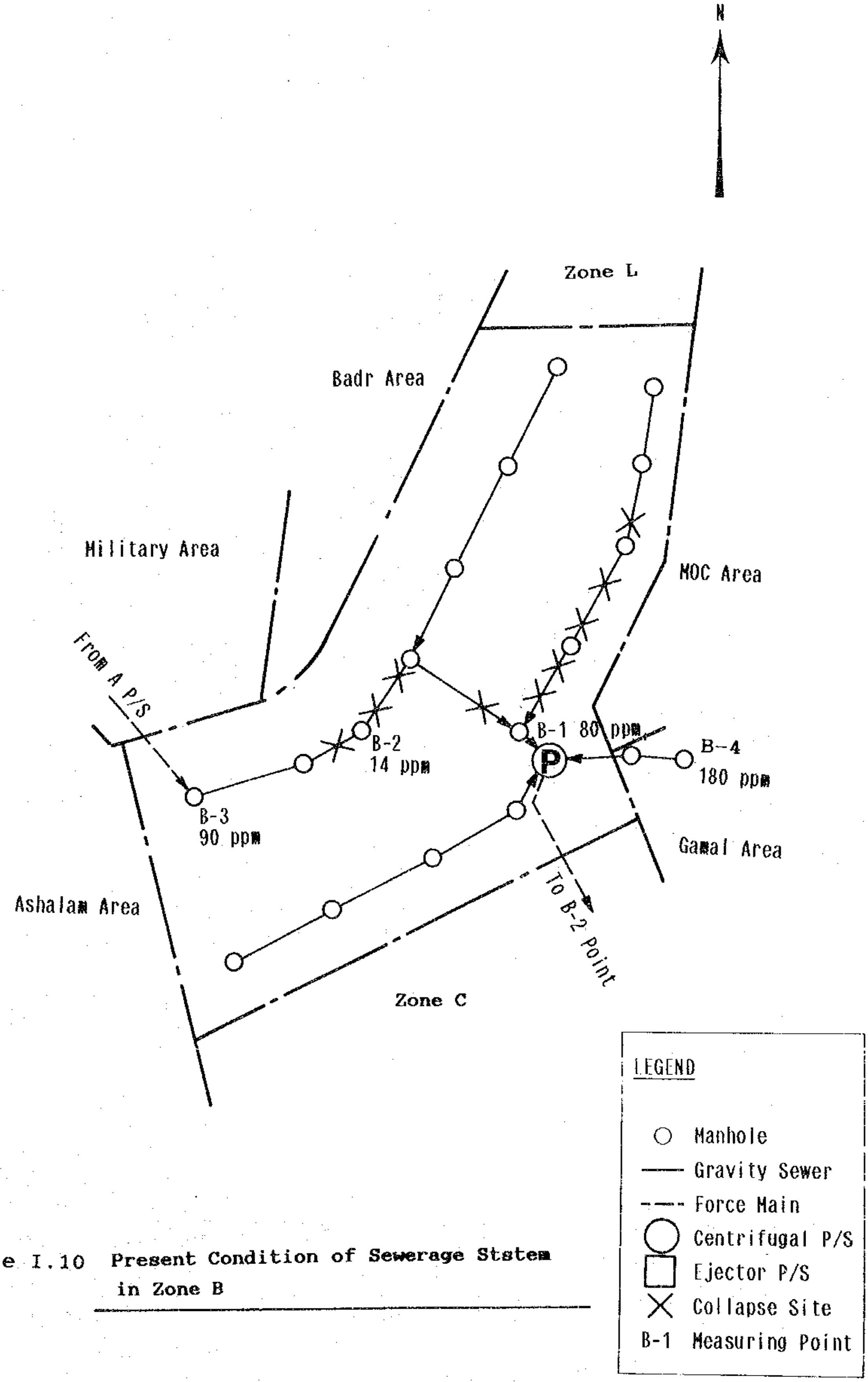

Figure I.10 **Present Condition of Sewerage Ststem in Zone B**

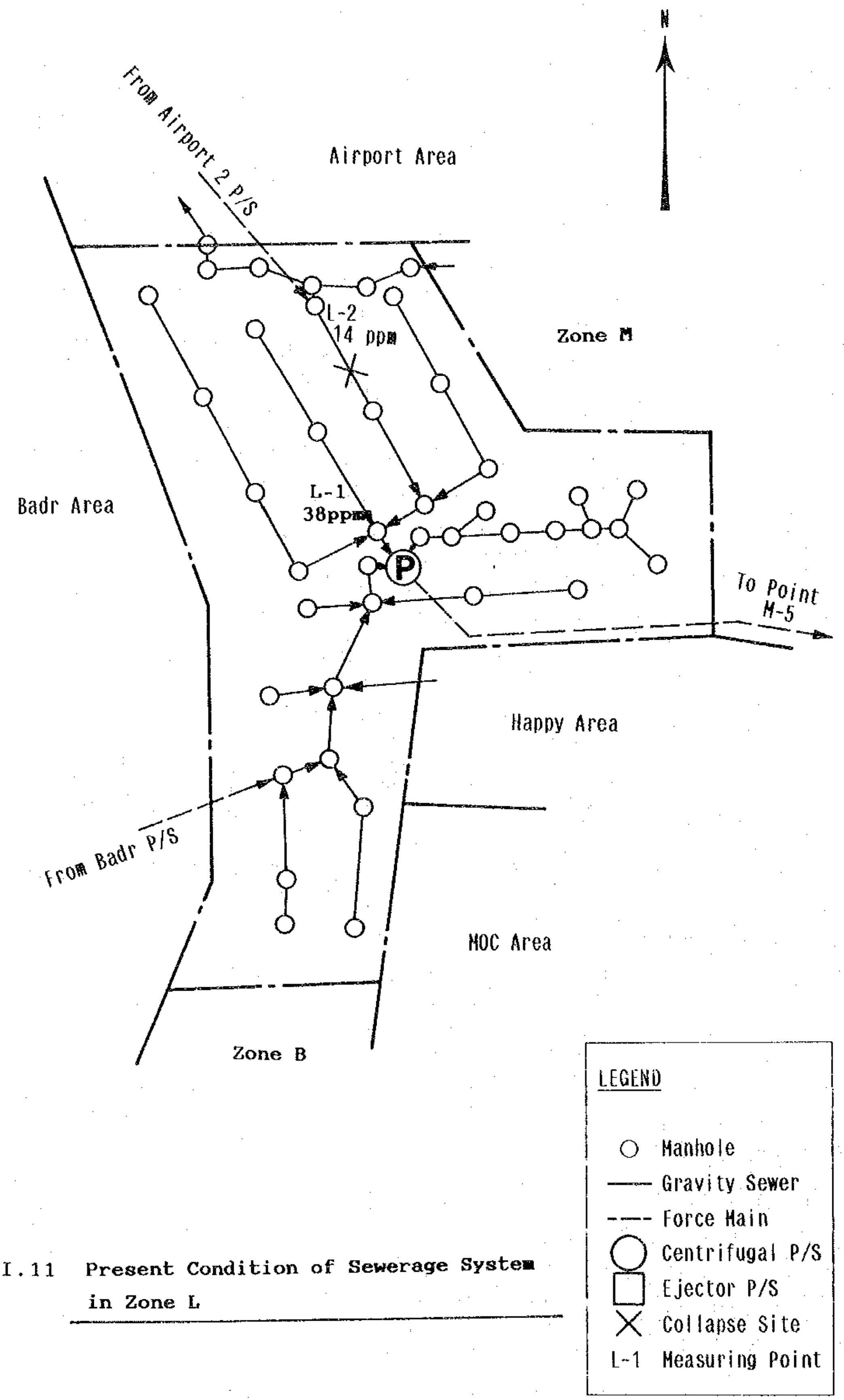

Figure I.11 **Present Condition of Sewerage System in Zone L**

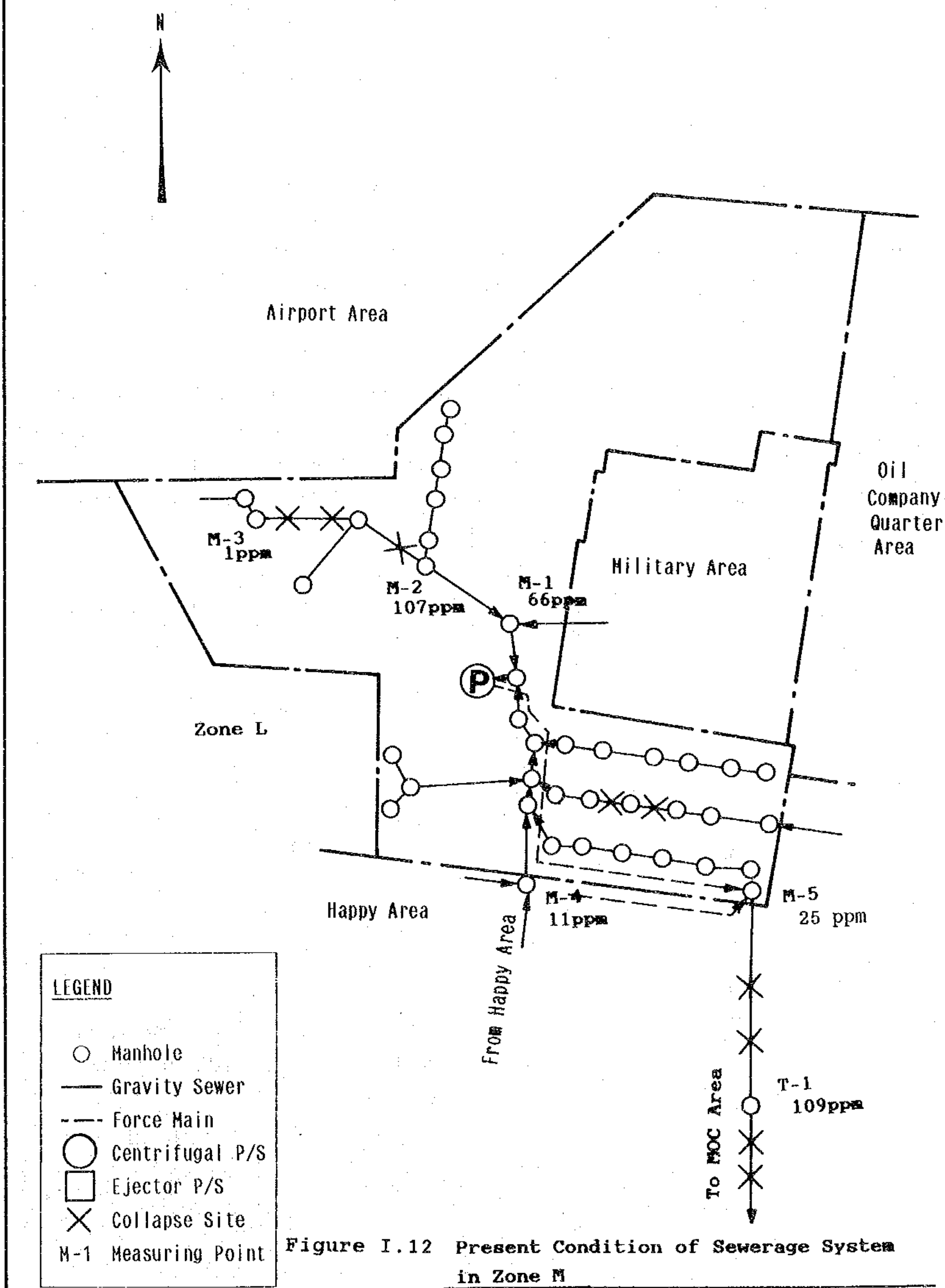

Figure I.12 Present Condition of Sewerage System in Zone M

The reasons for the high hydrogen sulphide gas at B-4 and M2 points are not clear. However, high temperatures were recorded at these points (air temperature 33 - 40°C, air temperature in the manholes 32°C, and water temperature 32 - 35°C). Besides high temperatures, flow velocities were low (14 -15 cm/sec) and a large quantity of sediments were observed. These conditions may probably promote the generation of hydrogen sulphide gas.

An interesting fact was observed at B1 point. The cover of this manhole was removed and a temporary drum cover was put on. Thus, ventilation of the manhole is quite good. However, a high concentration of 80 mg/l was observed. One reason for that is structure of the manhole. This manhole is 3.2 m deep, and a water fall of about 2 m is in it. Turbulence caused by water fall promote release of hydrogen sulphide gas.

In general, concentrations of hydrogen sulphide seem to be lower than expected. One of the reasons for this is improper function of the sewers. There are many collapses and break downs of pipes and pumping stations which affect the measurement of concentration. If there were not collapses nor break downs, the higher concentrations should have been observed.

A fact observed at E4 point that concentration increased from 2 mg/l before pump started to 150 mg/l after flow reached the manhole suggests the fluctuation of hydrogen sulphide gas generation. Therefore, there is a possibility of high concentration among the points which recorded low concentration.

Although hydrogen sulphide gas concentrations were lower than expected, they are high enough to necessitate some counter measures to protect laborers working in the manholes and materials of pipes and other structures.

4. Causes for High Concentration

In order to analyze the causes for high concentration of hydrogen sulphide gas, the results presented in Table I.1 were processed as shown in Figure I.13.

First, all of the sampling points are classified by their locations into three categories, viz. terminal point of force main, in the downstream section of force main and on the gravity sewers. As shown in the figure, no meaningful correlation was recognized between location of sampling point and hydrogen sulphide gas concentration. Higher concentration of more than 50 mg/l were measured at every category of location.

Second, correlation between the number of inlet pipes and concentrations was analyzed since it was realized during the measurement that higher concentrations were recorded in manholes with many inlet pipes. As shown in the figure, positive correlation between numbers of inlet pipes and concentration was recognized. As the number of inlet pipes increase, the higher concentration was recorded.

Many factors other than those mentioned here affect the generation of hydrogen sulphide gas and collapses caused by its attack. Number of inlet pipes is one of the important factors in the district. One of the reasons for this correlation is that more sediment accumulates in long and gradual inlet pipe than in public sewer line. As the number of inlet pipes increase chances for gas generation increase.

Present practice to connect inlet pipes to manholes forces long and gradual inlet pipes. Direct connection of inlet pipe to public sewer is recommended to avoid accumulation of sediment which will cause generation of hydrogen sulphide gas.

5. <u>Damaged Sewer Pipes</u>

Based on the investigation of the existing sewerage system in each zone, the lengths of the sewer pipe sections where collapses occurred and replacement is required have been calculated as shown in Table I.2.

Military areas are excluded from the table, since investigation could not be done. Zones named as Al Ghomhoria Hospital, Gamal, Ashalam and MOC are combined to neighboring zones for the convenience of calculation. There is no collapse in zones Milk Factory, Market Area, Ashalam, Happy and a part of Gamal. Sewer pipes in these zones are in good condition, and no replacement is required.

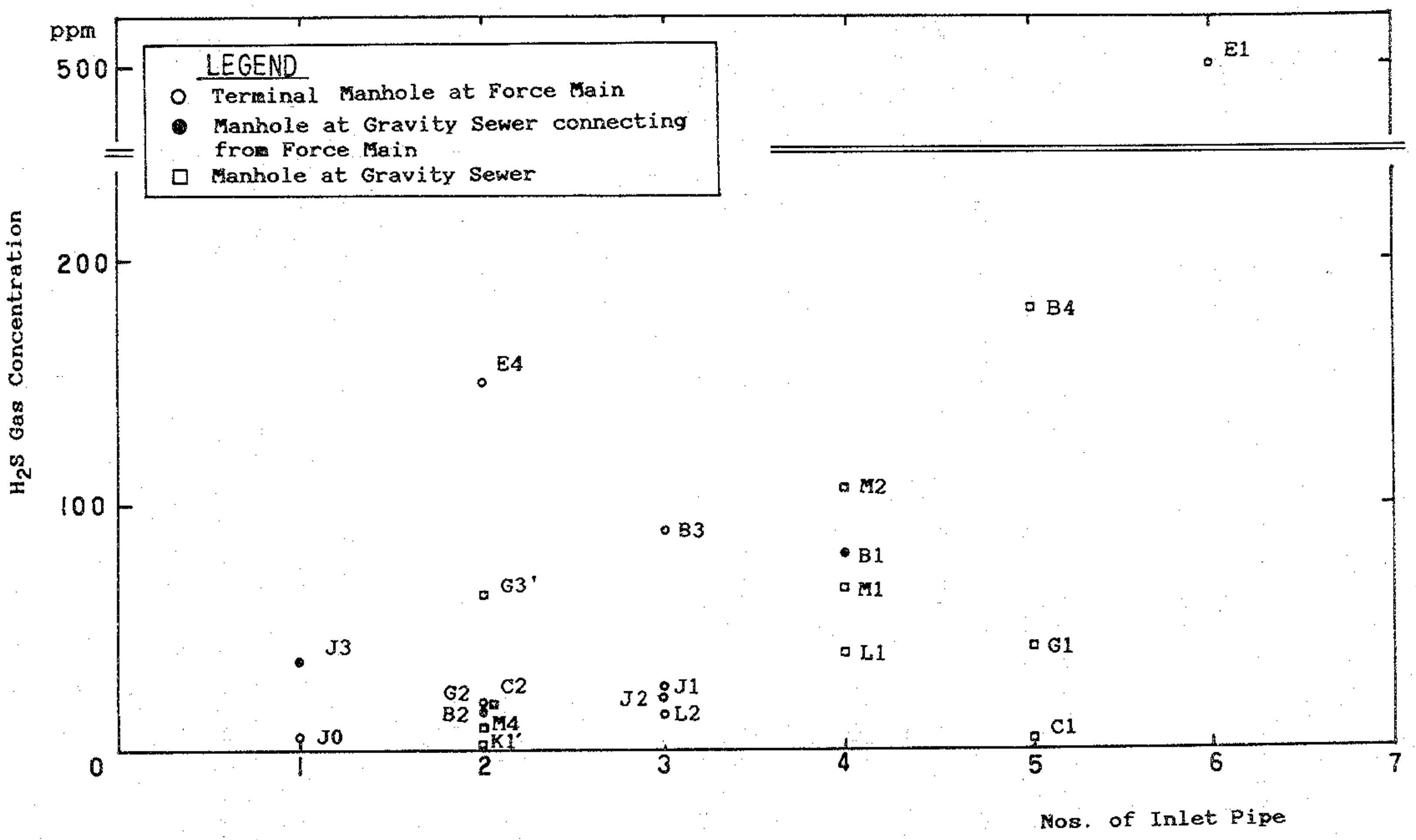

Figure I.13 Correlation between Hydrogen Sulphide Gas and Nos. of Inlet Pipe

Table I.2 <u>Length of Damaged Sewer Pipes</u>

Zone	Total Length of sewer (m)	Length of Damaged Sewers (m)	Percentage of Damaged Sewers (%)
H	1,960	550	28
J	1,639		
Al Ghomhoria Hos.	1,055		
Sub-total	2,694	735	27
E	2,244	250	11
C	1,863	270	14
G	1,624		
Gamal	295		
Sub-total	1,919	245	13
K	501	60	12
B	1,455		
Ashalam	-		
Sub-total	1,455	640	44
L	2,610	75	3
M	2,057		
MOC	845		
Sub-total	2,902	720	25
Total	18,148	3,545	20

Total length of sewer pipes in 9 zones is 18,148 m, out of which 3,545 m or 20 % is damaged and requires immediate replacement.

6. Need for Future Replacement

As described in the previous section, about 20 % of the existing sewer pipes are damaged and requires immediate replacement. This is significant figures. Most of the existing sewer pipes were installed in early 1960s. About 25 years of usage, 20 % of the sewers were damaged by hydrogen sulphide gas attack or other reasons.

The existing sewer pipes will be damaged further by the same cause because the condition of the area will remain same as it is now and pipe materials are asbestos cement which is subject to hydrogen sulphide gas attack.

Three factors, viz. hydrogen sulphide gas concentration, percentage of damaged sewers and present sewage flows, in each zone have been analyzed to evaluate the vulnerability of zones to further damages.

As a result of the analysis, the highest contributing factor is identified to be the percentage of damaged pipes, then second is hydrogen sulphide concentration and the last is sewage flow.

The result of the analysis is considered reasonable for the following reasons. Collapses reported in the past often occurred in the next section of the replaced sewers. This means that collapses are likely to occur in a certain specific sewer line. The high concentration of hydrogen sulphide gas indicates possibility of damages. However, because of the collapses presently occurred, measured values do not represent the actual ones under normal condition. Sewage flow has positive and negative effects on damages. A large flow means a large quantity of organic materials to cause the hydrogen sulphide gas generation. On the other hand, the larger is sewage flow, the higher is velocity, resulting in less chance for gas generation. These two effects canceled each other and result in slightly positive effect.

Taking into account the fact that average percentage of damaged section accounts for approximately 20 % after 25 years of operation, the average percentage of damaged section up to 2010 is assumed to be 50 %. Necessity of future replacement in each zone is analyzed in the same manner as described above on this assumption. As a result all the zones were classified into the four ranks according to the results of analysis as shown below.

Rank	Percentage of Replacement	Zone
1	100 %	B
2	60 %	C, E, G, H and K
3	30 %	J and M
4	0 %	Milk Factory, Market Area, Ashalam and Happy

Length of the future replacement based on the analysis is calculated as shown in Table I.3 below. These figures are calculated on the assumption mentioned above and not necessarily predict the actual length of the replacement. Nevertheless, vulnerability of each zone to damages by hydrogen sulphide gas is reasonably evaluated.

Table I.3 Length of Future Pipe Replacement

Zone	Total Length of Sewer (m)	Damaged Pipe Length (1989) (m)	Damaged Pipe Length (90-2010) (m)	Total Length of Replacement (m)
H	1,960	550	630	1,180
J	2,694	735	75	810
E	2,244	250	1,100	1,350
C	1,863	270	850	1,120
G	1,919	245	905	1,150
K	501	60	240	300
B	1,455	640	815	1,455
L	2,610	75	705	780
M	2,902	720	150	870
Total	18,148	3,545	5,470	9,015

APPENDIX J
SOIL TEST

APPENDIX J

SOIL TEST

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APPENDIX J

SOIL TEST

1. Introduction

In order to determine the most suitable borrow pit site for the construction of embankment at STP site and to obtain design parameters, soil test was carried out.

A huge quantity of impermeable soil, approximately 384,000 m^3, is required for the construction of embankment in STP. Therefore location of borrow pit greatly affects the construction cost of STP. The below mentioned four sites were suggested by GDLG and Aden Municipality as borrow pit.

- Bir Omar	27 km from STP
- Sheikh Othman	13 km from STP
- Hiswa	15 km from STP
- Dar Sa'ad	21 km from STP

Samples from the four sites and one from STP site were taken by the study team and brought to the MOC laboratory for test. Test items are as follows.

a. Sieving Test
b. Unit Weight
c. Permeability Test

Locations of sampling and test points are shown in Figure J.1.

2. Results of Test by MOC

(1) Permeability Test

Variable head method was adopted to get coefficient of permeability. The results of the test are shown in Table J.1 below. Coefficients of permeability at the four borrow pits are at the order of 10^{-3} to 10^{-4}.

These values are higher than expected from the observation at the sites. Bulk densities of the samples are unknown.

The test was carried out after adding of 10 % of water to the sample. Weight of water added was determined by pretest for optimum moisture contents. However, 10 % of moisture content may not be the optimum one.

(2) Sieving Test

Soil property at each borrow pit and STP site is considered as follows based on the results of the test shown as grading accumulation curve in Figure J.2.

- STP site	Sand
- Bir Omar	Silty clay
- Sheikh Othman	Silty sand
- Hiswa	Sandy silt

Results of the test for the sample taken from Dar Sa'ad has not been obtained.

Grading accumulation curves of the four samples are shown in Figure J.2. From the curves, it can be said although the grading in fine particle size area is not known that the order of well grading is 1) Bir Omar, 2) Hiswa, 3) Sheik Othman and 4) STP Site.

(3) Test in Japan

Two samples from Sheik Othman and Dar Sa'ad were brought back to Japan for further test. The results of the test are shown in Table J.2. Permeability coefficient were tested with the optimum moisture content and Dar Sa'ad samples are 10^{-7} and 10^{-4}, respectively. Order of the permeability coefficient of Sheik Othman sample is considerably different from that obtained in Aden. The reasons for this difference may be lie in the difference of compaction method and measurement of the optimum moisture content although the exact reasons are not apparent.

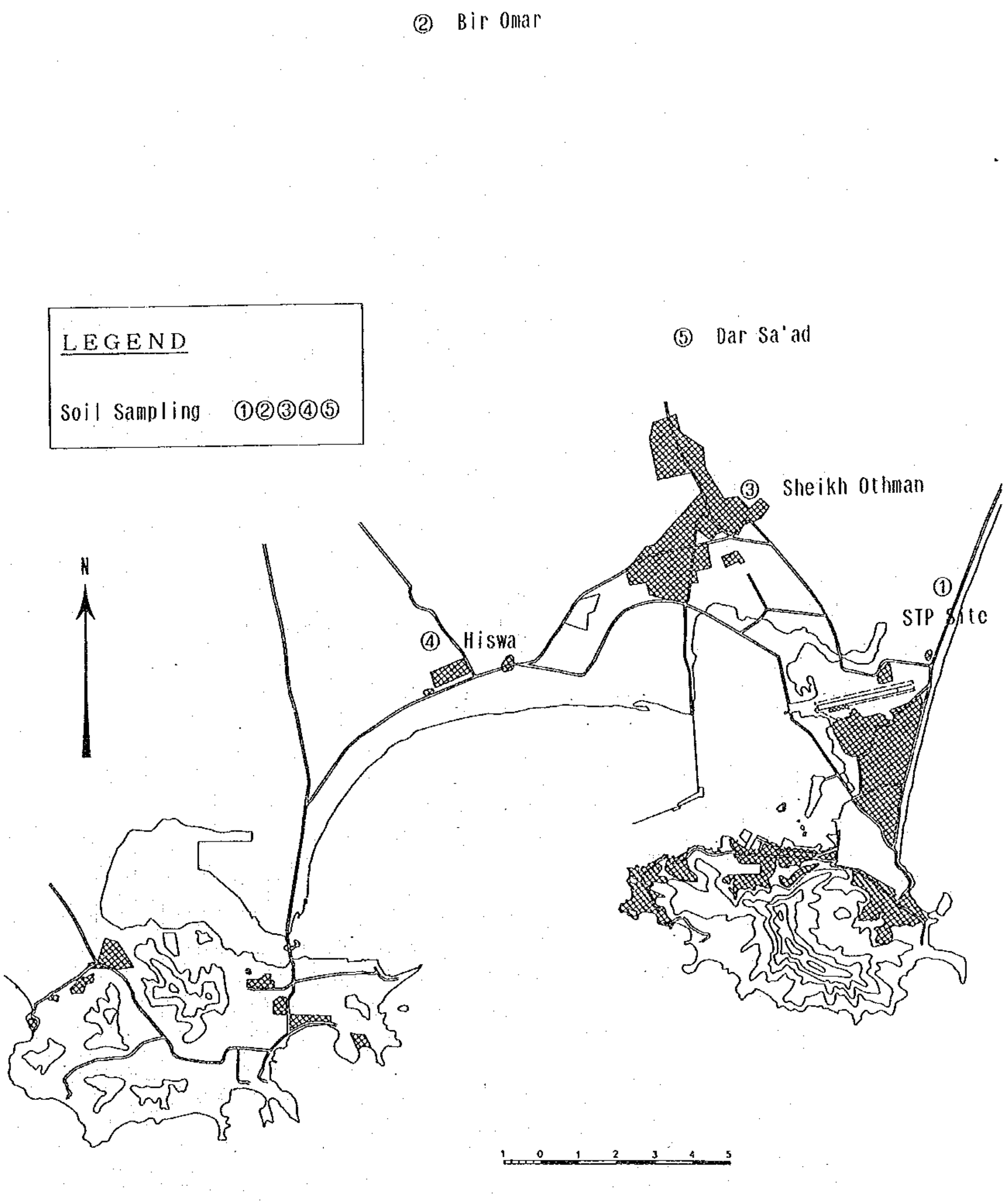
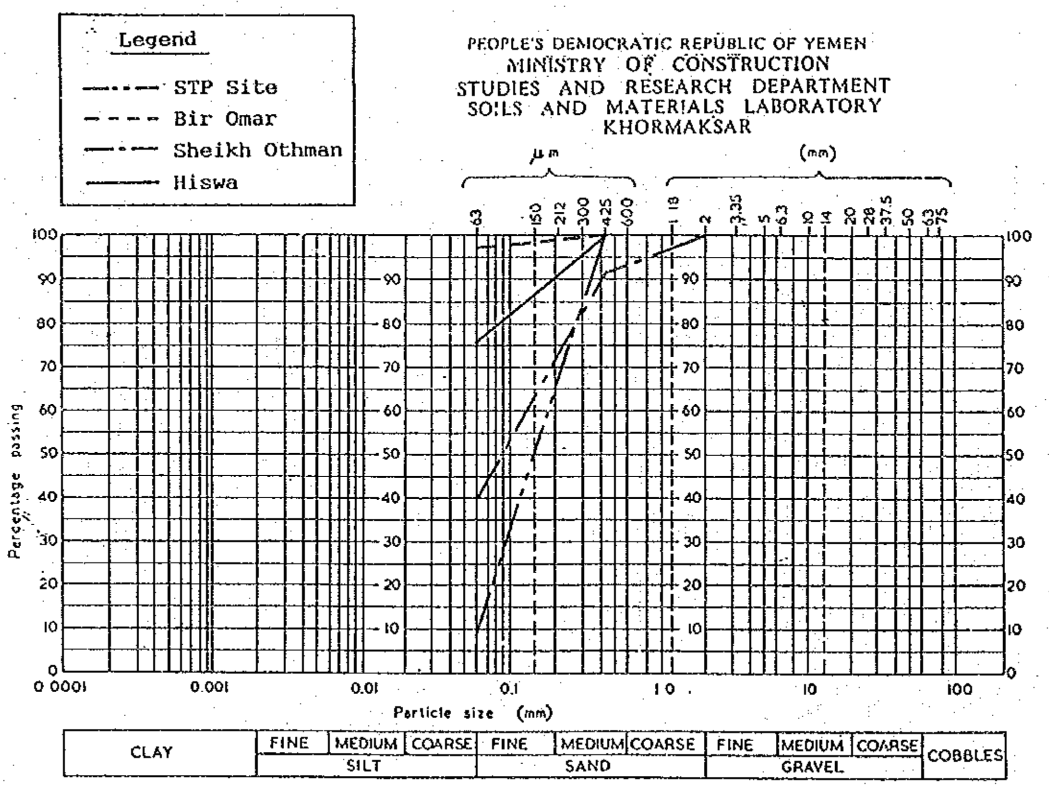


Figure J.1 Location for Soil Sampling

Table J.1 Result of Soil Test

Soil prameter / Location	Coefficient of permeability	Moisture content	Loose density
STP Site	cm/s 1.55×10^{-3}	% 6	t/m^3 1.22
Bir Omar	6.99×10^{-4}	3	1.0
Sheikh Othman	6.04×10^{-4}	3	1.17
Hiswa	4.99×10^{-4}	2	1.24
Dar Sa'ad	3.85×10^{-4}	Not Received	Not Received
Note	Data from Aden Univ.	Data from MOC	Data from MOC



Operator:

Job:

Site:

Date: 12/ 8/89

Borehole No:

Description of soil:

Sample No:

Depth of sample

British Standard test sieves

Figure J.2 Grading Accumulation Curve

Table J.2 Results of Soil Test in Japan

Sampling Point		Sheikh Othman	Dar Saad
Grain Size Properties	Gravel Fraction (≧2,000μm) %	0.0	3.5
	Sand Fraction (74~2,000μm) %	12.0	54.5
	Silt Fraction (5~74μm) %	39.5	37.0
	Clay Fraction (≦5μm) %	48.5	5.0
	Maximum Particle Size (mm)	0.84	19.1
	Uniformity Coefficient	–	8.3
	Coefficient of Curvature	–	1.2
Consistency Properties	Liquid Limit %	38.2	N.P
	Plastic Limit %	18.1	N.P
	Plasticity Index	20.1	–
Classification	Japanese Unified Soil Classification System	CL	SMg
	Triangular Soil Classification	Fine-grained Soil	Sandy Soil
Specific Gravity of Soil Particle		2.817	2.788
Natural Properties	Moisture Content %	9.4	1.6
	Wet Density g/cm²	1.732	1.664
Triaxial Compression Test	Condition of Test	UU	UU
	Cohesion kgf/cm²	0.12	0.98
	Angle of Shear Resistance	0°	10°00"
Compaction Properties	Method of Test	1·1·C	1·1·C
	Optimum Moisture Content %	20.2	16.5
	Maximum Dry Density t/m²	1.717	1.709
	(by Falling Head Permeability Test)	1.00×10^{-7}	8.00×10^{-4}

Note: UU : Unconsolidated-undrained Test

APPENDIX K

STABILIZATION POND SEWAGE TREATMENT PLANTS IN LAHEJ AND ABYAN

APPENDIX K

STABILIZATION POND SEWERAGE

TREATMENT PLANTS IN LAHEJ AND ABYAN

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APPENDIX K

STABILIZATION POND SEWERAGE TREATMENT PLANTS IN LAHEJ AND ABYAN

1. Introduction

The JICA study team made a visit to two sewage stabilization pond treatment facilities in Lahej Governorate on 29 June, 1989. Two treatment facilities are located in Sabir and Al Hota. The visit was prepared by Mr. Ahmed N. Aboteeba, head of Sanitary Engineering Section, GDLG. He accompanied the study team members to the site. Lahej Governorate engineers, including Mr. Mahadi, director also accompanied the team.

Another visit to the sewage treatment plant located in Lawdar, Abyan Governorate was made by the study team members and Mr. Aboteeba on 16 September, 1989. Stabilization pond system is adopted for the treatment. This appendix describes the present conditions, design outlines and the considerations on the three pond systems.

2. Present Conditions

Present conditions of the three pond system are as follows.

1) Sabir stabilization pond system is located in the agricultural land near Sabir town. This pond system consists of an anaerobic pond, a facultative pond and two stages of maturation ponds in series. Construction of the facilities started in October, 1988, and the construction work still continues at present. This pond system will treat the sewage from nearby Sabir town. There is no pumping station in the sewer network from Sabir town to the pond. All sewage gravitates into the pond.

   Construction work has been carried out by the Governorate. Pipes are supplied by GDLG. Construction machines owned by the Governorate have been used. During eight month period from October, 1988 to date, approximately YD 74,000 has been spent. Costs for the pipes supplied by GDLG and machines are not included in the construction cost. Labor and material costs accounted for the most of the construction cost. Total construction cost is estimated to be YD 120,000. This cost is converted to a unit cost of YD 30 per served population.

All the ponds are constructed by excavation, and water levels in the ponds are below the present ground level. Slopes of the ponds are all 1 vertical in 3 horizontal. Surfaces of slopes are protected from the bottom to the top by stone covering. Ductile cast iron pipes with 400 mm diameter are used for inlet and connecting works. Mechanical equipment installed in the system is limited to a bar screen in the inlet work. The screen is made of steel with 20 mm spaces. Flow measurement will be provided by V-notch in the inlet work. Treated effluent is planned to be reused for irrigation purpose. Pumps are needed to discharge effluent to the irrigation channel. Effluent will be mixed with groundwater before deliver to farms.

2) Al Hota stabilization pond system is located in the agricultural area in the southern suburb of Al Hota, the Governorate capital. This pond system was designed by GDLG and constructed by a Japanese contractor, Kubota Construction in 1986. The Treatment system consists of an anaerobic pond and a facultative pond. This system is treating the sewage which is collected by sewer lines constructed in the town in 1986. The present sewage flow is said to exceed the design flow, although no flow measurement has been practiced. Construction of the second process train which is identical to the existing one is considered, and diversion to the second train was provided in the inlet work.

The anaerobic pond is colored pink, but no aggressive odor was recognized. The facultative pond is colored green because of green algae. Dense population of green algae was recognized, and at the leeward corner of the pond thick scum of green algae was observed. Green algae are carried over to a long drain canal and decomposite there. Carry over of green algae presents visible nuisance in the receiving channel. A sample had been taken from the facultative pond and BOD concentration of 31.6 mg/l had been obtained suggesting proper functioning of the pond. Treated effluent is currently used by farmers voluntarily for growing lime trees, although the amount of water reused seems to be small in quantity.

3) Lawdar treatment plant is located outside the Lawdar Town which is situated at an altitude of about 1,000 m above mean sea level. One facultative pond was constructed by GDLG in 1984 as a part of integrated water supply and sanitation program prepared by UNICEF. This

stabilization pond is the first one of the kind designed for local sanitation program. Since the town is located on the gentle hill and pond is located on the outskirts of the town, sewage flows into the pond by gravity.

Although the water supply system was constructed at the same time, the wells, water sources of the system, could not yield sufficient quantity of water. The total population in the town, approximately 2,000 at present, still rely on the water brought by lorries from the other place. Thus, sewage flow to the pond is far less than expected. Sewage stagnates in a small part of the pond and no effluent goes out of the pond. The pond system does not function properly at all.

3. Design Outline

Two pond systems in Lahej Governorate were designed by GDLG engineers, and that in Abyan Governorate by UNICEF. Outline of the design are summarized below.

1) Sabir

a. Year of commission: Under construction (from Oct. 1988)
b. Design population: 4,000 in 1999
c. Design sewage flow: 480 m^3/d
d. Design temperature (ambient): 20° C
e. Size, retention time and BOD loading

	Anaerobic	Facultative	Maturation (1)	Maturation (2)
Size (LxW, m)	27.5x17.5	50 x 45.5	48.5 x 22	48.5 x 22
Area (m2)	481	2,275	1,067	1,067
Depth (m)	2.5	1.5	1.5	1.5
Volume (m^3)	1,203	3,413	1,601	1,601
R.T. (day)	2.5	7.1	3.3	3.3
Infl. BOD5 (mg/l)	375	150	44	
Effl. BOD5 (mg/l)	150	44		10
BOD Load (kg/d)	180	72	21	
V.B.L. ($kg/m^3/d$)	0.15			
A.B.L. (kg/ha/d)	3,742	316		
Coliform (/100 ml)	10^7			100
pH	6.8			

Note: Pond size is at the middle of water depth

2) Al Hota

a. Year of commission: 1986
b. Design population: 10,000 in 1994
c. Design sewage flow: 1,300 m^3/d
d. Design temperature (ambient): 20 C
e. Size, retention time and BOD loading

	Anaerobic	Facultative
Size (LxW, m)	45.5x35.5	125.1x87.1
Area (m2)	1,615	10,896
Depth (m)	2.5	1.7
Volume (m^3)	4,038	18,523
R.T. (day)	3	14
Infl. BOD5 (mg/l)	375	150
Effl. BOD5 (mg/l)	150	29
BOD Load (kg/d)	488	195
V.B.L. ($kg/m^3/d$)	0.12	
A.B.L. (kg/ha/d)	1,209	179
Coliform (/100ml)	10^6	3,038

Note: Pond size is at the middle of water depth.
Effluent BOD of 31.6 mg/l and pH value of 8 were obtained by analysis.