2. Survey on Rivers and Waterways

(1) Sampling and Analysis

Water quality of rivers and waterways in the Project Area were surveyed by the project team on December, 1976.

Sampling points are shown in Figure D-2.

Most of the samples were collected at surface, and water temperature, pH, dissolved oxygen, and electric conductivity were measured at the site by portable "water quality checker". The samples collected from the Prai were analyzed on suspended solids, chemical oxygen demand, faecal coliforms, and chlorides at the laboratory.

The analytical methods used were as follows:

Permanganate Value (PV)	:	Oxygen absorbed from Acid Permanganate, 27°C 4 hours
SS	:	Glass Fiber Filter Method
Faecal Coliforms	:	Desoxycholate Method
Chlorides	:	Silver Nitrate Titration Method
Hydrogen Sulfide	:	Filter Colorimetry by Zink Acetate

These methods are based on "Standard Method", 14th edition, 1975, APHA-AWWA-WPCF.

(2) Findings of the Survey

The results of water quality analyses are shown in Table D-1.

1) Water Temperature

As shown in Table D-1, water temperature of rivers and drains vary according to their flow condition, higher in slack waters, lower in rapid streams. The highest temperature of 34.4°C was recorded at the Butterworth A-1, (Ref. Figure D-2), during the survey on December, 1976.

As the hottest season of the State is from February to May, the annual highest temperature of drains may be more than 37°C. While the average water temperature of the Prai River was 28.4°C.

In general, water temperature of natural waters in the Project Area is suitable for bacterial activity to decompose organic load in all seasons.

2) Electric Conductivity

Electric conductivity, which is a indicator of tidal water penetration to the streams, indicates that, at flood tide, tidal water comes up to the point No. P-7, which is located at about 10 km of upstream from the river mouth. Further, tidal variation of water level is observed at the point No. P-10 which is located at more than 20 km of upstream from the river mouth. (Ref. Figure D-2)

In case of the Juru river, as the tidal gate has been constructed at the Tuan Abdul Rahaman Bridge, the sampling point No. J-2, tide water is stopped at the gate, so that electric conductivity of the upstream water from the point No. J-2 are low. (Table D-1).

3) DO

The level of dissolved oxygen is shown in Figure D-3. Zero (less than 1 mg/l.) DO concentrations are recorded in the drains of the Butterworth A-1, A, C, and D, the Derhaka river, and the Prai drains, and the tidal gate area of the Juru. This remarkable decrease of DO is due to organic loads included in domestic, industiral, and animal farm wastewaters. The colour of the lower stream of the drains is changed to blackish one, and black ooze is accumulated in the area referred above. This is the results of successional reactions, namely, organic loads, oxygen consumption, sulfate reduction, sulfide formation, and then ferous sulfide (black) accumulation. Additional formation of sulfide leads to bad smell, releasing hydrogen sulfide.

After wastewaters discharged into the rivers, the conditions are rapidly recovered by the flushing-out effects of tidal movements.

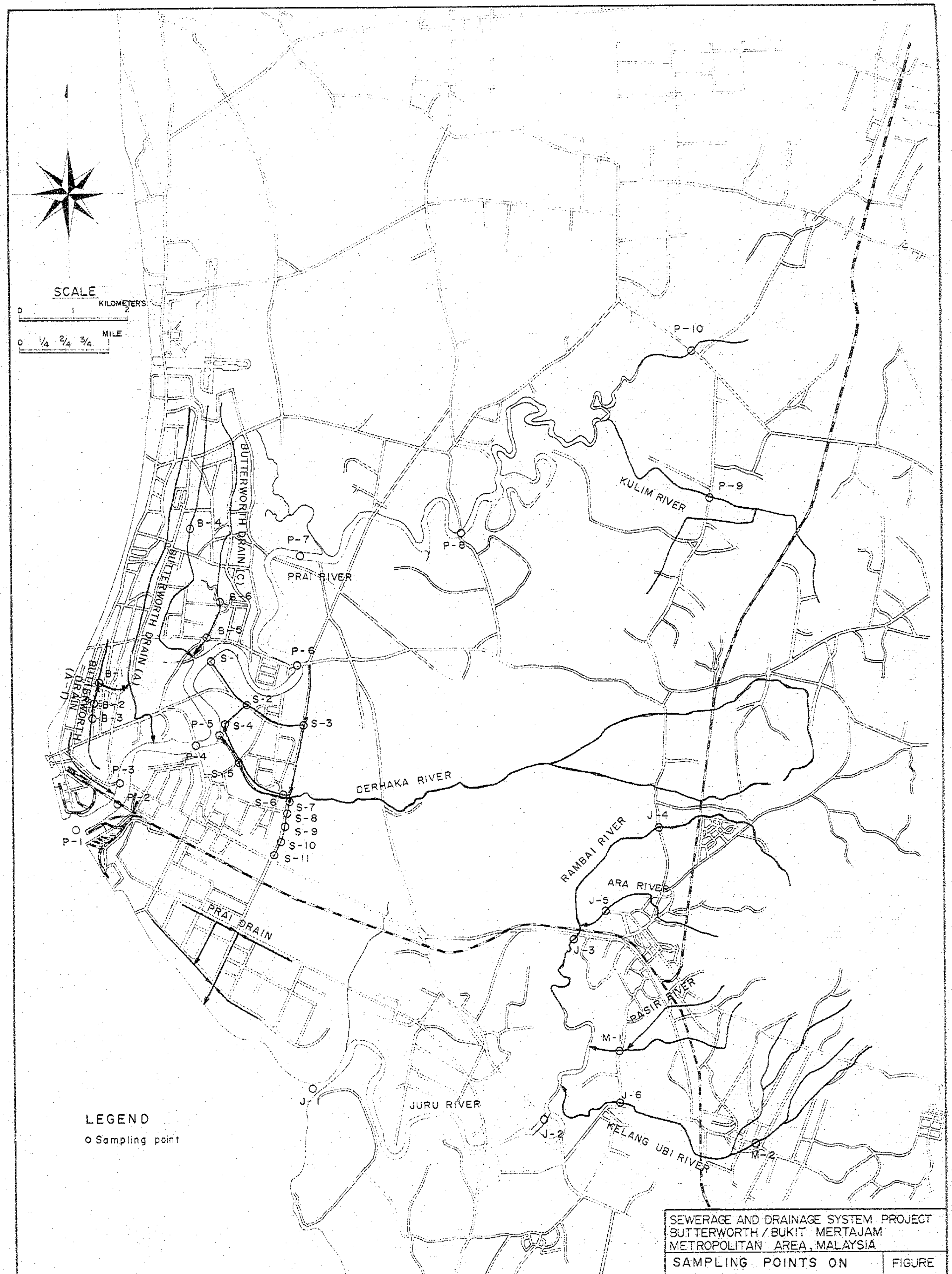
However, tidal gate of the Juru is interfering the flushing-out of the upper streams, so that accumulation of the upper stream pollution of Juru is accelerated.

The water temperature of the Seberang Jaya drains averaged about 30°C during the survey gives a DO saturation value of 7.5 mg/l. The DO content of the waters at the sampling points, S-1, 2, 3 and 11 was more than the saturation value (Table D-1). This is due to photosynthesis of aquatic plants in the slack waters of the drains, and due to scarce organic loads. If organic loads discharged into the slack waters, the DO content may be greatly decreased as in case of the points S-7 to 9. This is a shortcoming of slack water in the drains.

4) BOD & PV

According to the findings of the Juru River Pollution Survey, in spite of discharging heavily polluted waters at the upper tributaries from the tidal gate, the BOD and PV values are comparatively low. This may be explained by tidal flushing. The same effect can be expected at the Prai River because of comparatively low PV contents although the data are very limited (Table D-1).

FIGURE D-2



SEWERAGE AND DRAINAGE SYSTEM PROJECT
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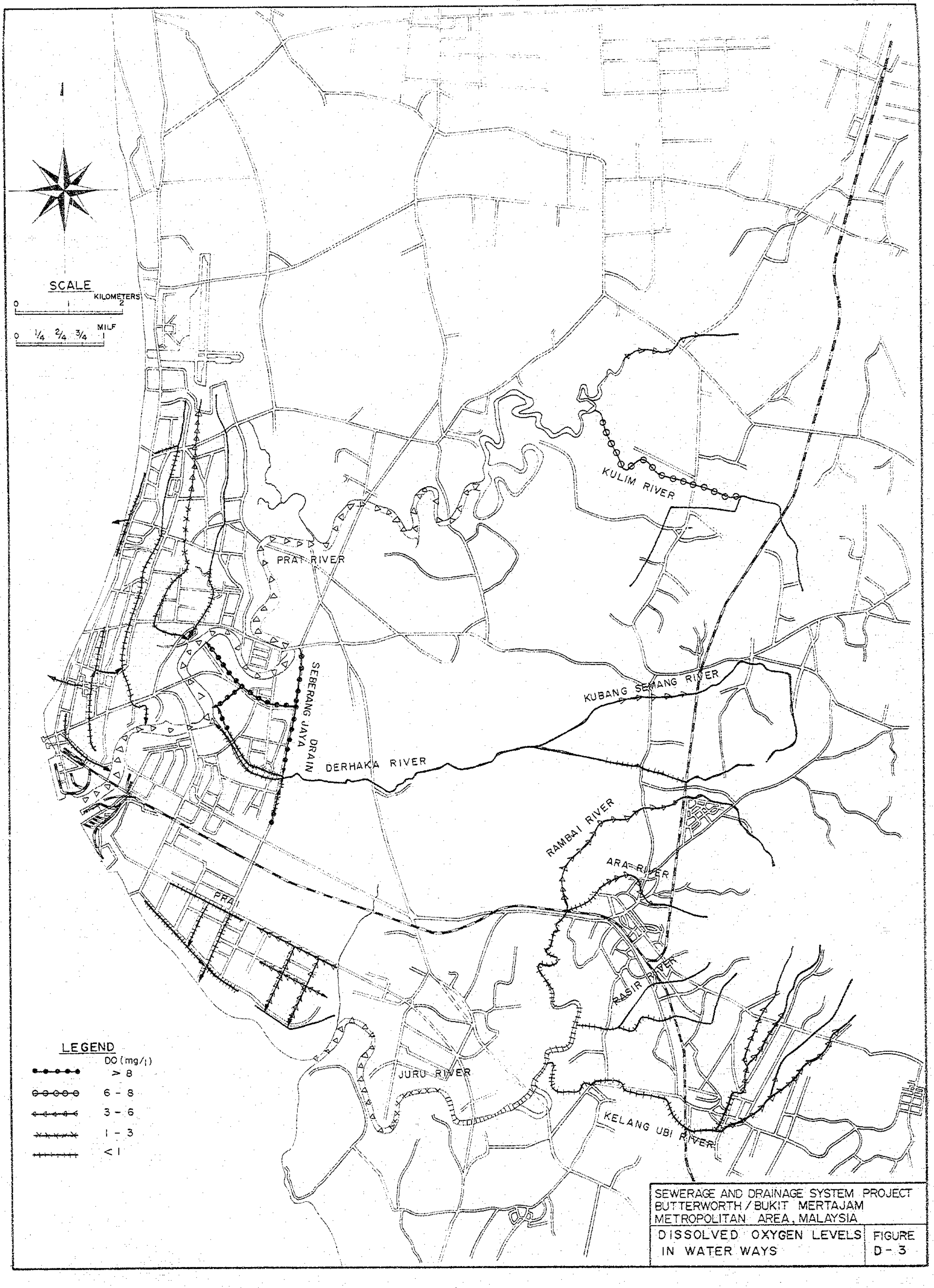
METROPOLITAN AREA, MALAYSIA

SAMPLING POINTS ON WATERWAYS SURVEY

FIGURE D - 2

FIGURE D-3


SCALE
KILOMETERS
0 1 2
MILF
0 1/4 2/4 3/4 1
KULIM RIVER
PRAI RIVER
SEBERANG JAYA
DRAIN
KUBANG SEMANG RIVER
DERHAKA RIVER
RAMBAI RIVER
ARA RIVER
PRAI
PASIR RIVER
JURU RIVER
KELANG UBI RIVER
LEGEND
DO (mg/l)
> 8
6 - 8
3 - 6
1 - 3
< 1
SEWERAGE AND DRAINAGE SYSTEM PROJECT
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METROPOLITAN AREA, MALAYSIA
DISSOLVED OXYGEN LEVELS
IN WATER WAYS
FIGURE
D - 3

TABLE D-1 Water Quality of Rivers and Drains in the Project Area

No. of Station	Date/Time	WT	pH	EC	DO	PV	SS	Cl^-	Coli-forms
		°C		mʊ/cm	mg/l	mg/l	mg/l	mg/l	N/ml
P-1*	16 Dec.13:00	28.6	7.3	32.5	5.2	3.4	15	9500	-
P-1**	17 Dec. 7:45	27.1	7.0	41.9	6.0	1.8	8	14700	-
P-2**	9 Dec.16:20	28.9	8.3	47.2	8.4	-	-	-	-
P-3*	3 Dec.14:20	32.2	6.0	41.3	5.3	8	303	12190	138
P-4**	9 Dec.15:55	29.3	8.2	45.1	8.7	-	-	-	-
P-5**	4 Dec. 9:57	27.7	6.9	28.2	4.2	-	-	-	-
P-6**	4 Dec.10:10	27.7	6.7	22.6	3.9	3	15	7160	561
P-6**	9 Dec.14:00	28.4	7.6	32.2	4.4	-	-	-	-
P-7**	9 Dec.15:00	29.2	7.7	27.1	5.2	-	-	-	-
P-8**	4 Dec.11:00	27.2	6.8	0.1	3.8	2	20	222	25
P-9**	4 Dec.11:40	25.8	7.2	0.4	7.5	1	21	9	110
P-10**	4 Dec.11:45	28.2	6.7	0.4	4.3	1	11	8	43
J-1*	5 Dec. 9:20	27.4	6.6	37.2	3.6	-	-	-	-
J-1**	17 Dec. 8:44	26.7	7.8	43.6	6.9	3.3	62	15600	15
J-2*	5 Dec. 8:50	27.9	6.2	3.8	0.5	-	-	-	-
J-3**	5 Dec.12:30	28.4	6.5	0.1	0.9	-	-	-	-
J-4**	5 Dec.12:55	26.6	6.6	0.2	2.0	-	-	-	-
J-5**	5 Dec.11:35	30.1	5.8	0.5	0.3	-	-	-	-
J-6**	5 Dec.12:40	27.7	6.5	0.2	0.7	-	-	-	-

* low tide

** high tide

TABLE D-1 Water Quality of Rivers and Drains in the Project Area (Continued)

No. of Station	Date/Time	WT	pH	EC	DO	PV	SS	Cl^-	Coli-forms
		°C			mg/l	mg/l	mg/l	mg/l	N/ml
B-1	7 Dec.14:30	33.4	5.8	0.6	0.9	-	-	-	-
B-2	7 Dec.14:45	34.4	6.6	0.6	0.7	-	-	-	-
B-3	7 Dec.15:00	33.4	6.7	0.6	0.7	-	-	-	-
B-4	14 Dec.18:30	28.8	7.0	0.5	1.1	-	-	-	-
B-5	3 Dec.15:00	31.3	6.2	4.8	0.1	-	-	-	-
B-6	3 Dec.15:10	33.7	6.2	0.5	0.5	-	-	-	-
S-1	3 Dec.15:50	30.7	6.4	8.1	11.5	-	-	-	-
S-2	3 Dec.15:45	30.7	6.4	2.6	12.9	-	-	-	-
S-3	3 Dec.14:40	31.8	6.6	0.2	11.7	-	-	-	-
S-4	3 Dec.15:58	28.5	6.6	0.2	0.2	-	-	-	-
S-5	3 Dec.16:10	28.2	6.2	0.7	0.5	-	-	-	-
S-6	3 Dec.16:15	31.4	6.1	0.1	0.5	-	-	-	-
S-7	3 Dec.16:40	30.5	7.8	0.6	0.9	-	-	-	-
S-8	3 Dec.16:35	27.7	7.8	0.1	2.3	-	-	-	-
S-9	3 Dec.16:30	32.0	7.9	0.4	0.7	-	-	-	-
S-10	3 Dec.16:32	32.3	7.9	0.3	1.8	-	-	-	-
S-11	3 Dec.16:25	33.5	8.5	0.6	21.9	-	-	-	-
M-1	5 Dec.11:40	28.4	6.5	0.3	0.7	-	-	-	-
M-2	5 Dec.10:00	24.7	7.3	0.3	2.3	-	-	-	-

-: not measured

3. Survey on Sea Water Quality

(1) Sampling and Analysis

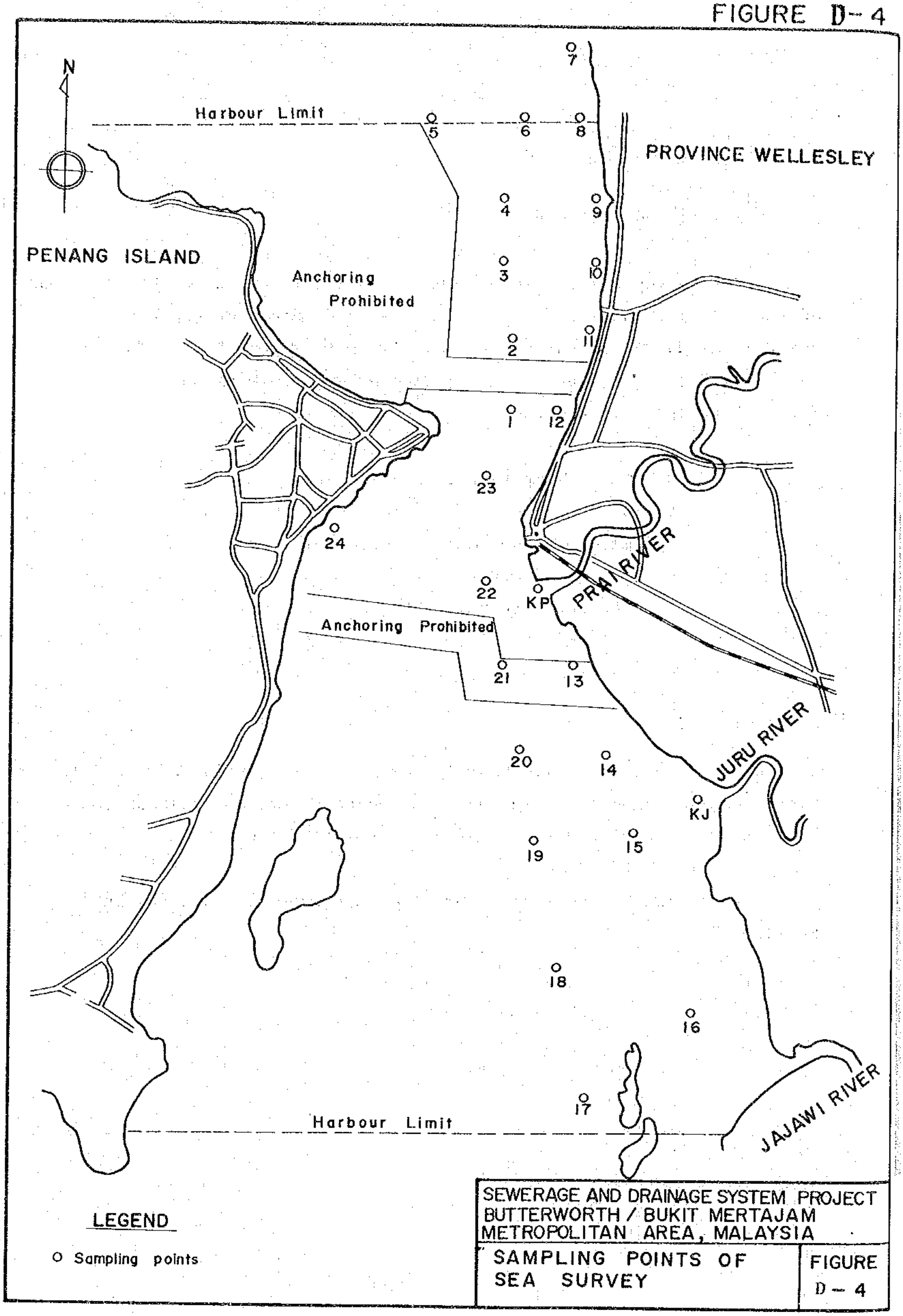
Sea survey was carried out on December, 1976, along the east coast of the Penang Channel extending from the river mouth of the Prai to about 13 km south and north respectively (Ref. Figure D-4). The sea water samples were taken at the surface at 26 points as shown in Figure D-4, and analyzed for;

a. Water Temperature (WT)
b. pH
c. Electric Conductivity (EC)
d. Dissolved Oxygen (DO)
e. Permanganate Value (PV)
f. Chlorinity (Cl^-)
g. Suspended Solids (SS)
h. Faecal Coliform (Coliform)

WT, pH, EC, and DO were immediately tested after sampling by portable "water quality checker". The other components were analyzed at the laboratory as soon as possible after they were brought back.

Analytical methods used were the same ones as mentioned in Section 2.(1).

FIGURE D-4



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SAMPLING POINTS OF SEA SURVEY

FIGURE D-4

(2) Findings of the Survey

The results of water quality analyses are shown in Table D-2.

1) Water Temperature

The surface water temperatures obtained during the survey were 26.5 to 28.6°C, and rose in the morning in company with the sun rise. The diurnal variation of the surface water temperature may be more than 2.5°C. This diurnal temperature variation and strong tidal currents mentioned above accelerate vertical mixing of sea water.

2) Electric Conductivity and Chlorinity

Electric conductivity and chlorinity are indices of penetration of fresh water into the sea. The distribution of electric conductivity (Figure D-5) shows the fresh water feather of the Pri river run-off at ebb tide. The low values of the sampling points 8, 13, 15, 16, 22, KP, and KJ shown in Table D-2, are attributable to the dilution of sea water by river waters.

3) DO and PV

The values of DO and PV as shown in Table D-2 indicate that the water of the Penang Channel has not yet been heavily polluted by organic pollutants, but, near the river mouths, the water qualities are slightly unfavourable.

TABLE D-2 The Results of Seawater Analysis (The Penang Channel)

No. of Station	WT	pH	EC	Chlorinity (Cl^-)	DO	PV	SS	Coliforms
	°C		m℧/cm	o/oo	mg/l	mg/l	mg/l	N/ml
1	27.1	7.7	47.7	18.4	6.8	1.6	3	5
2	27.3	7.9	46.7	18.4	6.8	1.3	14	18
3	27.6	8.0	47.9	17.0	6.8	1.2	9	2
4	27.5	8.0	47.8	17.3	6.8	1.1	5	0
5	27.4	8.0	48.0	17.2	7.0	1.4	12	0
6	27.6	8.0	47.8	17.3	7.5	1.3	3	2
7	27.7	7.9	48.3	17.2	7.3	2.0	20	0
8	27.9	7.8	43.9	14.6	6.9	2.7	21	0
9	28.3	8.0	49.2	17.3	6.9	1.4	14	0
10	28.6	8.0	49.1	17.2	7.1	1.8	27	1
11	28.6	8.0	49.0	17.2	7.3	1.8	55	0
12	28.2	7.9	48.8	-	6.3	-	-	-
13	26.5	7.9	46.8	17.3	6.8	1.5	14	1
14	26.7	8.0	47.7	17.3	7.3	1.5	11	4
15	27.1	7.5	46.3	17.1	7.5	1.5	24	0
16	26.4	8.1	46.1	17.1	7.2	1.6	12	0
17	27.0	8.0	48.0	17.3	7.5	1.0	5	0
18	27.3	8.0	47.7	17.3	7.7	1.3	16	0
19	27.3	7.9	47.7	17.2	7.2	1.3	3	1
20	27.5	7.9	47.7	17.4	6.8	1.0	8	4
21	27.6	7.9	47.5	17.2	7.9	1.1	5	7
22	28.6	8.3	47.7	17.8	7.3	0.8	6	0

TABLE D-2 The Results of Seawater Analyses (The Penang Channel) (Continued)

No. of Station	WT	pH	EC	Chlorinity (Cl^-)	DO	PV	SS	Coliforms
	°C		m℧/cm	o/oo	mg/l	mg/l	mg/l	N/ml
23	28.2	8.3	47.8	17.4	7.5	1.2	3	2
24	28.2	8.3	48.2	17.3	7.3	1.4	12	365
KP	28.6	7.3	32.5	9.5	5.2	3.4	15	125
22	28.4	7.7	38.2	12.5	5.5	3.2	8	145
KP	27.1	7.0	41.9	14.7	6.0	1.8	8	20
KP*	26.2	7.5	43.1	17.0	6.8	2.4	114	305
KJ	26.7	7.8	43.6	15.6	6.9	3.3	19	15
KJ**	25.7	7.8	41.2	16.5	6.1	2.3	62	30

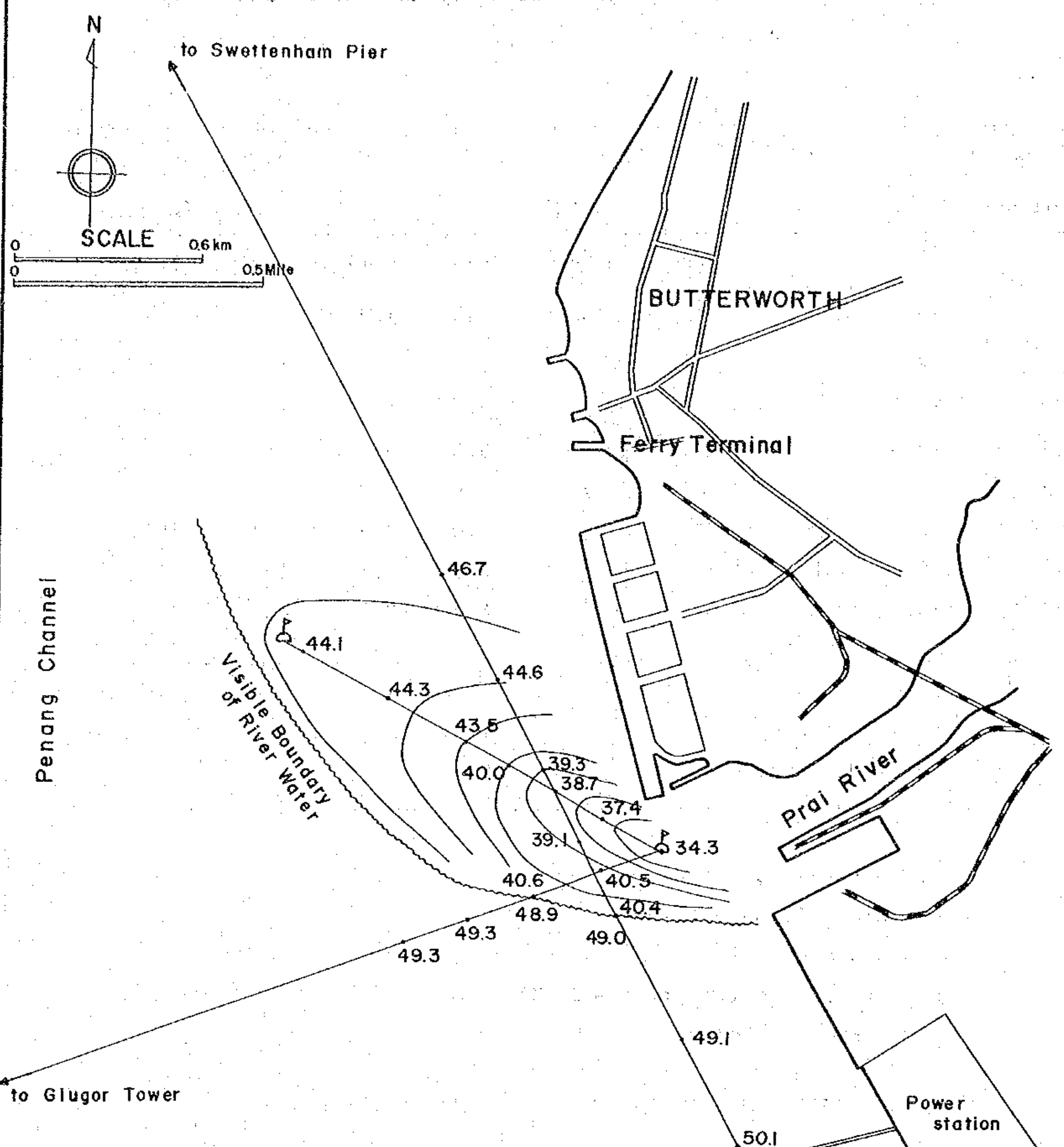
* 2.7 m deep
** 5.0 m deep

WT Water Temperature
EC Electric Conductivity
DO Dissolved Oxygen
PV Oxygen Absorbed from Acid Manganate
SS Suspended Solids

Notes:

The Surveys were carried out on 16th and 17th December, 1976.
All samples were collected at the surface except marked ones.

FIGURE D - 5



LEGEND

44.1 Sampling point and figures of electric conductivity in mʊ/cm

Note: Sampling date
Dec. 16, 1976
(13:00 ~ 15:00)

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FRESHWATER RUNOFF FROM PRAI RIVER AT EBB TIDE

FIGURE D - 5

4) Suspended Solids and Floating Matter

The results of analysis show that the concentration of SS is lower at the offshore than the near shore. This may be due to vertical mixing of shallow bottom material by strong tidal currents and waves. The sub-surface waters at both of the river mouths, the Prai and the Juru, keep higher SS concentration than the surface waters. This may be also due to turbulence of tidal movements.

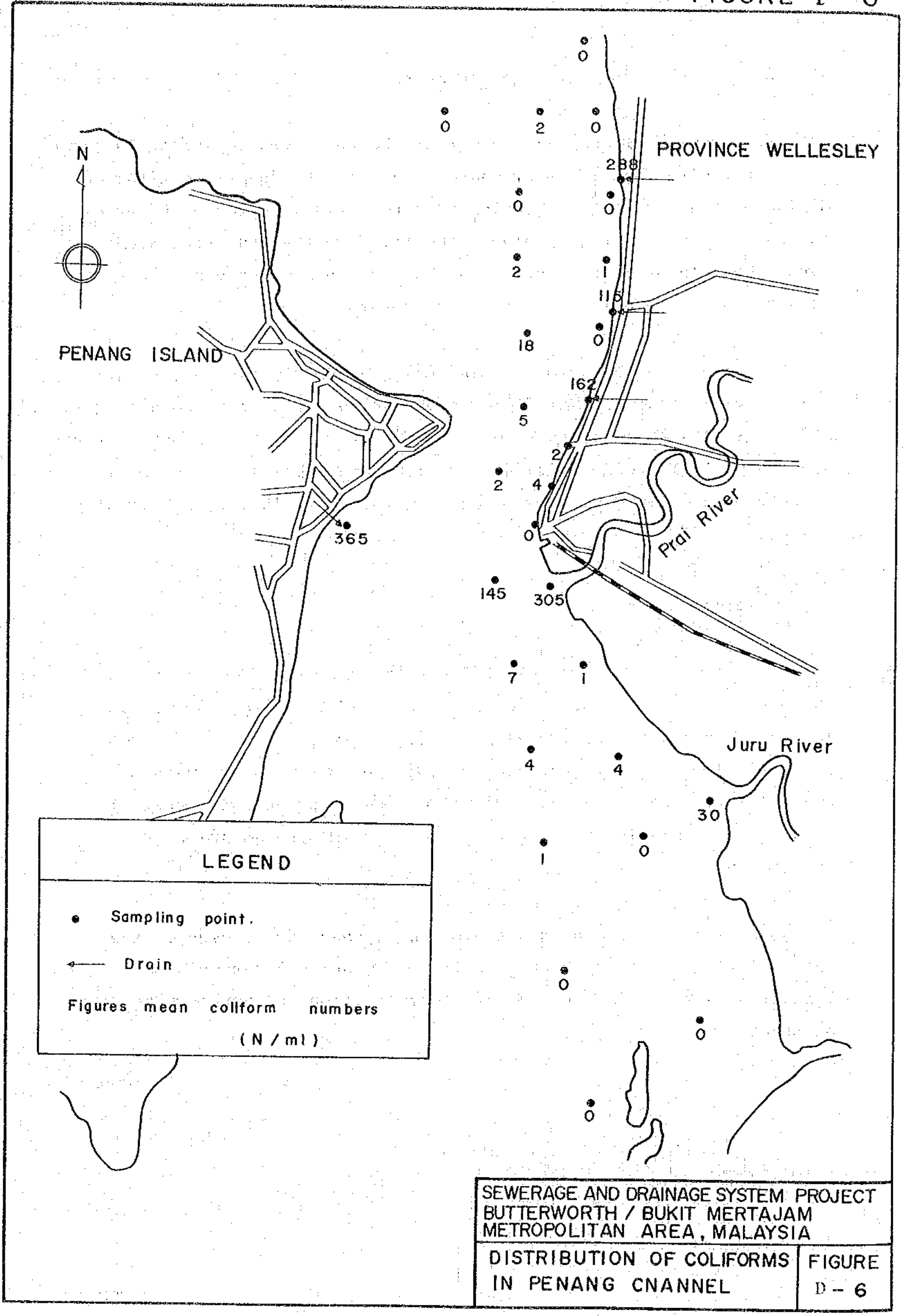
As the whole sea area surveyed are included in the Penang Port Area, many floating matters were found which included plastics, wood fragments, and other floatables desposed of from the ships, or flowed out of rivers and drains from George Town, Butterworth, and other town areas. These floatables are found up to the sampling points 5, 18 and further along the current lines.

5) Coliforms

In spite of discharged wastewaters from George Town, Butterworth, and other town area, the concentration of coliforms was comparatively low in the sea water except near the river mouth of the Prai and at the outfall point of the sewage from George Town. The results of coastal water survey on coliforms also shows that coliform contamination appears only in the water near sewer discharge as shown in Figure D-6.

As the coastal area in Butterworth is used for bathing by the people living along the beach, coliform contamination should be eliminated down to the permissible concentration for bathing and other recreation.

FIGURE D-6



N
PENANG ISLAND
PROVINCE WELLESLEY
Prai River
Juru River
0
0
2
0
288
0
0
2
1
115
18
0
162
5
2
2
4
365
0
145
305
7
1
4
4
30
1
0
0
0
0
LEGEND
Sampling point.
Drain
Figures mean coliform numbers (N/ml)
SEWERAGE AND DRAINAGE SYSTEM PROJECT
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METROPOLITAN AREA, MALAYSIA
DISTRIBUTION OF COLIFORMS IN PENANG CNANNEL
FIGURE D-6

The WHO criteria[1/] are based on faecal coliforms, and suggest a limit of 0.5 cells/ml for a satisfactory marine bathing water, and consider that faecal coliform concentrations between 0.5 to 2 cells/ml is considered as slight pollution, between 10 to 20 cells/ml as distinct pollution, and more than 20 cells/ml as heavy pollution.

California standard (10 to 100 total cells/ml), while effective, seems to be too conservative, and that the Brazil standard of 100 total cells/ml may be realistic to use in developing countries including Malaysia, with limited financial resources.[2/]

(6) Miscellaneous

1) Plankton

Planktonic blooming often appears along the shore in the Penang Channel. Redish motile type plankton is dominant in the bloom, and gathers in the surface thin layer. The patch of the bloom are comparatively small, approximately 1 to 2 km long, 0.5 to 1 km wide, and the chlorophyll content, which is a good index of the standing crop of phytoplankton of the water, was not so high, 0.4 mg/cu m at the time of survey on December 1976.

2) Bottom Sediment

The bottom sediment of the Penang Channel were observed at the same time when water quality survey was carried out. The bottom sediment in front of the Prai Industrial Complex is silty mud, which might have been accumulated by tidal currents for a long time. This silty mud is grayish, and does not include black organic ooze and/or coalblack mud, but has slight smell of hydrogen sulfide. The sediments of the river mouths of the Prai and the Juru are also grayish silty

---

1/ Document EVRO 3125/(1), 1974, by WHO working group.

2/ "Criteria for Marine Waste Disposal in Southeast Asia", by H.F. Ludwig (1973) clay, and not highly polluted in spite of receiving coalblack waters and muds from their tributatries. This may be due to the effects of tidal washing out. The most part of the Butterworth beach is sandy.

APPENDIX E

DESIGN DATA

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CHAPTER 1

FUNCTIONAL DESIGN DATA

1.1 Summary

In this chapter, the design basis necessary to design sanitary and storm water conduits, covering flow friction formulae, sizes of structures of facilities, hydraulics of sewers, materials of facilities, and measures needed for control of sulfides have been studied and criteria developed for this Project as summarized below:

(1) The Manning formula should be used for design of pipes and channels.

(2) No public sanitary sewer shall be less than 22.5 cm (9 in.) in diameter.

(3) Earth covering of public sewers should not be less than one meter unless special protection measures against the expected load are provided.

(4) All sanitary sewers shall be so designed and constructed to give mean velocities when flowing full or half-full of not less than 60 cm/sec for VCP, based on the Manning formula with 'n' value of 0.013. For RCP or any cement-bonded pipe materials, using an 'n' value of 0.013, the minimum flow velocity should be 75 cm/sec (2.5 ft/sec).

(5) For storm sewers the velocity of flow should be not less than 80 cm/sec (2.6 ft/sec).

(6) For sanitary sewers, full pipe capacity of the design peak flow rate should be provided.

(7) Minimum sewer slopes for different sewer pipe sizes are recommended, so that in no case, the velocity of flow will be less than 75 cm/sec for concrete pipe, and 60 cm/sec for VCP.

(8) Sewers should generally be laid with straight alignment between manholes. Exceptions should be allowed only when there is assurance the available cleaning methods will be workable in the curved sections.

(9) When a smaller sewer joints a larger sewer, the crown of both sewers should be placed at the same elevation.

(10) Sanitary sewers of smaller size up to 300 mm (12 in.) in diameter should normally be of vitrified clay. For larger size sewers up to 1,800 mm (70 in.) in diameter, centrifugally cast reinforced concrete pipes, conforming to internationally accepted standards, should be used.

(11) Joints of concrete pipe should be the rubber-gasket type, and factory applied 'push-fit' resilient type joints should be used for vitrified clay pipes.

(12) Manhole spacing should not be more than 200 meters (656 ft).

1.2 Design Factors

In determining the required capacities of sanitary and storm sewers the following factors should be considered:

(1) Sanitary Sewers

1) Peak flow rate of domestic sewage.
2) Additional maximum sewage or waste flow from industrial plants.
3) Ground water infiltration.
4) Depth of excavation.
5) Location of treatment plant.
6) Pumping requirements.
7) Design velocities needed to assure self-cleansing and prevention of sulfide buildup.

(2) Storm Sewers

1) Peak storm water runoff for the designed return period of rainfall.

2) Topography of area.
3) Condition of rivers.
4) Pumping requirements.

1.3 Flow Friction Formulae

For determining sewer capacities, a wide variety of equations have been developed. Among the equations widely used are:

(1) The Chezy and Darcy-Weisbach equations
(2) The Manning equation
(3) The Kutter equation, and
(4) The Hazen-Williams equation.

The Kutter and the Manning equations are most widely used for pipes and conduits of all shapes, flowing either full or partly full. Although the use of the Kutter equation has been extensive and the graphs and tables for the equation are available, its popularity is declining because of its empirical and cumbersome nature. The Manning equation tends to be used very extensively, because of its simplicity and because the "n" value is essentially the same as used in Kutter's equation.

A comparison was made between the velocities of circular pipes calculated by means of three different equations namely; Kutter, Manning, and Hazen-Williams. The velocities for full flow in sewer pipes from 225 mm to 1,800 mm in diameter were calculated using a friction coefficient 'n' value of 0.013 for the Kutter and Manning equations, and a 'C' value of 110 for Hazen-Williams which corresponds to 'n' value of 0.013.

As shown in Table E-1 , the results of the calculations indicate that the velocities given by the three equations are essentially the same, but with some minor variations. In smaller sewers the Kutter's equation gives the lowest values, but the values become practically the same as the sewer size increases, and the order is then reversed for the larger sewer pipes. It is not possible to judge the adaptability of the equations by such calculations; however, it is clear that Manning's equation gives intermediate values, hence appears to be the best choice for general application and has been adopted for use on this project.

TABLE E-1 Comparison of Flow Velocities in Pipes Calculated by Different Formula (meter/second)

Pipe dia. and slope	(i)	Kutter (n=0.013)	Manning (n=0.013)	Hazen-Williams (C=110)
225 mm	0.0045	0.700	0.758	0.824
300	0.0035	0.770	0.809	0.862
375	0.0026	0.784	0.809	0.845
450	0.0022	0.825	0.841	0.866
525	0.0018	0.835	0.843	0.857
600	0.0016	0.866	0.869	0.874
675	0.0014	0.881	0.879	0.876
750	0.0013	0.914	0.909	0.899
900	0.0011	0.955	0.944	0.922
1,050	0.0009	0.960	0.946	0.912
1,200	0.0008	0.991	0.975	0.931
1,350	0.0007	1.004	0.987	0.932
1,500	0.0007	1.078	1.058	0.996
1,800	0.0007	1.218	1.195	1.118
2,000	0.0007	1.306	1.239	1.194

Note: (i) Recommended minimum slopes for sanitary sewers.

In view of these facts the Manning equation is recommended for the design of sewers and channels. The equation is expressed as;

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

where: n = coefficient of roughness

R = hydraulic radius, m

S = slope

Care must be used in selecting the friction coefficient. In general, 'n' values from 0.013 to 0.015 are used in sewer design, depending upon the type of joint and the pipe material. Table E-2 is a summary of friction coefficients for different sewer materials for use with the Manning formula.

TABLE E-2 Values of 'n' to be used with the Manning Equation

Conduit Materials	Manning 'n' Value
1) Closed Conduits	
Asbestos-cement pipe	0.010 - 0.015
Brick	0.013 - 0.017
Cast iron pipe	
Uncoated (new)	
Cement-lined & seal coated	0.011 - 0.015
Concrete (monolithic)	
Smooth forms	0.012 - 0.014
Rough forms	0.015 - 0.017
Concrete pipe	0.011 - 0.015
Plastic pipe (smooth)	0.011 - 0.015
Vitrified clay pipes	0.011 - 0.015
2) Open Channels	
Lined channels	
Brick	0.012 - 0.018
Concrete	0.011 - 0.020
Vegetal	0.030 - 0.040
Excavated or dredged	
Earth, straight and uniform	0.020 - 0.030
Earth, winding, fairly uniform	0.025 - 0.040
Rock	0.030 - 0.045
Unmaintained	0.050 - 0.140
Natural channels (minor streams, top width at flood stage 100 ft)	
Fairly regular section	0.030 - 0.070
Irregular section with pools	0.040 - 0.100

Data Source: WPCF Design Mannual of Practice No. 9 (1970)

Factors which affect the choice of a coefficient are conduit material, Reynolds number, size and shape of conduit, and depth of flow. In addition to these interrelated factors the following should be considered;

(a) Rough, opened, or offset joints,
(b) Poor alignment and grade due to settlement or lateral soil movement.
(c) Deposits in sewers
(d) Amount and size of solids being transported,
(e) Coatings of grease or other matter on interior of sewer,
(f) Tree roots, joint compounds, and mortar dams resulting from poor or deteriorated jointing and other protrusions, and
(g) Flow from laterals disruption flow in the sewer.

The values are commonly used for sewer design and hence are higher than the values obtained in laboratory tests with clear water and clean conduits. The range in coefficients for a given pipe material is explained partially by the disturbing influences mentioned previously in the general discussion of coefficients.

It is recommended the Manning's 'n' of 0.013 be used for all proposed and future sewer and 0.015 be used for all existing sewers. Higher values of 'n' should be used for existing sewers if available data indicate deterioration, deposits, or inferior workmanship.

The 'n' value of 0.013 for proposed and future sewer is based on the use of pipe units having not less than 5-ft laying lengths, with true and smooth inside surfaces, and on the assumption that only first-class construction procedures will be followed.

1.4 Sewer Design and Construction

1.4.1 Minimum Size of Sewer

The adoption of a minimum size of sewer is necessary, because experience has shown that comparatively large objects, such as scrub bushes, and also tree roots, sometimes get into sewers and that stoppage resulting from them as much less likely if sewers are not smaller than 22.5 cm (9 in). Smaller pipes experience more frequent troubles in cleaning of settled debris, roots, etc., especially where slopes are flat.

Another factor determining the minimum size of pipe is construction cost, which may be greatly affected by topographical conditions. Where the ground surface slope in the area is flat, ranging between 0.1 and 0.3 meters per thousand meters, sewer must be deeper. Consequently, the construction cost will also be increased. For example, to keep the velocity of flow more than 75 cm/sec in a 22.5 cm pipe, the slope must be 0.0045, but for a 15 cm pipe the slope would be 0.0076 for the same velocity of flow, and the difference of depth will be 3.1 meters per one km of sewer length. Hence, the construction cost for 15 cm (6 in.) pipes would hardly be cheaper than 22.5 cm (9 in.) pipes, because the increased cost of excavation will overcome the reduced cost to be gained by the use of smaller pipes. For these reasons, the minimum size of sanitary sewers for this project, except house connection, should be 22.5 cm (9 in.)

For house connections, smaller sizes may be used; however, house connection pipes should be larger than the building sewers, so that articles which pass through the building sewers may readily pass through the building connection pipes. Experience shows that a diameter of more than 15 cm is usually satisfactory for house connection pipes, except for large buildings which have terminal pipes of more than 15 cm in diameter.

1.4.2 Minimum Depth of Sewer

Enough earth covering should be left between the top of the sewer and paved surfaces to protect the sewers from traffic loads and to avoid undue interference with other underground facilities. The minimum allowable cover may depend on the size of pipe, soil conditions, pavement and traffic loads.

The calculation indicates that for one meter of earth covering under a 20 ton truck load, pipes laid on continuous concrete cradle bedding will be capable of supporting the load. It was concluded that it is reasonable to use at least one meter of earth covering for sewer pipe in the Project Area.

Another factor to be considered in deciding the required earth covering for public sewer pipes, is the length and slope of private sewers to be connected. Where the private sewers are deep, it may be more economical pump from the buildings than to lower the public sewers to such depths. Deeper house sewers may be caused either because of low ground elevation or because the houses are located far from the street.

An estimation was made for new developed housing area, to check the depth of private sewer pipes. At the representative house, with a plot of 30 meters of frontage and depth, assuming an average slope of pipe at 2 percent and minimum earth covering at the starting point of the sewer as 30 cm, the minimum earth covering of the public sewer would be one meter to receive the sewage from the house by gravity.

In view of the above mentioned results, it is recommended that the earth covering of public sewers be not less than one meter except for specific situations where studies show that shallower depths are feasible.

1.4.3 Velocity of Flow

(1) Minimum Velocity

Sewage should flow at all times, with sufficient velocity to prevent settlement of solid matter and consequent loss of sewer capacity. This is particularly important in the Project Area because of the flat slopes. The most significant factors to be considered are discussed below:

1) The commonly accepted minimum velocity for self-cleansing of sanitary sewers is 60 cm/sec. A velocity of 60 cm/sec can prevent most deposits of solids in sewers.

2) Ground surface slopes in the area except in the part of Bukit Mertajam District generally range between 0.01 and 0.03 percent. Sewer slopes are generally steeper than the ground surface slopes and sewers will become deeper, and costs for construction will be significantly increased in higher minimum velocities are used. A minimum slope for 225 mm (9 in.) sewer pipe to give a flow velocity of 60 cm/sec is 0.30 percent, based on an 'n' value of 0.013, but for 75 cm/sec, 0.45 percent is necessary. In case of a ground surface slope of 0.03 percent, the difference of construction cost between two different velocities may be M$40,000/km of pipe length.

3) An important consideration is selecting the design flow velocities for sanitary sewers in regions of hot climate, including tropical areas like Malaysia, is the problem of sulfide generation because of the high temperatures. This is especially important where concrete or other cement-bonded pipe is used as the sewer material, because unless controlled the sulfides will attack and dissolve the cement which binds the pipes material together, so that sooner or later the pipe may be suffer structural failure. Experience with this problem in other countries has shown that the most effective method of sulfide control is to use a design velocity

at average flow not less than 75 cm/sec, and preferably more (Refer Annex). At velocities of 75 cm/sec or more sulfide generation will be avoided in the sewer. For purposes of final design more precise methods should be used for evaluating the sulfide hazard (which is a function of BOD and temperature as well as flow velocity) on a case by case basis, but the general rule noted above should be sufficient for master planning. Another solution to the sulfide problem, where concrete or other corrodable materials are used, is to protect the pipe with suitable lining or coating.

4) For storm water, a higher velocity is preferable, because stormwater generally contains heavier solids such as larger sand, and soil for which a higher cleansing velocity is necessary. For open channels, a flatter slope may be allowed where necessary, because it is comparatively inexpensive to remove silt deposits from open channels.

In view of the above mentioned comments, the following criteria are recommended:

All sanitary sewers shall be so designed and constructed to give mean velocities when flowing full or half-full of not less than 60 cm/sec for VCP, based on the Manning formula using an 'n' value of 0.013. For RCP or any cement-bonded pipe materials, using an 'n' value of 0.013, the minimum design flow velocity should be 75 cm/sec and suitable lining or coating pipes should be used.

In storm sewers, the velocity shall not be less than 80 cm/sec. For open channels, where ground surface slopes are comparatively flat, a velocity of 30 cm/sec may be allowed if removal of deposits is easy and inexpensive.

(2) Maximum Velocity

The maximum velocity should not exceed 3.0 m/sec, to protect sewer erosion. Where the ground surface slope is steep and velocities of more than 3.0 m/sec may result, special provision should be made to protect against displacement by erosion and shock.

1.4.4 Design Depth of Flow

Average temperature in Penang State is around 27°C and the sewage temperature will also be high, hence fresh sewage tends to rapidly become anaerobic and to generate sulfides. As noted in the previous descussions, among the measures available for solving sulfide problems, it is believed the effective method for use in the Project Area is to use flow velocities to prevent sulfide buildup or to use suitable lining or coating pipes.

The survey on sewage fluctuation in selected representative districts indicated (Ref. Appendix F) that peak flows usually occur around 8:00 a.m. and 5:00 p.m., each lasting about one hour. The rest of the day, the sewage flow rate is less than the peak rate, therefore, if the sewer pipe is designed on the basis of 100 percent of the design peak flow, there will be some space above the water surface elevation in the pipe most of the day.

Considering the above mentioned conditions, all circular pipes are recommended to be designed on the basis of 100 percent full capacity.

1.4.5 Peak Flow Rate

Peak flow is the instantaneous maximum rate that can be expected. Peak flow is usually obtained by applying a multiplying factor to average flows. Small contributing populations give large factors while large populations give smaller factors.

The pattern of use and experience in Malaysia is comparable to most of the other countries, where the Babbit formula

$$M = \frac{5}{P^{1/5}}$$ is widely accepted.

However, in George Town Sewerage Study, the formula $M = \frac{5}{P^{1/7}}$ is used.

where, M : Peak to average ratio (peaking factor)

P : Design Population in thousand

In view of the fact that conditions in the Project Area under consideration is similar to those in George Town, it is recommended that the equation of George Town be used as peak flow rates formula in the Project Area. Therefore, sanitary design flow is expressed as the formula;

$$Q = P' \times q \times M$$

where, Q : sanitary sewer design flow, cu m/day

P' : population contributed

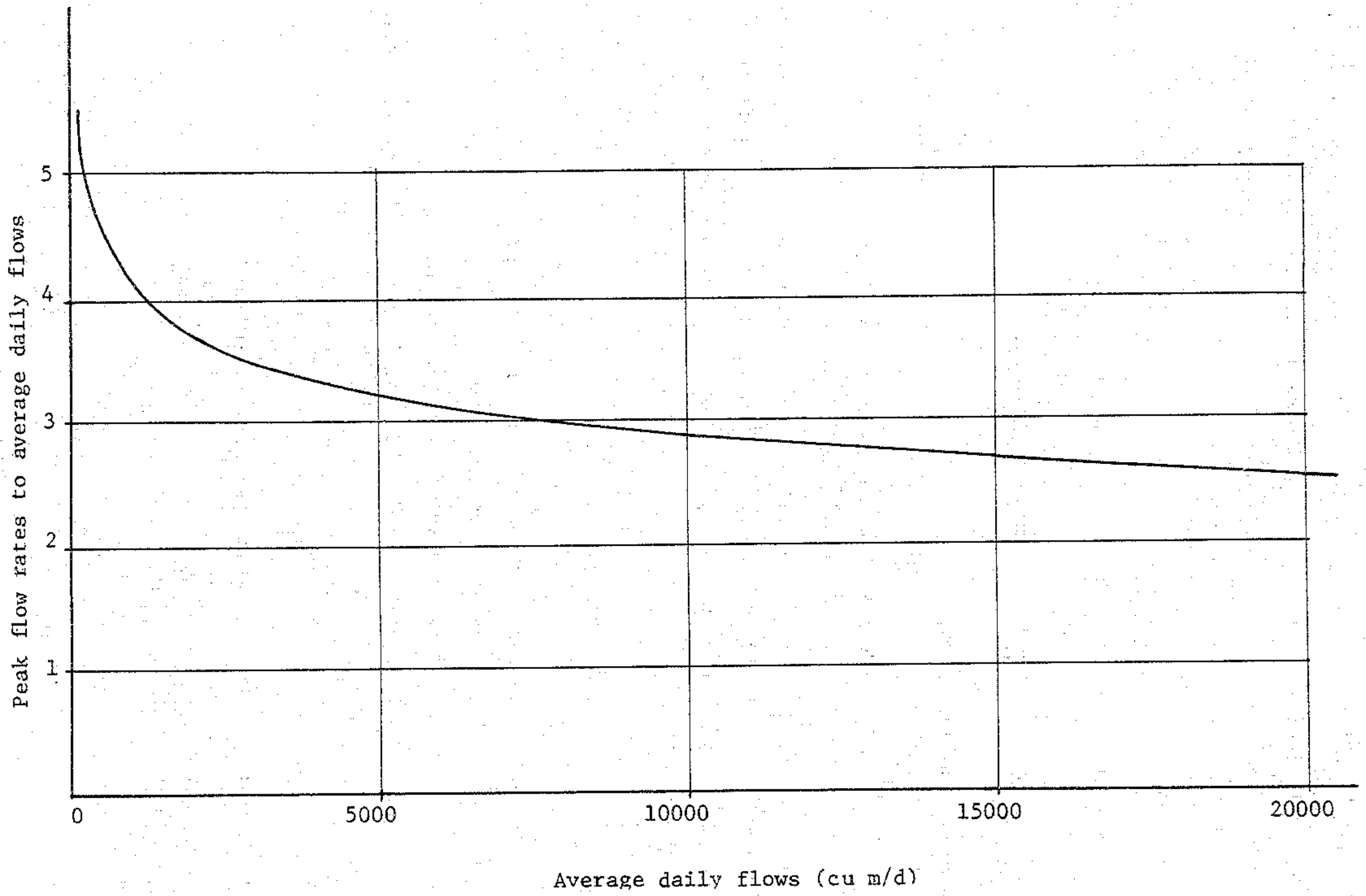
q : daily average flow rate, cu m/cap/day

M : the peak to average ratio (peaking factor), expressed by the formula

$$M = \frac{5}{P^{1/7}}$$

This formula is shown in graphical form in Figure E-1.

FIGURE E-1 Peak flow rates to average daily flows



1.4.6 Slope

Sewer sections and slopes should be designed so that the velocity of flow will not be less than 60 cm/sec for clay pipes and 75 cm/sec for cement-bonded pipes, each pipe section will be separately evaluated to determine the minimum design velocity necessary to control sulfide.

The followings are the minimum slopes which should generally be provided, however slopes greater than these are desirable.

TABLE E-3 Minimum Slope for Sanitary Sewers

Sewer Size (mm dia)	Minimum Slope m/1,000 m		Velocity m/sec	
	VCP	RCP	VCP	RCP
225	3.0	4.5	0.619	0.758
300	2.2	3.5	0.642	0.809
375	1.7	2.6	0.655	0.809
450	1.4	2.2	0.671	0.841
525	1.2	1.8	0.688	0.843
600	1.1	1.6	0.720	0.869
675	1.0	1.4	0.743	0.879
750	0.9	1.3	0.756	0.909
900	0.8	1.1	0.805	0.944
1,050 and larger	0.7 or less	0.9 or less	0.834 and more	0.946 and more

Note: Manning formula using an 'n' value of 0.013.

1.4.7 Alignments

Sewers should generally be laid with straight alignment between manholes. Laying curved sewers should be avoided, unless the available sewer cleaning equipment can handle curvilinear alignments. Also curvilinear alignments are acceptable for large trunks where physical access inside the sewers in readily accomplished.

1.4.8 Increasing Size

When a smaller sewer joins a larger one, the invert of the large sewer should be a sufficiently lower elevation to maintain the same energy gradient. There are four methods which may be used:

(a) To place the crown of both sewers at the same elevation.
(b) To place the water surface of both sewers at the same elevation.
(c) To place the center of both sewers at the same elevation.
(d) To place the invert of both sewers at the same elevation.

From the hydraulical reason the method (b) is the most desirable, however it is impossible to construct both sewers at the same water surface elevation to meet hourly flow rate variation.

Since the sewer depth is the smallest, the method (d) will show the lowest construction cost and the method (c) will show the second lowest. But, the difference will not be significant in the area of average topographic condition. It is therefore recommended to adopt the method (a) which has hydraulic advantages and small extra cost.

1.4.9 Type and Material of Conduit

Sewer pipes are most commonly made of clay or of concrete. Asbestos-cement, and other materials are also suitable for sewer pipes, but may not be available locally at competitive price.

Pipes currently available in Malaysia are limited both in sizes and materials. The following pipes are produced and available on markets in Malaysia:

(a) Clay pipe up to 300 mm in dia.

(b) Centrifugally cast reinforced concrete pipe up to 1,800 mm in dia.

(c) Asbestos-cement pipe up to 600 mm in dia.

(d) Pitch-fibre pipe 100 and 150 mm in dia.

For the selection of sewer materials for the Project, careful considerations should be given to the problem of corrosion of pipes by sulfide build-up in sewers. Even though the sewer system should be designed and operated to be sulfide-free, such corrosion might not be completely prevented in all sewers. Preference should therefore be given to corrosion-resistant materials, such as vitrified clay pipe or lining or coating pipe.

The resistance of vitrified clay pipe to corrosion from acids, alkalies, and virtually all corrosive substances gives it a distinct advantage over other materials as well as excellent resistance to erosion and scour. Disadvantages of vitrified clay pipe are the limited range of sizes and strengths and the fact that it is more brittle than other pipe.

Centrifugally cast reinforced concrete pipe is available in the market in sizes up to 1,800 mm in Malaysia. The advantages of concrete pipe are the relative ease with which the required strength may be provided and the wide range of sizes and laying lengths available. A disadvantage is that all cement-bonded pipes are subject to corrosion, hence a higher design flow velocities must be used to prevent sulfide corrosion problem. Higher velocities require more slope, hence greater excavation and pumping cost.

TABLE E-4 Price of Sewer Pipe (M$/m in 1976)

Diameter (mm)	Pipe Material		
	Centrifugally Cast Reinforced Concrete	Centrifugally Cast Reinforced Concrete High Alumina Cement Mortar Lining (1/2 in.)	Vitrified Clay
150	11.47	18.85	12.99
225	17.05	28.36	21.65
300	20.98	35.25	32.50
375	30.33	49.34	
450	35.25	57.87	
525	42.46	68.69	
600	47.57	76.88	
675	63.44	97.21	
750	70.82	107.70	
900	92.95	137.54	
1,050	122.95	174.75	
1,200	136.23	192.79	
1,350	179.84	246.07	
1,500	208.85	283.77	
1,800	281.47	369.67	

Note: inclusive of joint material

Pitch fibre pipes are also available in Malaysia in sizes 100 and 150 mm diameter. The pipes are generally of good quality and meet internationally accepted standards.

In view of the above mentioned conditions, the following considerations should be taken into account in selecting sewer materials:

(a) Sanitary sewers of smaller sizes up to 300 mm in diameter should normally be virtrified clay pipe, this pipe is available locally at competitive price.

(b) Sanitary sewers of 375 mm or more in diameter should be of centrifugally cast reinforced concrete pipes conforming to internationally accepted standards, with high alumina cement mortar lining.

1.4.10 Joints

Infiltration is a major cause of hydraulic overloading of both collection system and treatment plant. Most infiltration occurs through faulty or poor sewer joints. Private house connections to public sewers have in many cases contributed more infiltration than the system itself. It is therefore recommended that MPSP develop a strong and adequate code covering materials and also construction of house connections.

Experience in many countries shows that the compression type and rubber gasket type joints show generally very superior performance in preventing groundwater infiltration into sewers. Various proper forms of flexible joints are available on the market. Among them, the most reliable joint which has water tightness, flexibility, and durability is probably the rubber gasket type joint.

In view of the above mentioned comments, the following joints are recommended for different materials of sewer pipe:

(a) Concrete Pipe

Recently concrete pipe manufacturers have successfully employed compression rubber gaskets for bell and spigot and tongue and groove concrete pipe. A variety of these types of joints are available. It is therefore recommended that all concrete pipe joint be of the rubber-gasket type.

(b) Vitrified Clay Pipe

Vitrified clay pipe can be obtained with factory-applied 'push-fit' joints. These can incooperate polyester rings and a rubber 'O' ring, or they may be of polyurethane with an integral nob. Any of these modern type joints which prevent infiltration would be acceptable.

1.4.11 Manholes

(a) Location

Manholes shall be installed at the end of each line; at all changes in grade, size, or alignment; and at all intersections. On larger main sewers, however, which can be entered for cleaning, these changes may be made without the requirement of manholes.

(b) Spacing

Spacings of manhole by size of sewer should not be more than as shown in Table E-5. Manholes should, in any case, not normally be more than 200 meters apart, so that men working in a sewer can easily reach a manhole in an emergency.

TABLE E-5 Maximum Manhole Spacings

Pipe Dia. (mm)	300 or Less	600 or Less	1050 or Less	1500 or Less	1650 or More
Maximum Spacing (m)	50	80	100	150	200

In fixing these maximum spacings, similar cities, in Malaysia and other countries, were studied. In case of George Town, where a separate sewerage system has been in operation since 1933, a maximum manhole spacing of 90 meters for sewer sizes up to 600 mm and on larger sized sewers spacing up to 150 m has been used as a design standard, without much trouble in cleaning of sewer pipes. Spacing should be dictated by the type of sewer cleaning equipment used.

The rod type cleaning instruments will be used as the main cleaning device for years to come, instead of highly mechanized equipment such as hydraulic sand ejectors, because of the much lower cost, ease in handling, and plentiful availability of labour and the need to develop employment opportunities for labourers. Accordingly, the spacings in Table E-5 are recommended, except in cases where modern equipment adequate for greater spacing is provided.

(c) Dimensions

Except for very shallow drains and sewers of less than 1 meter depth to the invert (special case) all manholes should be of adequate dimensions for entry and for operation of cleaning rods. The internal size of manhole should not be less than 120 cm; but larger sizes are preferable. The recommended standard classification of manhole diameters and internal sizes are as shown in Table E-6.

TABLE E-6 Recommended Manhole Size (mm)

Minimum Internal Size	Connecting Sewer Diameter
1,200 mm	900 or less mm
1,500	1,200 or less
1,800	1,500 or less

(d) Materials

Watertight manhole covers, either of reinforced concrete or cast iron, are to be used wherever the manholes tops may be flooded by street runoff or high water. The size of manhole cover should be greater than 60 cm.

Generally manholes should be circular, with a reinforced base and reinforced wall construction.

For larger and deeper manholes, it is recommended that a precast concrete base, tapered sections, shaft sections and cover slabs be used in order to sustain heavy loads.

CHAPTER 2

COST ESTIMATING PROCEDURES FOR SEWERS

2.1 General

The costs associated with constructing and operating the sewerage system are difficult to estimate for planning purposes. This is true particularly when the planning area includes a variety of geological and topographical features. Also, the costs of treatment processes must be related to the effectiveness of the processes in removing water contaminants to meet a variety of receiving water condition.

In the master planning of the Butterworth/Bukit Mertajam Metropolitan Area sewerage and drainage systems, alternatives should be considered and evaluated in order to establish to most desirable plan. Estimation of costs of these alternatives will be almost impossible in the project duration, unless cost function relationships are developed. The cost functions for conveyance are developed on the basis of 1976 price levels in Penang State.

2.2 Construction Costs

Construction costs of the project may be defined as the sum of all expenditures required to bring the project to completion. These expenditures are divided into direct items and indirect items. The direct items include excavation of trenches, laying and construction of sewers, and all the related construction works including indirect items and any other expenditures expected. In this study, preliminary designs have first been made to obtain quantities and then these have been multiplied by appropriate unit prices to obtain the total costs of project components. For the indirect items, 20 percent was added to the direct items.

2.2.1 Basic Costs

In estimating the construction costs of the facilities, unit costs for domestic items such as labour, materials to be purchased in Malaysia, power, equipment and transportation, materials and equipment to be imported, were collected and checked by both survey team staff and local contractors.

Labourers required for the sewerage constructions may include a wide range of occupational categories, from common labourers to skilled operators for heavy equipment. The current (1976) applicable labour costs for various types of labour in Penang State are from M$ 8 to 20 per day as given in the Table below.

TABLE E-7 Labour Costs

Type of Labourer	M$/day
Common worker	8
Skilled worker	15
Carpenter	12
Stone masonry	12
Plumber	15
Foreman	20

Data source: PWD

Generally, for construction of structures, including pumping stations and treatment facilities, most of the materials required in the project are available, except mechanical equipment which will be imported on an international basis.

Reinforcing bars, timber, sand and gravel for concrete products, vitrified clay pipes, and centrifugally cast reinforced concrete pipes (less than 1,800 mm in diameter) are available in Malaysia. The unit price of these basic materials are given in the following Tables.

Land acquisition cost in 1976 price level in the Project Area is estimated on the basis of information obtained from MPSP as shown in Figure E-2.

TABLE E-8 Price of Basic Materials - (1)

Item	Unit	Price (M$)	Remarks
Cement	ton	109	
Sand	cu m	12	
Gravel	cu m	27	
Steel bar	ton	610	
Timber	ton	410	
Vitrified clay pipe			
ϕ150	m	12.99	
ϕ225	m	21.65	
ϕ300	m	32.50	

Data source: Local contractor

TABLE E-9 Price of Basic Materials - (2)

Item	Unit	Price (M$)	Remarks
Centrifugally cast re-inforced concrete pipe (mm in dia.)			with high alumina cement mortar linings and rubber ring
ϕ150	m	19.94	
ϕ225	m	29.45	
ϕ300	m	36.34	
ϕ375	m	50.71	
ϕ450	m	58.85	
ϕ525	m	70.00	
ϕ600	m	78.36	
ϕ675	m	98.85	
ϕ750	m	109.67	
ϕ900	m	140.16	
ϕ1,050	m	178.03	
ϕ1,200	m	196.89	
ϕ1,350	m	250.99	
ϕ1,500	m	290.66	
ϕ1,800	m	377.87	

Data source: Hume Industry at K.L & K.L Sewerage Master Plan

TABLE E-10 Unit Costs for Construction, including Labour and Materials

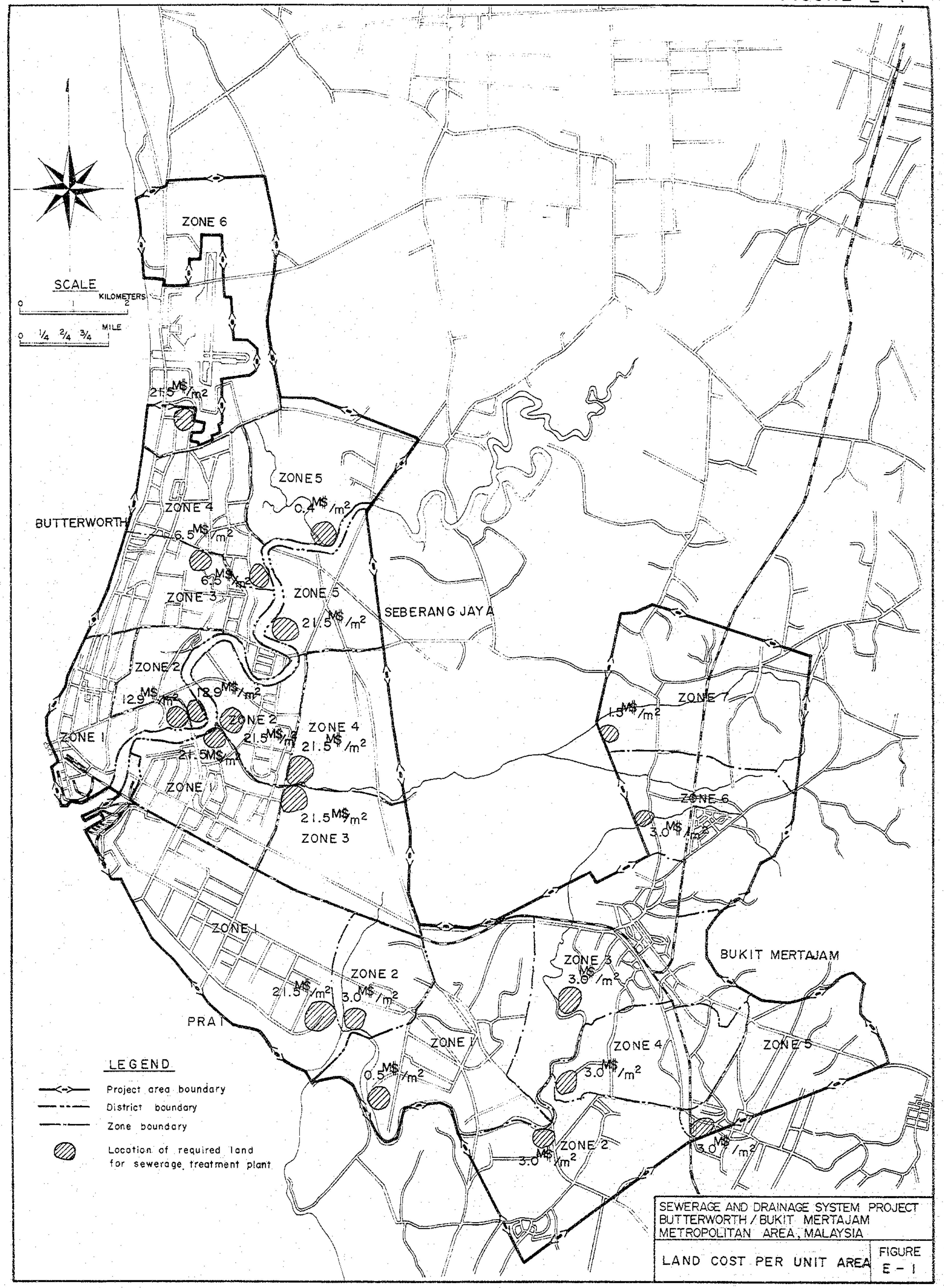
Item	Description	Unit	Cost (M$)
Concrete	mix. 1:2:4	cu m	94.17
"	mix. 1:3:6	"	78.47
Reinforced concrete		"	392.39
Mortar works	mix. 1:2	"	103.60
Surplus soil removal		"	1.96
Excavation	open cut	"	1.96
"	trench (depth 0-1.5m)	"	6.54
"	" (" 1.5-3.0m)	"	9.16
"	" (" 3.0-4.5m)	"	19.62
"	" (" 4.5-6.0m)	"	31.39
"	" (" 6.0-)	"	39.00
Backfilling and compaction		"	1.57
Bedding	sand bed	"	18.31
"	gravel bed		26.16
Forming		sq m	13.99
Dewatering		hour	5.50
Restoration of paving		sq m	20.00
Sheeting		ton	393.70

Data source: PWD & Local contractor

2.2.2 Unit Costs for Sewerage System

Construction costs were estimated for the sewerage system, taking into account the known or estimated costs of excavation, sheeting, dewatering, bedding, pipe supplying and laying, concrete placing, forming, reinforcing, restoration of paving and contractor's profit

FIGURE E-1


SCALE
KILOMETERS
0 1 2
MILE
0 1/4 2/4 3/4
ZONE 6
21.5 M$/m2
ZONE 5
0.4 M$/m2
ZONE 4
6.5 M$/m2
BUTTERWORTH
6.5 M$/m2
ZONE 3
ZONE 5
21.5 M$/m2
SEBERANG JAYA
ZONE 2
12.9 M$/m2
12.9 M$/m2
ZONE 2
21.5 M$/m2
ZONE 4
21.5 M$/m2
ZONE 1
21.5 M$/m2
ZONE 1
21.5 M$/m2
ZONE 3
ZONE 1
ZONE 2
21.5 M$/m2
3.0 M$/m2
PRAI
ZONE 1
0.5 M$/m2
ZONE 7
1.5 M$/m2
ZONE 6
3.0 M$/m2
BUKIT MERTAJAM
ZONE 3
3.0 M$/m2
ZONE 4
3.0 M$/m2
ZONE 5
3.0 M$/m2
ZONE 2
3.0 M$/m2
LEGEND
Project area boundary
District boundary
Zone boundary
Location of required land for sewerage treatment plant
SEWERAGE AND DRAINAGE SYSTEM PROJECT
BUTTERWORTH / BUKIT MERTAJAM
METROPOLITAN AREA, MALAYSIA
LAND COST PER UNIT AREA
FIGURE
E-1

and overhead. The cost estimations were developed for normal conditions excluding such additional costs as required for rock excavation, relocation of under-ground utilities, foundation or dewatering for which special technics are quired, and any works required for special conditions.

Five different sizes of circular pipes, 15 cm, 30 cm, 60 cm, 120 cm, and 180 cm in diameter, each for different earth covering of 2 m, 4 m, 6 m, 8 m, and 10 m were considered together with estimation of construction costs.

The average unit costs, as estimated for circular pipes, are summarized in the following Table:

TABLE E-11 Estimated Construction Costs of Circular Pipes, Including Manhole

(M$/m)

Diameter (m) \ Depth of Excavation (m)	2.0	4.0	6.0	8.0	10.0
0.15	120	220	790		
0.30	160	270	860	2,320	2,860
0.60	270	410	1,070	2,590	3,180
1.20		780	1,540	3,200	3,870
1.80		1,270	2,120	3,950	4,710

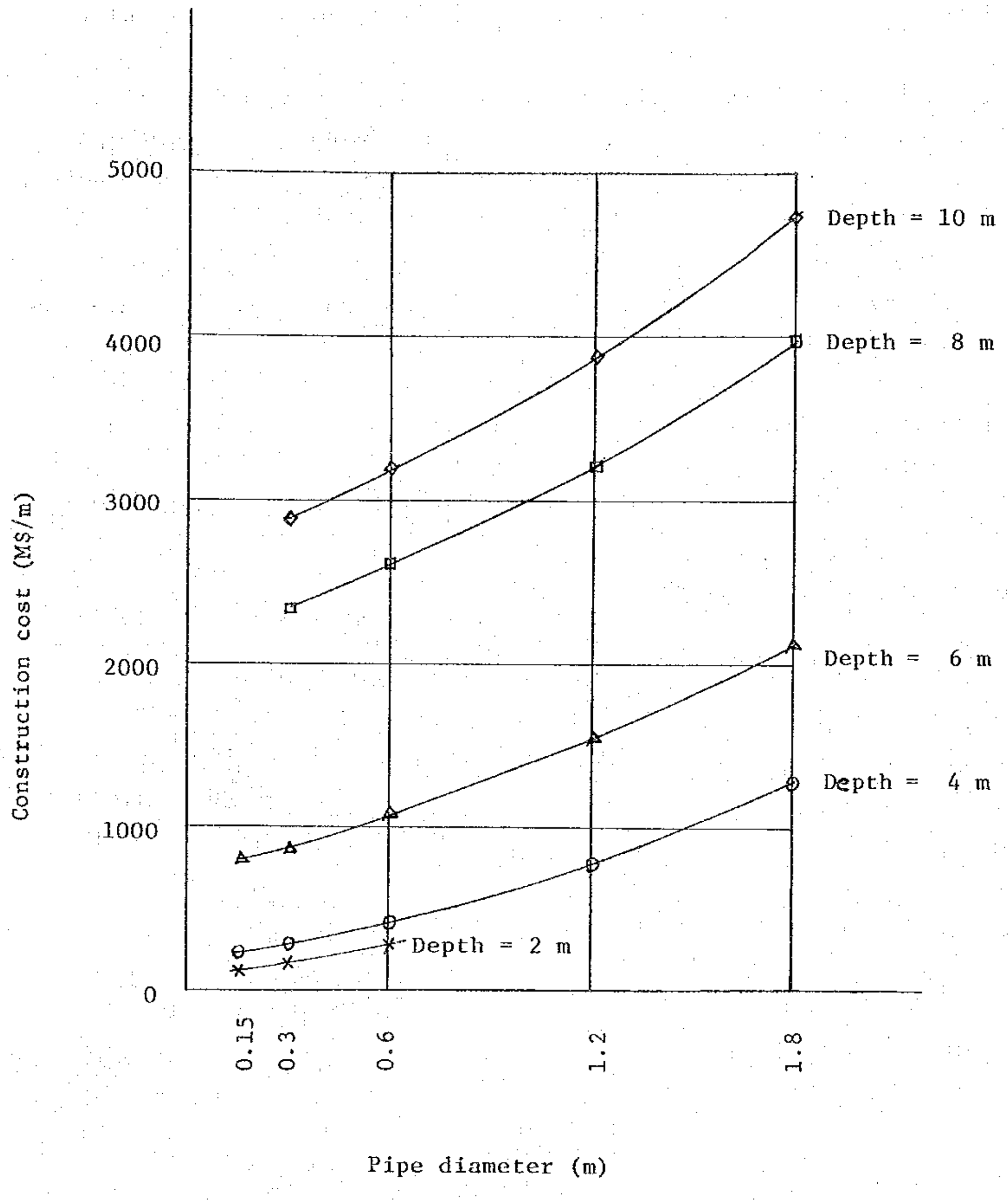


FIGURE E-3 Estimated construction cost of circular pipe, including manhole

2.3 Cost Functions

Cost functions were derived on the basis of the unit costs calculated in the previous paragraphs. The equations to be used for planning were selected, then the functions were developed by the least square method.

Depth = 2.0 m	$C_p = 222.2\ D^2 + 166.7\ D + 90$
= 4.0	$C_p = 173.5\ D^2 + 300.3\ D + 168$
= 6.0	$C_p = 150.7\ D^2 + 516.6\ D + 702$
= 8.0	$C_p = 175.9\ D^2 + 714.7\ D + 2{,}093$
= 10.0	$C_p = 175.5\ D^2 + 860.8\ D + 2{,}591$

Where:

C_p : Construction cost, M\$/m

D : Pipe diameter, m

CHAPTER 3

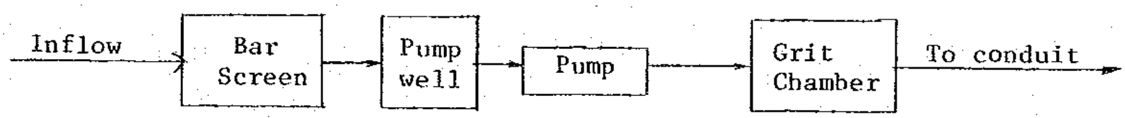
COST FUNCTIONS FOR PUMPING STATIONS

3.1 General

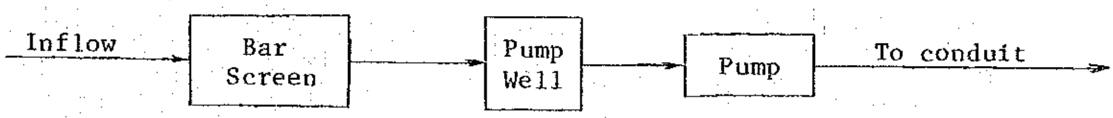
In developing cost functions for pumping stations, cost estimates were made for six stations, different capacities of 0.05 cu m/sec, 0.2 cu m/sec, 0.5 cu m/sec, 0.87 cu m/sec (75,000 cu m/day), 1.73 cu m/sec (150,000 cu m/day), and 8.68 cu m/sec (750,000 cu m/day) for depth of inflowing conduit at 10 m (assumed average depth of pumping station).

The pumping stations which have capacity of more than 0.5 cu m/sec consist of gates, screens, grit chambers, grit removal facilities, pump well, building for pumping equipment and controling devices, piping, etc., but for the smaller pumping stations of less capacity than 0.5 cu m/sec, no grit chambers are installed.

A flow sheet of the station of more capacity than 0.5 cu m/sec is given as follows:



A flow sheet of the station of less capacity than 0.5 cu m/sec. is given as follows:



3.2 Construction Cost

Construction costs including 20 percent of over head are estimated for each of the 6 cases for their civil works, piping, buildings, equipment, electrical and controling devices, and other appurtenances, and are summarized in the following Table.

TABLE E-12 Construction Costs of Pumping Stations of 10m Depth by Capacity

Unit : 1000 M$

Capacity (cu m/sec)	Civil works & Building	Machinery & Electricity	Total	Remarks
0.05	108	76	184	without grit chambers
0.2	170	137	307	"
0.5	237	227	464	"
0.87	2,411	1,881	4,292	with grit chambers
1.73	2,954	4,015	6,969	"
8.68	9,668	11,325	20,993	

3.3 Cost Function

As illustrated in Figures D-4&5, the cost function of a pumping station may be expressed in the linear form as :

$$C_p = a\,Q + b$$

where a, b : Constants

C_p : Construction cost, 1000 M$

Q : Peak flow rate, cu m/sec.

The values of "a" and "b" are obtained by the least square method. Hence, the cost functions may be expressed as:

$$C_p = 608.1\,Q + 166 \qquad (Q \lesssim 0.5 \text{ cu m/sec})$$

$$C_p = 2092.0\,Q + 2{,}885 \qquad (Q > 0.5 \text{ cu m/sec})$$

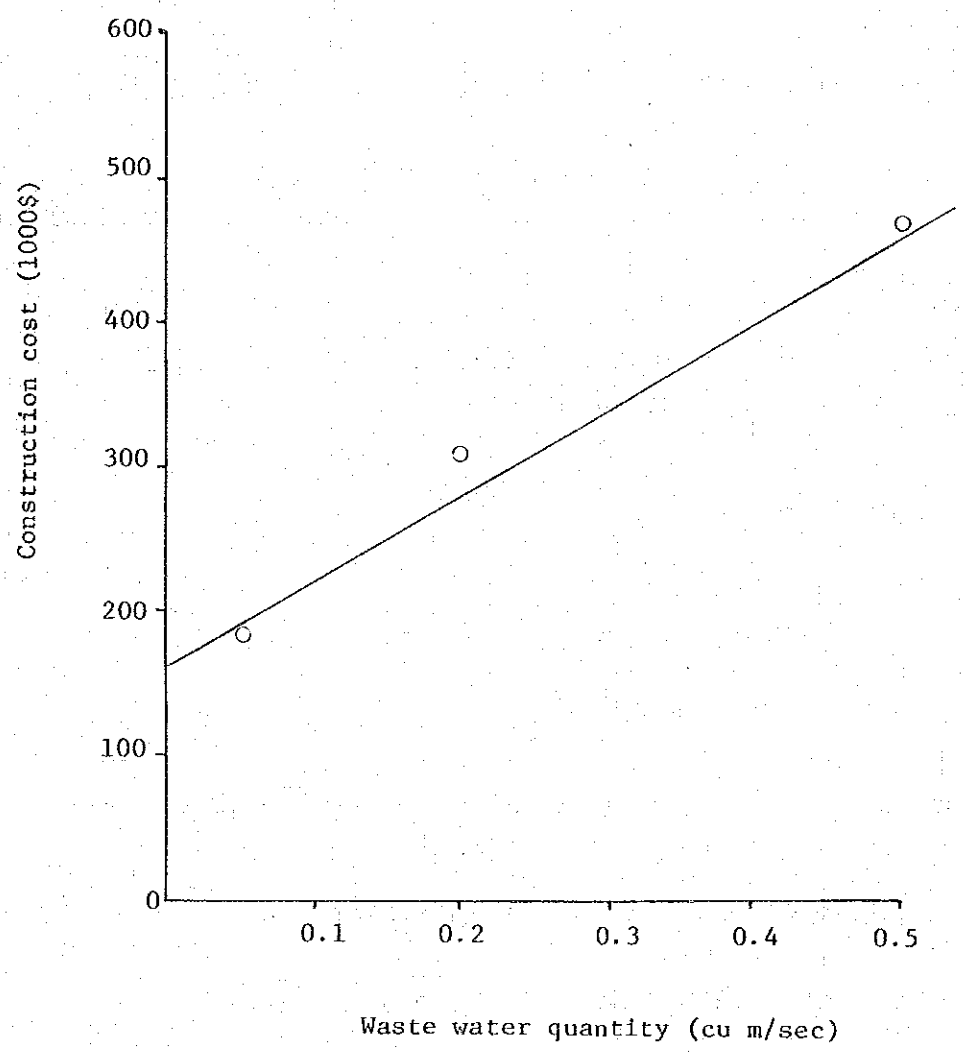


FIGURE E-4 Construction cost for pumping station of 10 m depth by capacity (less than 0.5 cu m/sec)

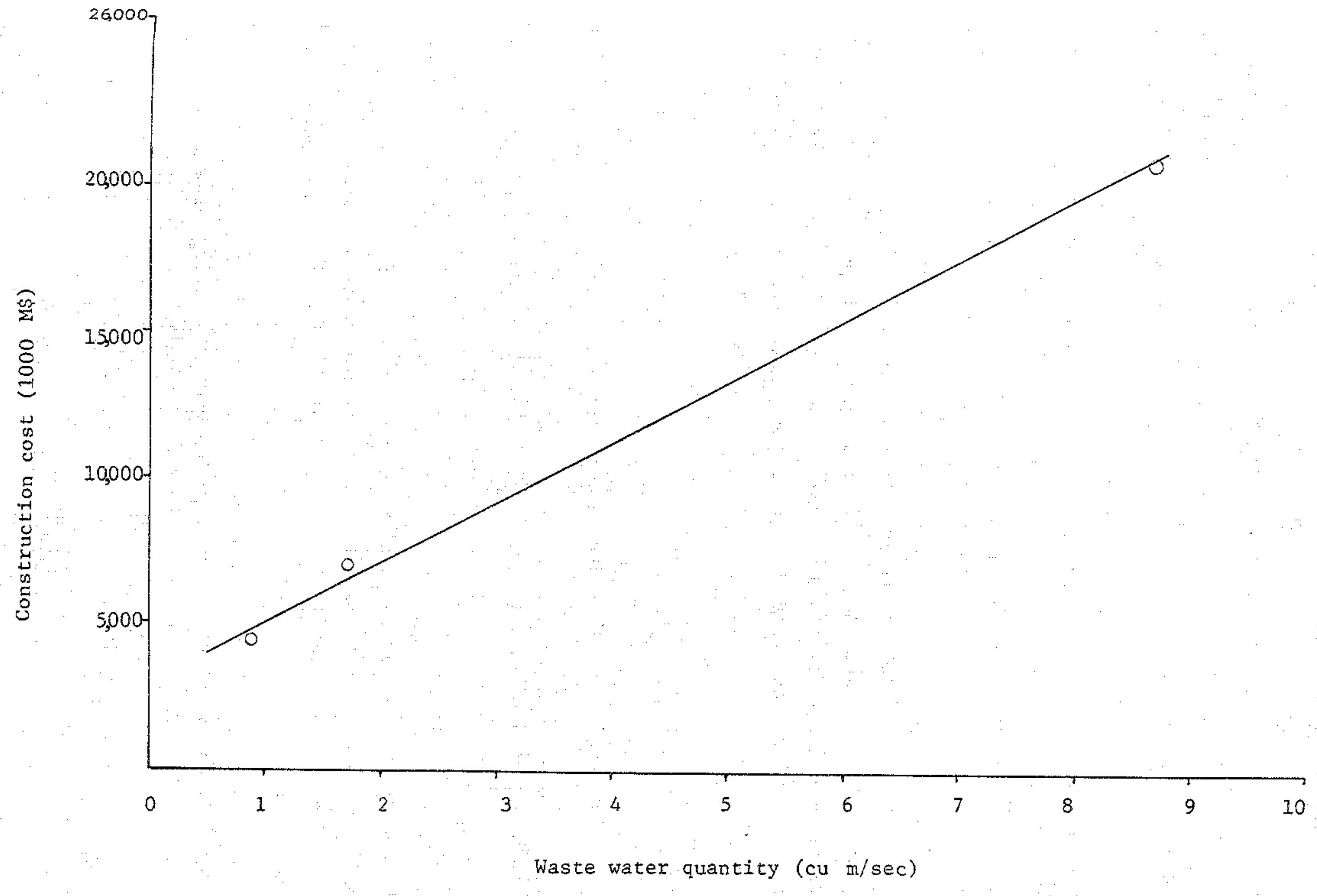


FIGURE E - 5 Construction cost for pumping station of 10 m depth by capacity (more than 0.5 cu m/sec)

CHAPTER 4

COST FUNCTIONS FOR TREATMENT PLANTS

4.1 General

Development of a relationship between capacity and cost of treatment in the form of a cost function is the most practicable approach in sewerage system planning so that various alternative can easily be estimated.

The construction costs of three different treatment processes, stabilization pond, aerated lagoon, and oxidation ditch, at five different capacities, 5,000 cu m/day, 10,000 cu m/day, 50,000 cu m/day, 100,000 cu m/day, and 200,000 cu m/day, were evaluated, then cost functions were developed.

4.2 Stabilization Pond Process

A flow sheet of the stabilization pond process is given as follows:

Inflow → Facultative Pond → Maturation Pond → Effluent

The construction costs of civil works were on the basis of material costs at 1976 Penang State price levels. Costs for equipment were estimated based on costs in Japan but adjusted by adding shipping charge and customs duties.

Table E-13 shows estimated construction costs including 20 percent of overhead.

TABLE E-13 Construction Cost for Stabilization Pond Process by Capacity

Unit: 1000 M$

Item \ Capacity	5000cu m/day	10,000cu m/day	50,000cu m/day	100,000cu m/day	200,000cu m/day
Civil works & Building	524	1,048	5,240	10,480	20,960
Machinery & Electricity	-	-	-	-	-
Total	524	1,048	5,240	10,480	20,960

As illustrated in Figure E-6, the cost function of a stabilization pond process may be expressed as linear form in the logarithmic diagram.

$$C_s = a\ Q^b$$

where a, b: constants

C_s : Construction cost, 1000 M$

Q : Daily average flow, cu m/day

The values of "a" and "b" are obtained by the least square method. Thus, the cost function may be expressed as:

$$C_s = 0.1048Q$$

where C_s: Construction cost, 1,000 M$

Q : Capacity, cu m/day

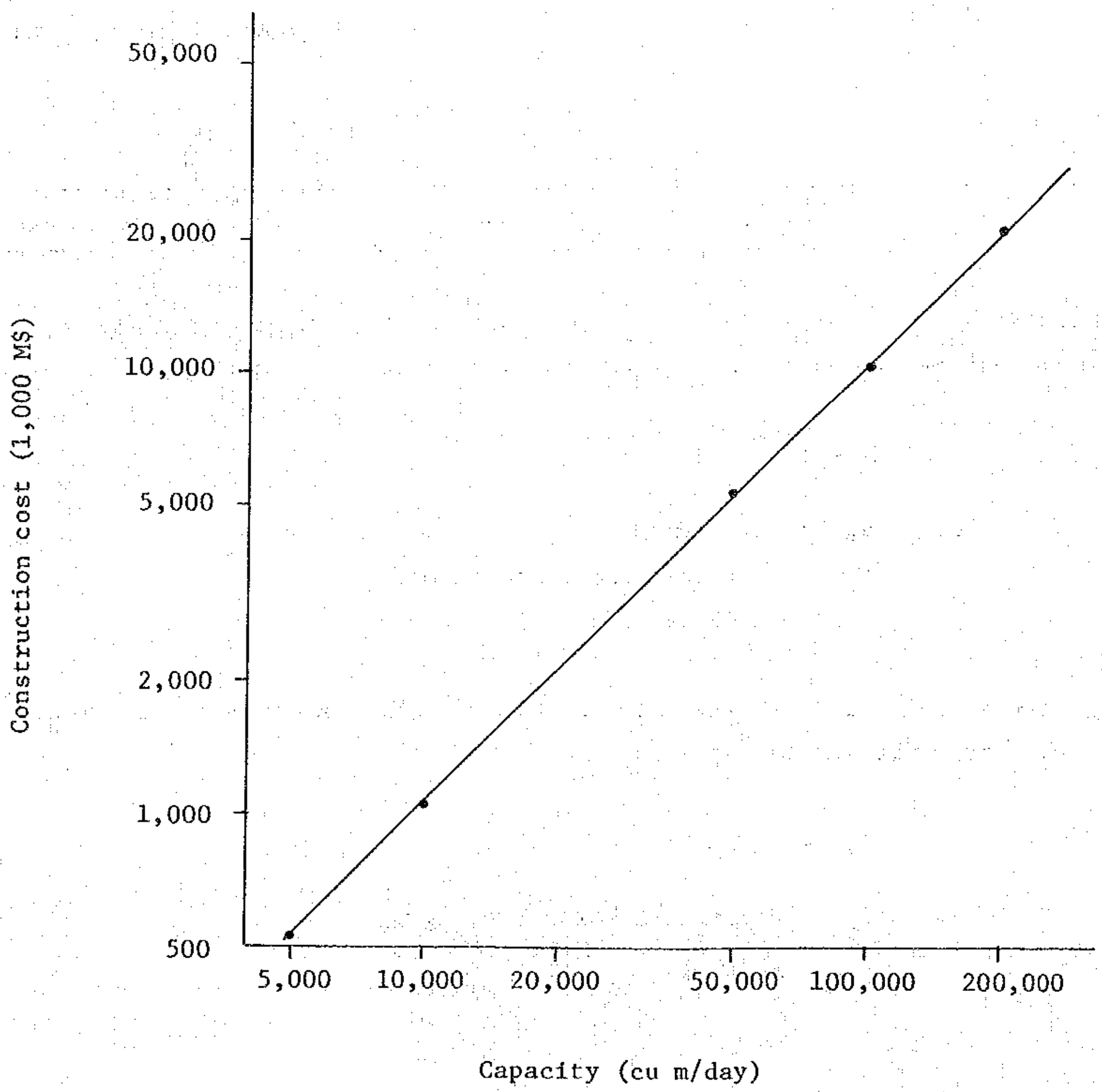


FIGURE E-6 Construction cost for stabilization pond process

4.3 Aerated Lagoon Process

A flow sheet of the aerated lagoon process is given as follows:

Inflow → Aerated Lagoon → Maturation Pond → Effluent

Construction costs including 20 percent of overhead is tabulated in Table E-14.

TABLE E-14 Construction Cost for Aerated Lagoon Process by Capacity

Unit: 1,000 M$

Item \ Capacity	5,000 cu m/day	10,000 cu m/day	50,000 cu m/day	100,000 cu m/day	200,000 cu m/day
Civil works & Building	1,081	2,162	10,810	21,620	43,240
Machinery & Electricity	60	114	540	1,020	2,040
Total	1,141	2,276	11,350	22,640	45,280

The cost function may be expressed as:

$$C_A = 0.2323\ Q^{0.998}$$

where

C_A: Construction cost, 1,000 M$

Q : Capacity, cu m/day

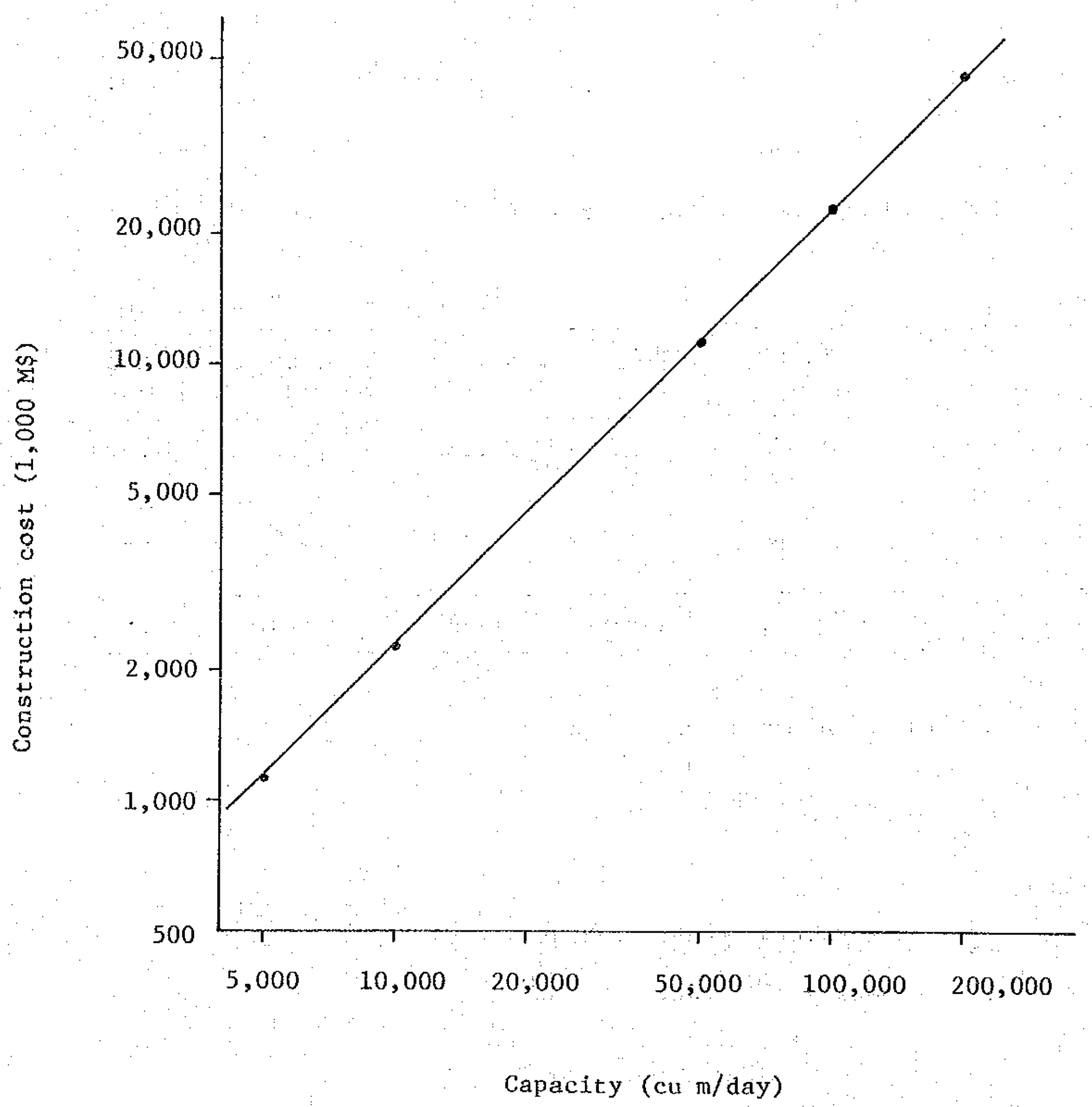
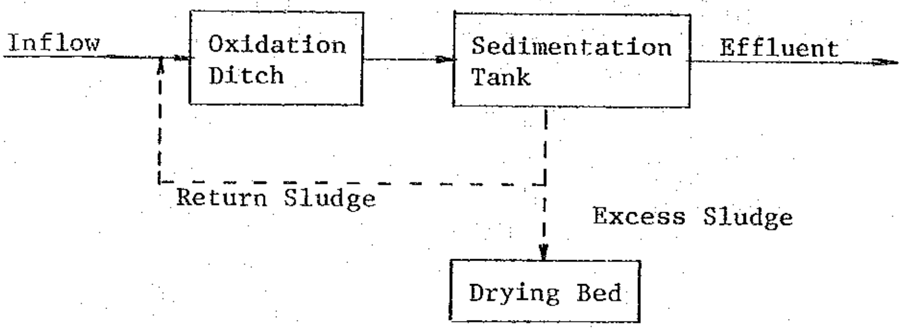


FIGURE E-7 Construction cost for aerated lagoon process

4.4 Oxidation Ditch Process

A flow sheet of the oxidation ditch process is given as follows:



Construction costs including 20 percent of overhead is tabulated in Table E-15.

TABLE E-15 Construction Cost for Oxidation Ditch Process by Capacity

Unit: 1,000 M$

Item \ Capacity	5,000 cu m/day	10,000 cu m/day	50,000 cu m/day	100,000 cu m/day	200,000 cu m/day
Civil Works & Building	213	426	2,130	4,260	8,520
Machinery & Electricity	769	1,461	6,921	13,073	26,146
Total	982	1,887	9,051	17,333	34,666

The cost function may be expressed as:

$$Co = 0.2614\ Q^{0.966}$$

where

Co: Construction cost, 1,000 M$

Q : Capacity, cu m/day

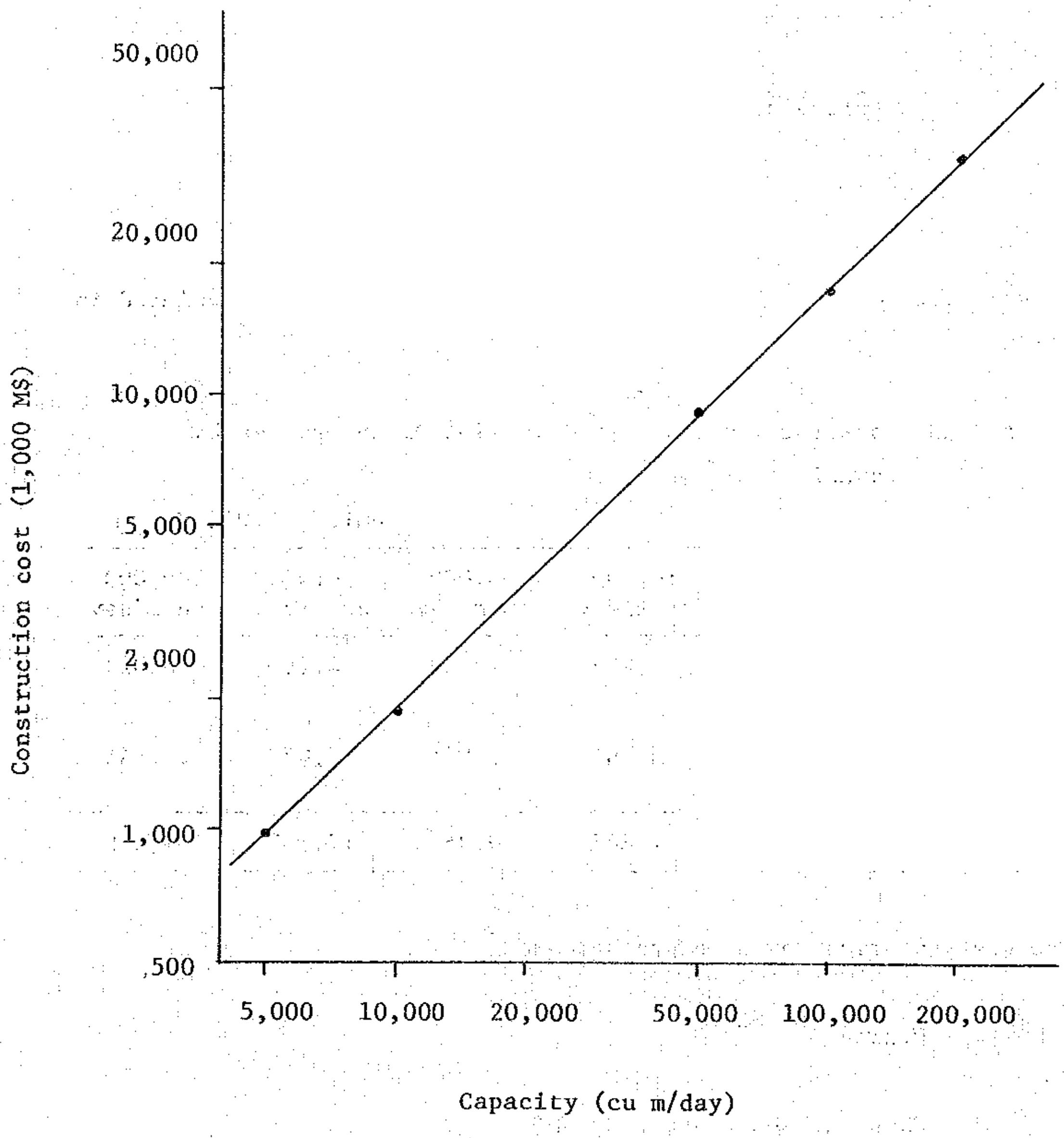


FIGURE E-8 Construction cost for oxidation pitch process

CHAPTER 5

OPERATION AND MAINTENANCE COSTS

5.1 General

Generally, comprehensive sewerage system consists of sewers, pumping stations, and treatment facilities. And to maintain these facilities significant expenditures are required. They are salary for operators and labours, electricity, purchase of equipment, purchase of machine oil, repairing cost, etc.

Cost functions for sewer maintenance, operation and maintenance of pumping stations and treatment facilities are developed respectively.

5.2 Sewers

Maintenance costs for sewers were estimated based on the following assumptions:

(a) Frequency of cleaning for public sewers is once in every 4 years.
(b) Ability to clean by one team for public sewers is 150 m/day.
(c) Useful life of the cleaning equipment is 10 years.
(d) Team member, for public sewers is 6 persons.
(e) Others: 50 percent of equipment cost including costs for parts, repairing, overhauling, etc.
(f) Annual rehabilitation cost of sewer is 0.5% of construction cost.
(g) Working days and hours

Working days are 300 days/year.

Working hours are 6 hours/day.

(h) Worker cost is 8.00 M$/day.
(i) Price of machine

Power driven bucket machine is 121,000 M$/set.

Flexible rod type equipment and high pressure cleaning machine is 77,000 M$/set.

Based on the data and assumptions above, it was estimated that 1.70 M$ will be necessary for maintenance of one meter of public sewers per year.

5.3 Pumping Stations

According to the capacity, different system is considered for pumping station, that is, no grit chamber is installed for station of smaller capacity than 0.5 cu m/sec. Therefore, two cost functions, for more capacity than 0.5 cu m/sec and for less capacity than 0.5 cu m/sec, are developed.

In developing the cost functions, followings are assumed in advance.

(a) For station of more capacity than 0.5 cu m/sec, daily average number of operator is 1 (one) person per station,

(b) For station of less capacity than 0.5 cu m/sec, daily average number of operator is 0.1 person per station, and

(c) Electricity is assumed at 8 M¢/kWh, and average salary of operator is assumed at 15 M$/day.

The operation and maintenance costs by capacity were then estimated as shown in Table E-16 and Figure E-9.

TABLE E-16 Operation and Maintenance Costs for Pumping Station by Capacity

Unit: 1,000 M$/year

Item \ Capacity	0.05 cu m/sec	0.2 cu m/sec	0.5 cu m/sec	0.87 cu m/sec	1.73 cu m/sec
Salary	0.5	0.5	0.5	5.5	5.5
Electricity, etc.	15.3	23.7	52.3	102.0	127.2
Total	15.8	24.2	52.8	107.5	132.7

On the basis of these figures and Figure E-9, cost functions for operation and maintenance of pumping station were obtained as follows:

$$C_{MP} = 84.1\ Q_P + 9.9 \quad (Q_P \geqq 0.5 \text{ cu m/sec})$$

$$C_{MP} = 29.3\ Q_P + 82.0 \quad (Q_P > 0.5 \text{ cu m/sec})$$

where

Q_P : Peak flow, cu m/sec

Q_{MP}: Operation and maintenance cost, 1,000 M$/year

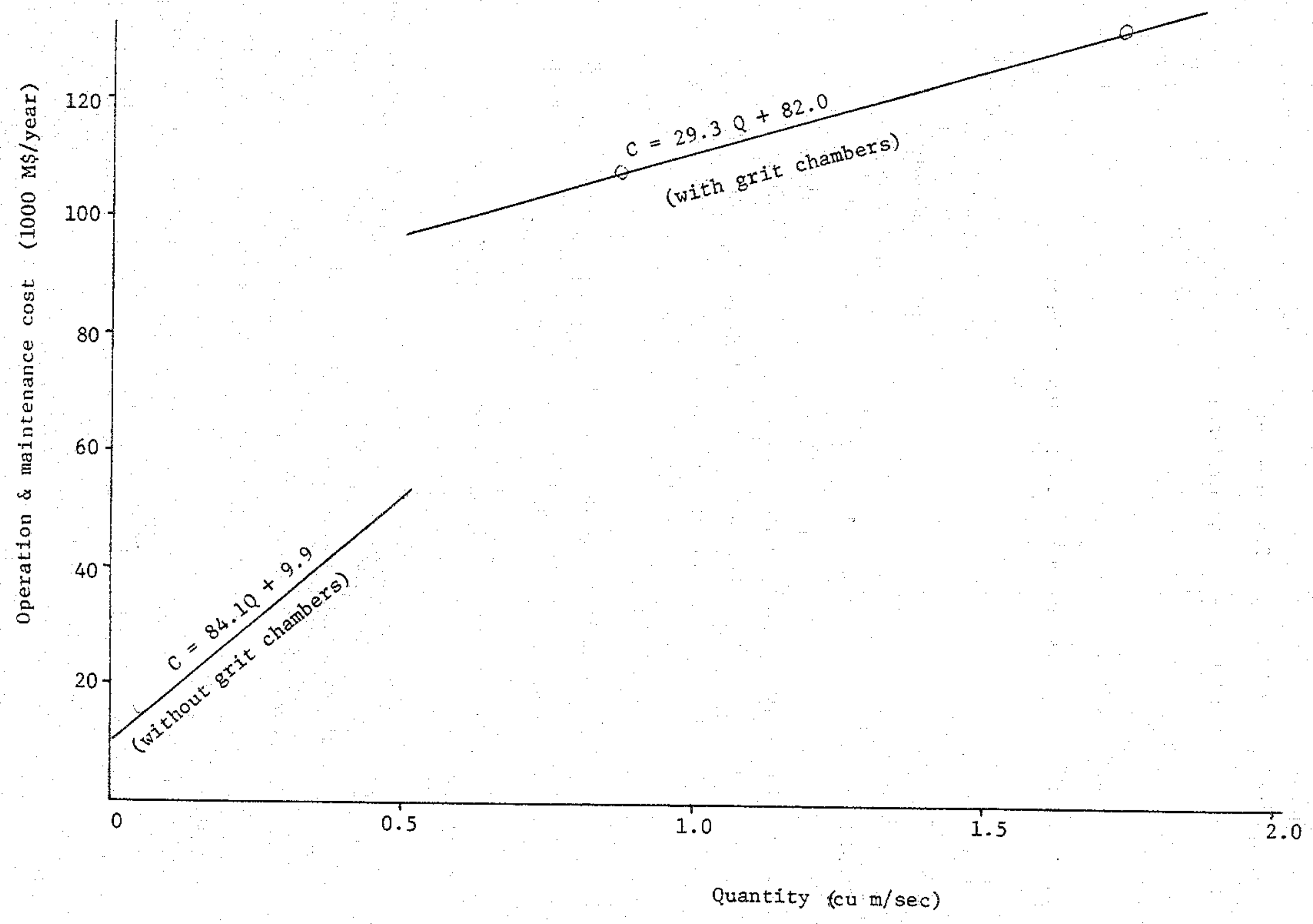


FIGURE E-9 Operation & maintenance cost of pumping station

5.4 Treatment Plants

Cost functions for three different treatment processes, stabilization pond, aerated lagoon, and oxidation ditch, are developed.

In developing the cost functions for treatment plants followings are assumed in advance.

(a) Daily average number of operator is 2 (two) persons for 5,000 cu m/day plant and 4 (four) persons for 200,000 cu m/day plant for stabilization pond and aerated lagoon processes. And, 4 (four) persons for 5,000 cu m/day plant and 20 persons for 200,000 cu m/day plant for oxidation ditch process are estimated.

(b) Electricity is 8 M¢/kWh and average salary of operator is 15 M$/day.

The operation and maintenance costs by capacity by treatment process were then estimated as shown in Table E-17 and Figures E-10, E-11, and E-12.

TABLE E-17 Operation and Maintenance Costs for Treatment Plants by Capacity by Treatment Process

Unit: 1,000 M$/year

Item \ Capacity	5,000 cu m/day	10,000 cu m/day	50,000 cu m/day	100,000 cu m/day	200,000 cu m/day
(a) Stabilization Pond					
Salary	10.95	11.20	13.50	16.30	21.90
Electricity, etc.	-	-	-	-	-
Total	10.95	11.20	13.50	16.30	21.90
(b) Aerated Lagoon					
Salary	10.95	11.20	13.50	16.30	21.90
Electricity, etc.	17.40	34.56	164.05	327.86	655.49
Total	28.35	45.76	177.55	344.16	677.39
(c) Oxidation Ditch					
Salary	21.90	32.85	43.80	54.75	109.50
Electricity, etc.	54.08	110.08	544.26	1,046.48	2,167.42
Total	75.98	142.93	588.06	1,101.23	2,167.42

On the basis of these figures in Table E-17 and Figures E-10, 11 and 12, cost for operation and maintenance of treatment plant were obtained as follows:

(i) For stabilization pond process

$$C_{MS} = 5.292 \times 10^{-5}Q + 9.33$$

(ii) For aerated lagoon

$$C_{MA} = 3.327 \times 10^{-3}Q + 11.80$$

(iii) For oxidation ditch

$$C_{MO} = 1.067 \times 10^{-2}Q + 35.95$$

where C_{MS}, C_{MA}, C_{MO}: Operation and maintenance cost, 1,000 M$/year

Q: Daily average flow, cu m/day

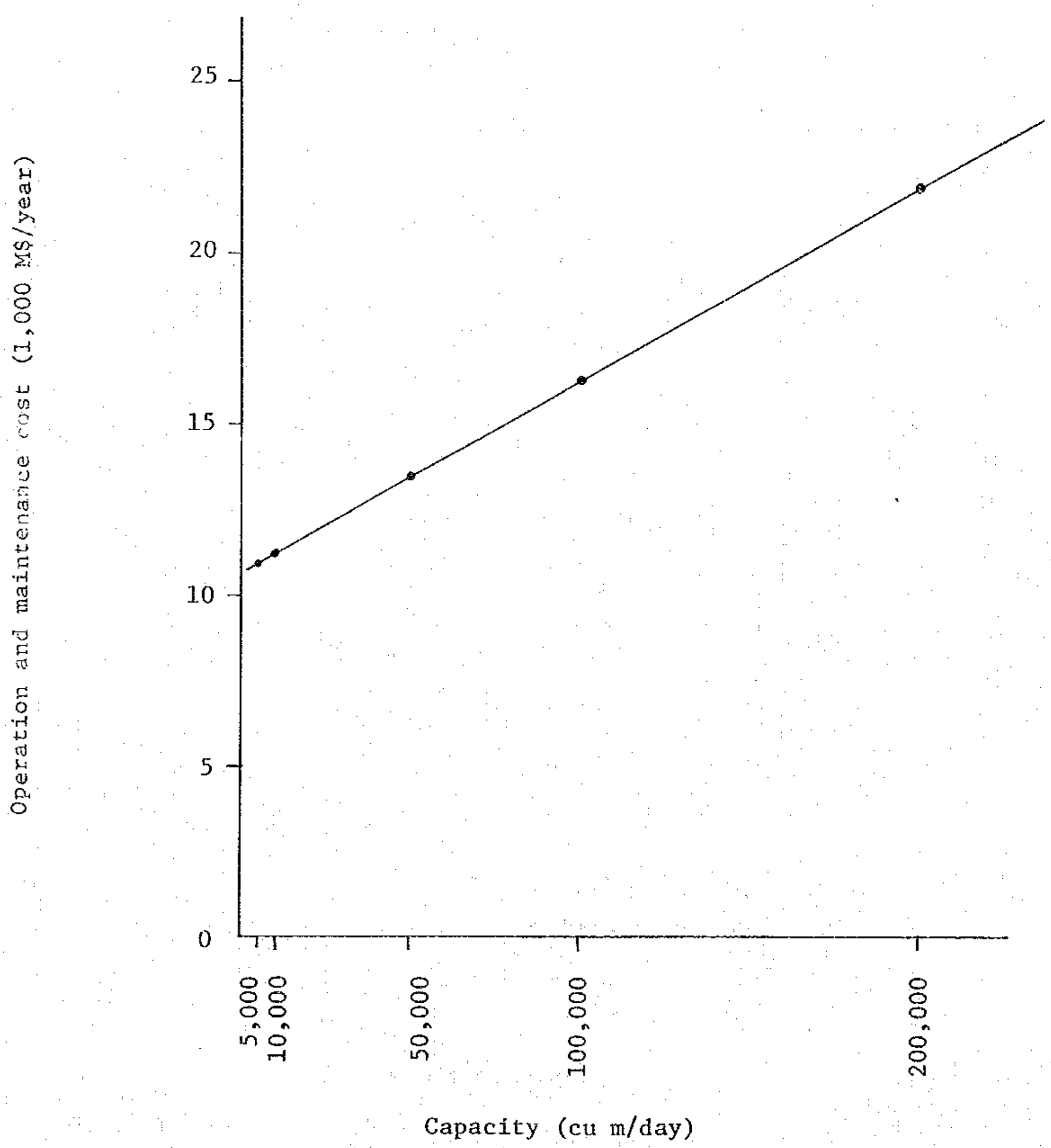


FIGURE E-10 Operation and maintenance cost for stabilization pond process

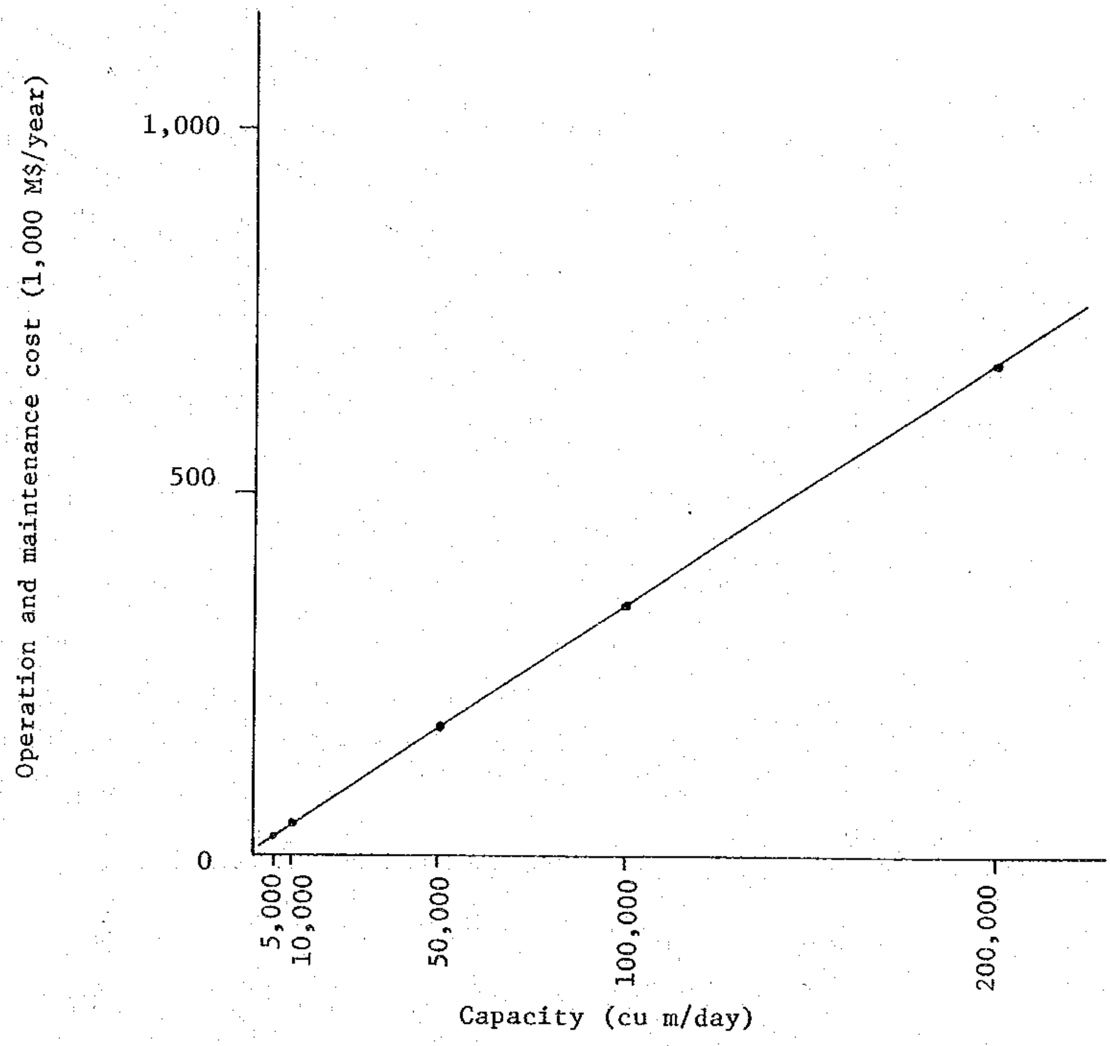


FIGURE E-11 Operation and maintenance cost for aerated lagoon process

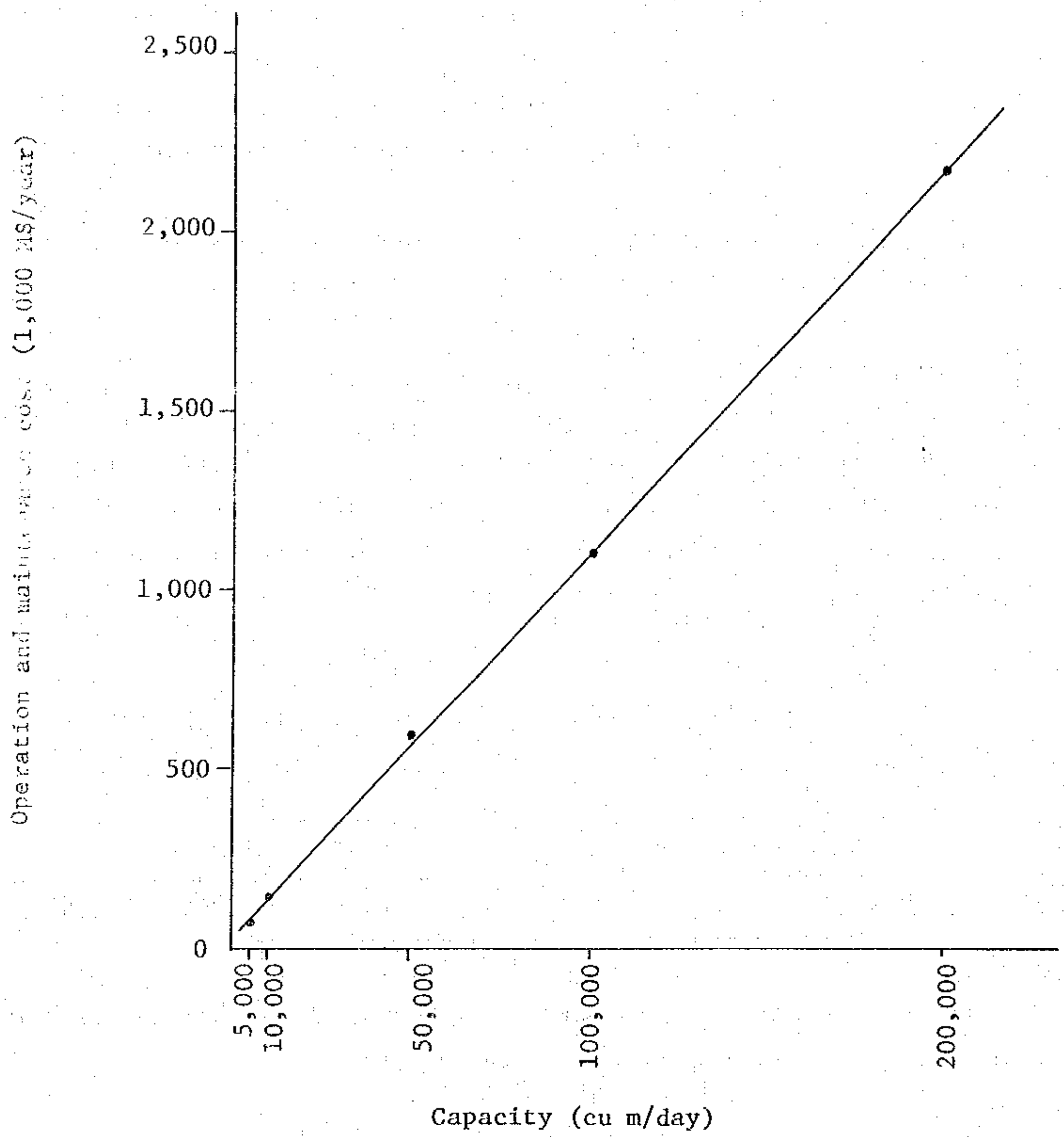


FIGURE E-12 Operation and maintenance cost for oxidation ditch process

CHAPTER 6

LAND REQUIREMENTS FOR SEWERAGE FACILITIES

6.1 General

Most of sewers are laid on the public roads, therefore no land acquisition is required. However, quite a huge land is needed for construction of treatment plant and pumping station, and these lands are usually private property so that land acquisition cost is required.

In this chapter, the relationship between required site area and capacity of pumping station and treatment plant is discussed.

6.2 Pumping Stations

In developing required land equation, 7 stations, different capacities of 0.05 cu m/sec, 0.2 cu m/sec, 0.5 cu m/sec, 0.87 cu m/sec, 1.73 cu m/sec, 3.0 cu m/sec, and 5.0 cu m/sec, were considered. From the layouts of 7 stations, site areas as shown in Table E-18 were obtained.

TABLE E-18 Required Site Area for Pumping Station

Peak flow, cu m/sec	0.05	0.2	0.5	0.87	1.73	3.0	5.0
Area, sq m	50	120	155	1,600	1,700	1,800	2,400

The relationship between peak flow and site area is illustrated in Figure E-13. The equation may be expressed as:

$$S_p = 216.7\ Q_p + 54 \qquad (Q_p \geqq 0.5 \text{ cu m/sec})$$

$$S_p = 192.3\ QP + 1{,}365 \qquad (QP > 0.5 \text{ cu m/sec})$$

where S_p: Site area, sq m

Q_p: Peak flow, cu m/sec

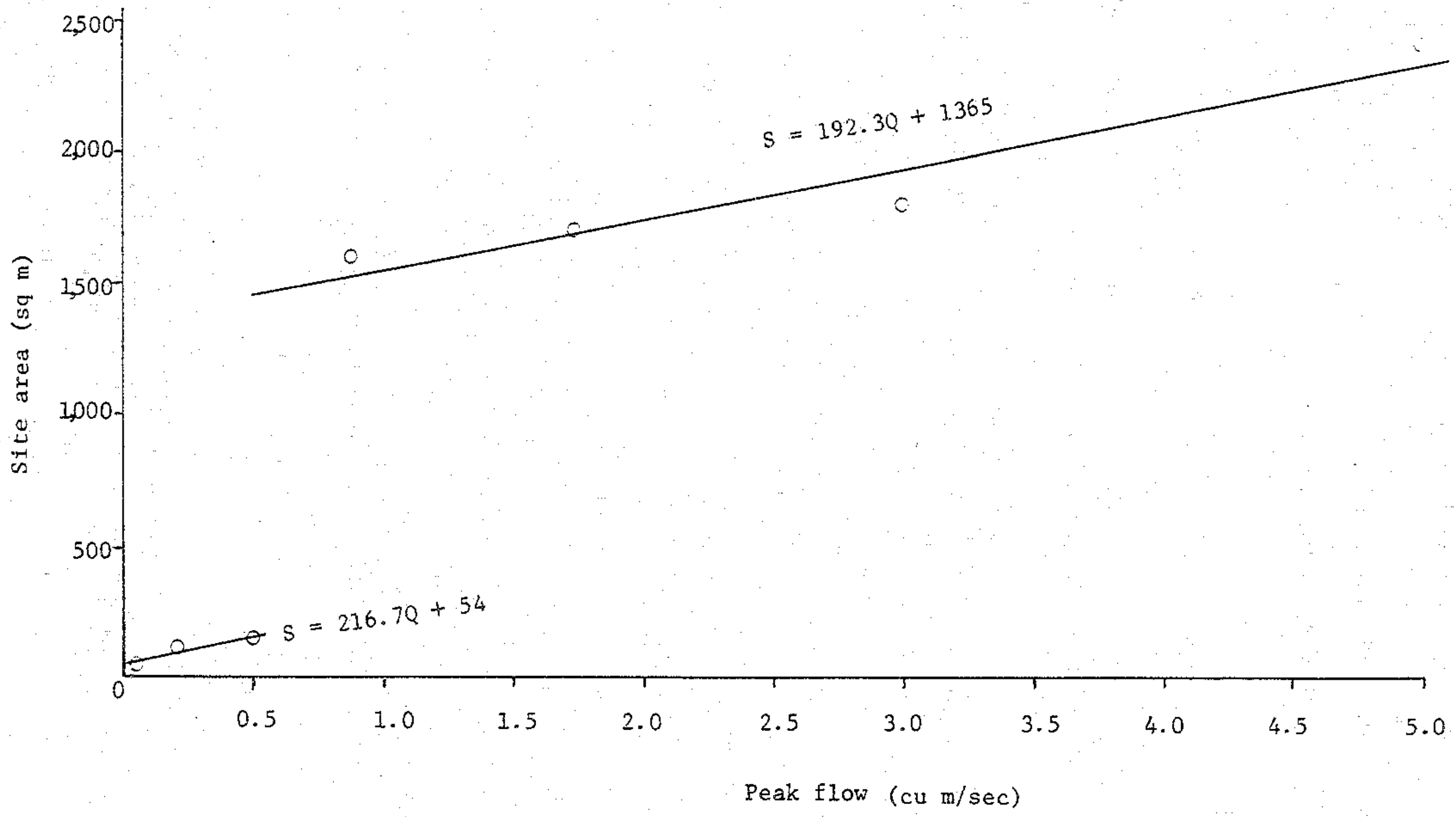


FIGURE E-13 Required site area for pumping station

6.3 Treatment Plants

On the basis of the plant layouts of 5 different capacities, 5,000 cu m/day, 10,000 cu m/day, 50,000 cu m/day, 100,000 cu m/day, and 200,000 cu m/day required site areas for different treatment processes were obtained as shown in Table E-19 and Figure E-14.

TABLE E-19 Required Site Area of Treatment Plant by Process

Unit: hectare

Capacity / Treatment process	5,000 cu m/day	10,000 cu m/day	50,000 cu m/day	100,000 cu m/day	200,000 cu m/day
Stabilization pond	6.0	11.2	52.4	98.7	197.3
Aerated lagoon	2.3	4.3	20.2	38.0	76.1
Oxidation ditch	0.6	1.1	4.9	9.2	18.5

From these table and figures, equations were developed as follows:

(a) Stabilization pond process
$S = 0.00186\ Q^{0.947}$

(b) Aerated lagoon process
$S = 0.00070\ Q^{0.948}$

(c) Oxidation ditch process
$S = 0.00022\ Q^{0.927}$

where S: required land area, ha
Q: Daily average flow, cu m/day

Total site area for treatment plant is obtained by adding the site area for pumping station which is calculated based on the peak flow instead of daily average flow.

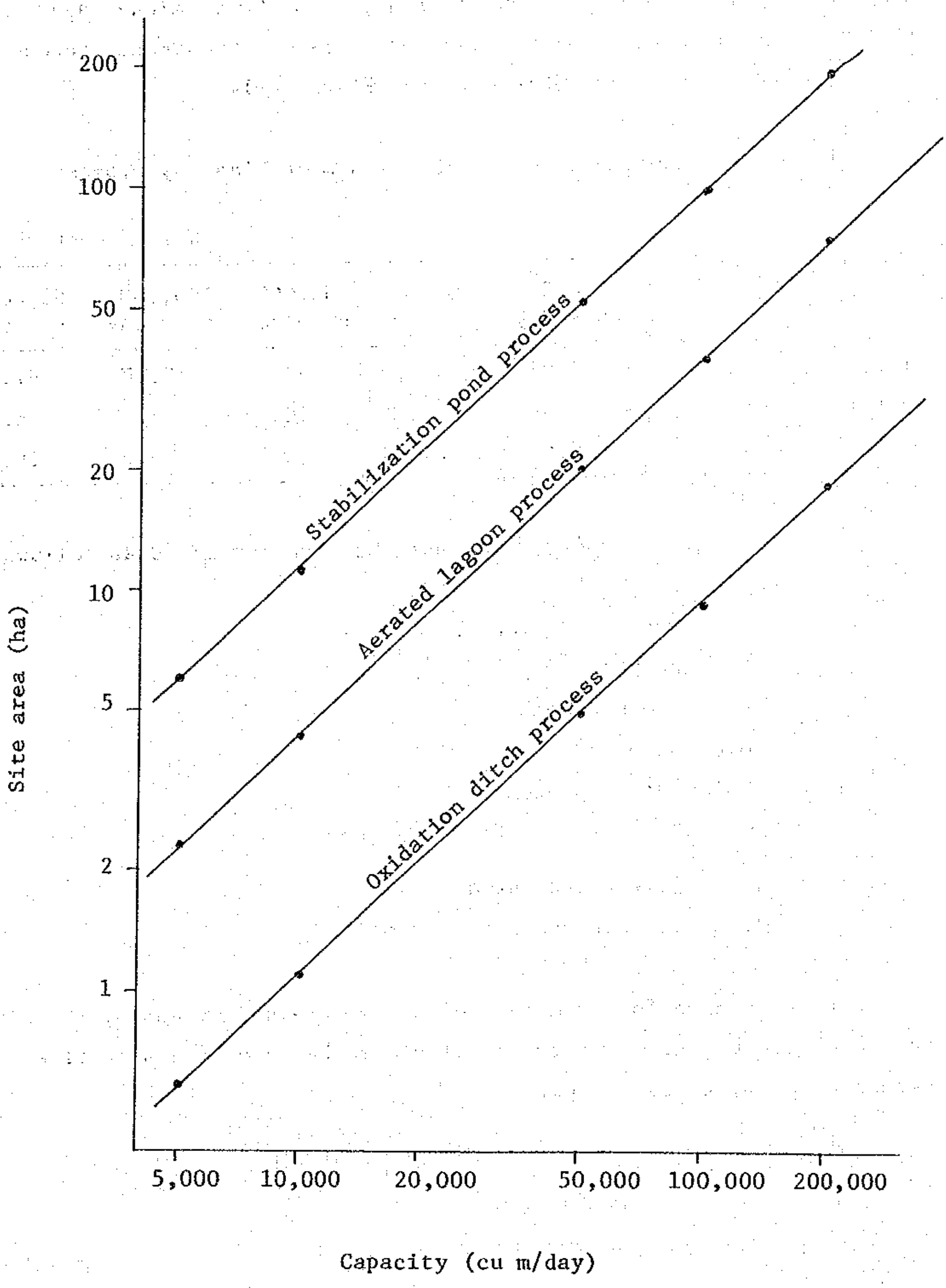


FIGURE E-14 Required site area for treatment plant

ANNEX SULFIDE CONTROL METHODS

1. Introduction

Hydrogen sulfide and other undesirable gases associated with the operation of sanitary sewers require an anaerobic environment. Therefore, the key to their control is keeping the wastewater aerobic. It has been observed that the rate of hydrogen sulfide buildup is closely related to sludge accumulation in the sewer. In other words, a well-designed, self-cleaning sewers should have little trouble from hydrogen sulfide.

Hence, the following three methods are brought up for sulfide control.

1) Keep sufficient flow velocity to prevent sulfide buildup without special sulfide corrosion protection measures.

2) Use anti-sulfide corrosion pipe or lining pipe without special velocity control where sulfide buildup is expected.

3) Inject air to keep sewage aerobic without special considerations on flow velocity and pipe material.

2. Sulfide Controlling Velocities

The equation relates flow velocities to marginal EBOD (effective BOD) is as follows:

$$\text{Marginal EBOD} = 787\ V^{3/4}\ b/P$$

where $\text{EBOD} = BOD_5 \times 1.07^{(T-20)}$

T: temperature, °C

V: flow velocity, m/sec

b/P: surface width/wetted perimeter, no dimention

The BOD concentration of the sewage in the year 2000 was estimated at 200 mg/l as discussed in the previous paragraph. The equivalent EBOD for the year 2000, at a temperature of 27°C, will be

$$200 \times 1.07^{27-20} = 321 \text{ mg/l}$$

Sulfide control velocity curve for the year 2000 condition was then developed, as shown in Figure D-14 of this Annex.

If peaking factor is expressed as:

$$P.F = \frac{5}{P^{1/7}}$$

where P : Population (1,000 persons)

and population is estimated at 4,800 persons, the P.F. will be 4. That is, the daily average flow will be one fourth of peak flow in such areas which has population of 4,800. Because, for sanitary sewer, full pipe capacity of the design peak flow rate is provided, the pipe diameter for this population will be 300 mm. (Per capita sewage flow is estimated at 230 l/cap/day). This is the upper limit of VCP market size. The minimum design flow velocity should be determined at least on the basis of the daily average flow velocity of above pipe size. Hence, the minimum design flow velocity is determined at 75 cm/sec.

The Figure indicates that if the minimum design flow velocity is decided at 75 cm/sec, the sulfide generation will be controlled from 0.25 to 0.70 of peak design flow rate. The problem of sulfide control is such more severe during the initial year of service of sewer pipeline when flows are considerably less than future design flows. However, as shown in Figure D-14, it is impossible to keep the sulfide control velocity to meet all flow variations.

Pomeroy-Davy Formula

for Marginal Effective BOD

Marginal EBOD = 787 $V^{4/3}$ b/P

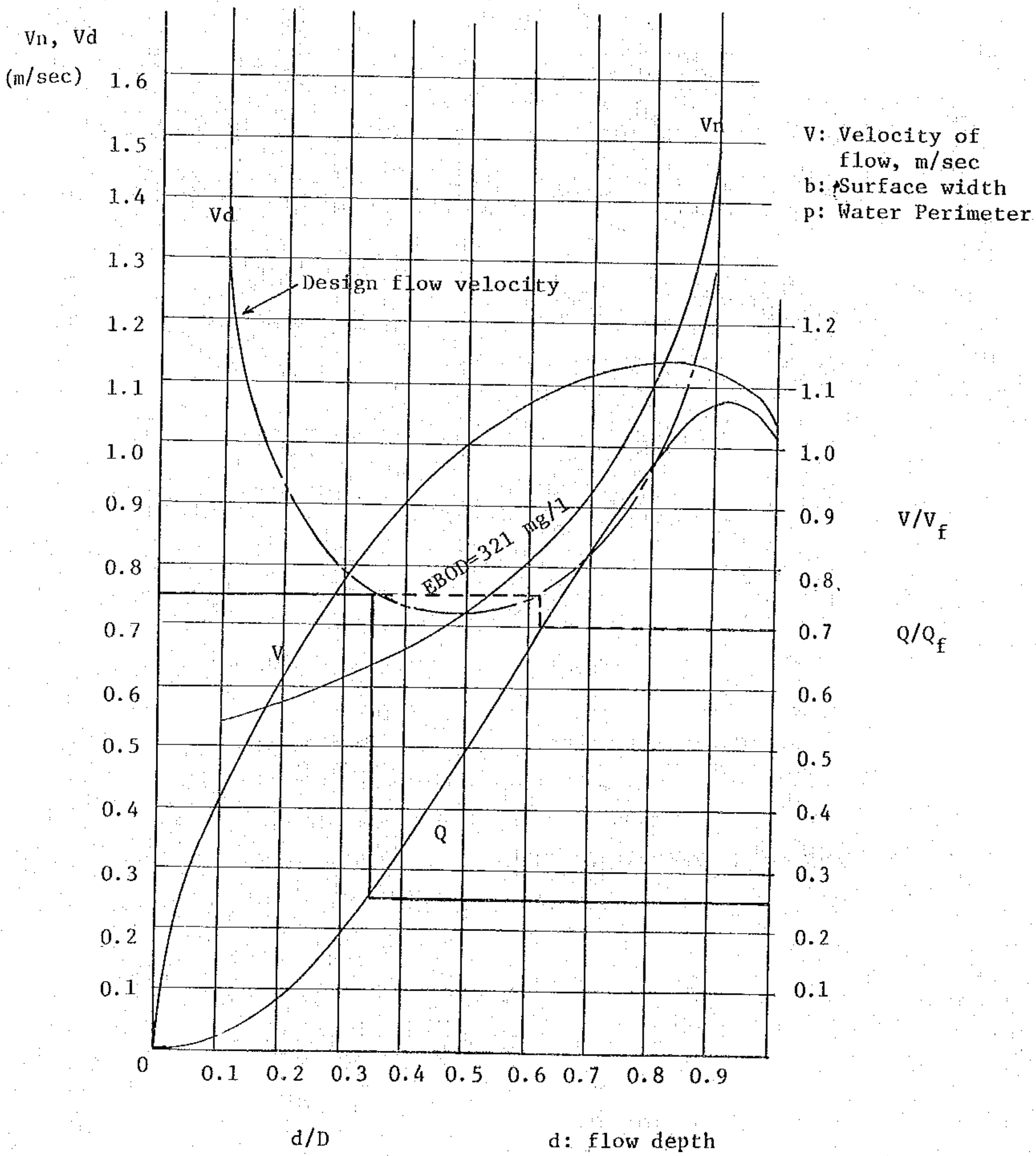


FIGURE E-14 Sulfide control velocity curve

3. Anti-Corrosion Pipe

VCP, RCP, ACP, and Pitch Fibre Pipe are available in Malaysia. Among them VCP is the best pipe against sulfide corrosion. However, the available VCP market size is up to 300 mm in diameter, and larger sewers will be of concrete-bonded pipes, either centrifugally cast or cast in place, which are likely subject to sulfide attack.

Coatings and linings of acid-resistant materials, such as vinyl and epoxy resins, PVC sheet, and high alumina cement mortar, will be effective for protecting concrete pipes against the acid attack.

4. Air Injection to Sewer

This method is useful only in the force main.

5. Conclusions

In view of the above considerations especially considering future operation and maintenance problems of the sewerage system, it is concluded that all sanitary sewers shall be so designed and constructed to give mean velocity when flowing full or half-full, or not less than 60 cm/sec for VCP, and for RCP or any cement-bonded pipe the minimum design flow velocity should be 75 cm/sec, and suitable linings or coating pipes should be used.

APPENDIX F

WASTEWATER CHARACTERISTICS

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CHAPTER 1

STUDIES ON DOMESTIC WASTEWATER

1.1 Survey on Domestic Sewage in the Project Area

1.1.1 Description of Survey

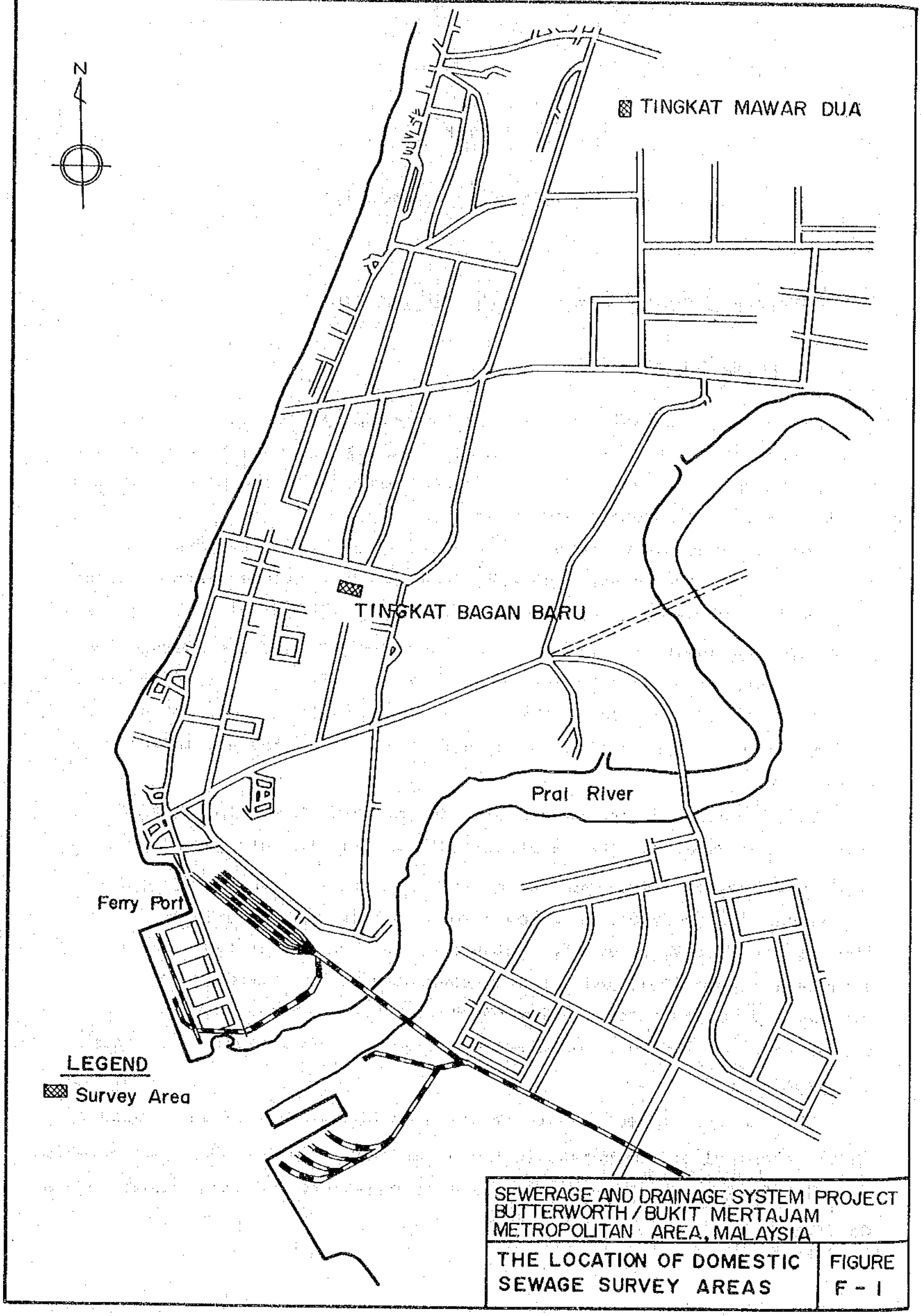
As one of the basic information for per capita waste loads and sewage flow rate projection for domestic wastewater, quality and quantity surveys of open ditches in the residential areas selected were performed by survey team in November and December, 1976.

For the domestic wastewater characteristics study, two typical housing blocks were selected whereby open concrete ditches surrounded and the domestic wastewater discharged into these distches directly at the sites of each houses by gravity. These blocks selected are average level typical residence in the Project Area, where no night soil is discharged into open ditch. Therefore, the waste load by night soil is not estimated in this survey. The location of selected housing blocks are indicated in Figure F-1.

As the outlet of the surrounding ditch, V-notch was installed to measure flow rate, and the sampling and measurements were carried out. These performances were done during the time of 6:00 a.m. to 12:00 p.m. of the day for every one to 2 hours for sampling and every 15 to 30 minutes for wastewater quality measurements. Wastewater temperature, pH, dissolved oxygen (DO), and electric conductivity were measured at the site by a potable "water quality checker", which are consisted of thermistor, pH-electrode, DO-electrode, electric conductivity electrode, and amplifiers.

The samples collected were analyzed on biochemical oxygen demand (BOD), chemical oxygen demand (CDM-Mn), suspended solids (SS), and chloride ions in the laboratories of Department of Chemistry and Indus Laboratories Co., Ltd.

FIGURE F - I



SEWERAGE AND DRAINAGE SYSTEM PROJECT
BUTTERWORTH / BUKIT MERTAJAM
METROPOLITAN AREA, MALAYSIA

THE LOCATION OF DOMESTIC SEWAGE SURVEY AREAS	FIGURE F - I

Analytical methods used are as follows :

BOD : standard method, 5 days at 20°C

COD-Mn : Japanese sewage analytical method, 100°C 30 minutes

SS : standard method

Chloride : standard method

Ref. "Standard Method", 14th edition, 1975, APHA-AWWA-WPCF

1.1.2 Results and Discussion

The findings of the domestic wastewater survey are shown in Tables F-7 and F-8.

Using the findings of the survey, quantity, and BOD and SS contents of sullage from typical residences in the Project Area were calculated as shown in Table F-1.

TABLE F-1 Quantity, and BOD and SS Contents of Sullage from the Typical Residences in the Project Area

Block	Quantity	BOD		SS	
	l/cap/day	mg/l	g/cap/day	mg/l	g/cap/day
R-1	93	241	22.4	31	3
R-2	95	224	21.3	42	4

Sullage water quantity from the typical residence is approximately 95 l/cap/day, and the BOD range from minimum of 105 mg/l to maximum of 370 mg/l as shown in Tables F-7 and F-8. The daily BOD load discharged into sullage water is estimated at approximately 22 g/cap/day as shown in Table F-1. The SS contents of the sullage water collected during the survey time are very low. This may be due to settling of SS matters in the surrounding ditches.

In estimation of quantity and BOD contents of domestic sewage for the sewerage system planning, appropriate allowances should be considered for settling matters in drains and flushing water for water closet.

Night soil is treated by communal septic tank with flushing water at the surveyed blocks. Flushing water quantity used for water closet is estimated at 60 l/cap/day empirically.

Further, the PWA's data on water supply indicate that the monthly variation of domestic water consumption extends to ±7% of average monthly water consumption, and that the field surveys mentioned above were done in lower water consumption season, therefore, approximately 15 l/cap/day of allowance is estimated for variation component.

Conclusively, present quantity of domestic sewage production is estimated at approximately 170 l/cap/day on typical residential area, as indicated below.

Water	Quantity (l/cap/day)
Sullage	95
Flushing	60
Variation allowance	15
Total	170

Because of extremely low concentration of SS in the collected samples as shown in Table F-1, it is expected that BOD matter is removed through settling in the collection system due to low velocities. This is often investigated in the Project Area because the slope of the drains are very flat in ordinary housing areas and also in surveyed blocks. Assuming 10 to 20% reduction through settling in a drain during collection of samples, 2 to 4 g/cap/day of BOD should be allowed for BOD production shown in Table F-1.

Night soil BOD has been estimated at 13 to 20 g/cap/day in Japan and other Asian countries. In this planning, 13 g/cap/day of night soil BOD is applied on the basis of experience in Japan.

Therefore, as summarized below, per capita BOD load is estimated at approximately 37 g/cap/day on the present living condition in the Project Area:

BOD load	BOD Load (g/cap/day)
Actual measurement on sullage	22
Allowance components for settling	2
Night soil BOD	13
Total	37

Although suspended solids contents of sullage water from the residential blocks surveyed were very low as shown in Table F-1, SS/BOD ratios of total domestic wastewaters are normally extended in the range of 0.8 to 1.3. Therefore, assuming SS/BOD ratio is equal to one for the normal domestic wastewater in the Project Area, per capita SS load is also presumed to be approximately 37 g/cap/day.

The present per capita flow rate and waste load of domestic sewage produced in the Project Area are estimated as shown in Table F-2.

TABLE F-2 Present per Capita Flow Rate and Waste Load of Domestic Sewage Produced in the Project Area

Flow Rate (l/cap/day)	Waste Load (g/cap/day)		Concentration (mg/l)	
	BOD	SS	BOD	SS
170	37	37	220	220

1.1.3 Data on Domestic Water Consumption

In addition to direct measures of sullage water at housing blocks, the information obtained by questionnaire which were directed to the selected households in the Project Area for the purpose of the survey on domestic water consumption rate, wastewater disposal system, and night soil disposal system, together with income level and the type of house accommodation, is employed in determining per capita sewage flow.

Table F-3 shows domestic water consumption rate by housing type which shows rough income level (i.e. type A; lower, B; medium and C; higher).

TABLE F-3 Per Capita Water Consumption Rate

Water Consumption	Number of Households			
(l/cap/day)	A	B	C	Total
less than 100	1	4	0	5
101 - 150	5	12	1	18
151 - 200	5	11	1	17
201 - 250	3	5	2	10
251 - 300	3	6	1	10
301 - 350	0	2	2	4
351 - 400	0	2	0	2
more than 401	1	6	1	8
Total household No.	18	48	8	74
Average Water Consumption	181	190	269	196

A: Kampong house (wooden house)
B: Attached terrace house, Flat house
C: Isolated or semi-detached house

Per capita water consumption rate increases from 180 to 270 l/cap/day as income increases, and about 190 l/cap/day of municipal water is consumed by medium income class.

Ther percentage of each housing types existing in the project area is shown in Table F-4 calculated on the basis of the housing census of Malaysia (1970).

TABLE F-4 Percentage of Each Housing Types Existing in the Project Area

Housing Type	A	B	C
Percentage	58.6	24.2	9.3

By applying the ratio (Table F-4) to the findings of the questionnaire mentioned above, average water consumption rate at present is estimated at 182 l/cap/day in the project area. And it can be considered that the difference between this value and the present per capita sewage production of 170 l/cap/day estimated in the preceding section may be a part of water used for car-wash, sprinkling, etc.

1.1.4 Daily Variation of Domestic Wastewater Flow Rate

The flow rate of domestic wastewater was measured every 15 to 30 minutes at the selected residential blocks. The results are shown in Figure F-2. Two different patterns, 2 peaks and 3 peaks, were obtained. These patterns reflect each living activities, cooking, bathing, washing, etc., although each peaks cannot exactly correspond to each activities.

1.2 Design Values of Domestic Wastewater

1.2.1 Sewage Flow and Strength

The results of the domestic wastewater survey, described in Section 1.1 of this Chapter, state that the present domestic wastewater volume is estimated at 170 l/cap/day. It will increase due to future economic growth, improvement of social services including water supply, toilet facilities, and other factors.

On the basis of the increasing rate of the water supply plan of PWA and other cities' plans, the future sewage flow rate is estimated at 230 l/cap/day in the year 2000.

Although both of per capita waste load and water consumption increase due to economic and living standard growth, in general, the increasing rate of latter is slightly higher than that of the former. Therefore, the strength of domestic sewage may slightly decrease in the future. So that, BOD concentration of domestic wastewater is estimated at 200 mg/l for planning purpose.

Using this value and flow contribution of 230 l/cap/day, average BOD load is given as 46 g/cap/day.

1.2.2 Comparison with Design Criteria in Various Countries

Table F-5 shows comparison of characteristics of wastewater in various cities.

TABLE F-5 Comparison with Design Criteria in Various Cities*

		BOD (mg/l)	SS (mg/l)	BOD (g/c/d)	SS (g/c/d)	Flow (l/c/d)	
Butterworth	2000	200	200	46	46	230	Design criteria recommended
Ipoh	1968	370	276	-	-	205	Average
Ipoh	2020	200	250	45	54	227	Design criteria
Kuala Lumpur	1985	250	-	55	-	220	Design criteria
Kuala Lumpur	2002	222	-	60	-	270	Design criteria
San Juan	1967	204	264	45	59	318	
Tema Ghana	1965	280	219	45	36	168	
Seoul	1985	312	374	59	73	232	Design criteria
Japan	1970	-	-	44	40	340	Design mannual
Keelung	1963	200	250	41	50	241	Design criteria

* Modified and rearranged from the followings:

(1) Manila Sewerage Report (1969)
(2) Manucipality of Ipoh Sewerage Feasibility Study; ENNEX (1974)
(3) Kuala Lumpur Master Plan for Sewerage and Sewage Disposal; D. Balsour & Sons (1975)
(4) Japanese Design Manual for Sewerage System (1972)

The value of sewage flow rate in the Project Area is predicted at 230 l/cap/day and similar to the value of Ipoh (2020), Kuala Lumpur (1985), and Seoul (1985). As the sewage flow reflects the living pattern of the area, and as it is considered that the social and commercial area would not drastically extend in the Project Area by the year 2000, the above estimated value of 230 l/cap/day is considered to be reasonable.

The estimated value of per capita BOD load is similar to the value of Ipoh (2020), San Juan, Tema Ghana, and Japanese standard (1975). While although the values Kuala Lumpur and Seoul are slightly higher than the other cities, this may be due to highly developed commercial areas in the cities because of the metropolis of the countries. Therefore, the estimated BOD value of 46 g/cap/day is considered to be reasonable.

At the ratio of SS/BOD fluctuates in every cities, it is difficult to predict the future SS value accurately. Assuming the ratio is one, the value of per capita SS production is estimated at 46 g/cap/day for planning purpose.

1.2.3 Proposed Design Criteria of Domestic Wastewater

The proposed design value of domestic wastewater for this Project is shown in Table F-6.

TABLE F-6 Proposed Design Criteria of Domestic Wastewater

Volume	Per Capita Waste Load		Concentration	
	BOD	SS	BOD	SS
(l/cap/day)	(g/cap/day)	(g/cap/day)	(mg/l)	(mg/l)
230	46	46	200	200

TABLE F-7 The Findings of Survey for Domestic Wastewater Quantity and Quality (Residential Area - 1)

Time	Q	WT	pH	DO	BOD	COD-Mn	SS	Cl$^-$
	(cu m/d)	(°C)		(mg/l)	(mg/l)	(mg/l)	(mg/l)	(mg/l)
7:40	-	-		-	365	195	30	16
8:30	6.2	23.8	6.5	2.0	360	455	47	41
9:30	24.8	23.8	6.9	2.4	285	195	39	31
10:30	16.9	27.9	7.8	1.1	300	325	49	29
11:30	19.3	26.4	7.1	1.5	130	195	24	25
13:30	26.2	28.1	5.5	2.3	180	195	24	44
15:30	12.9	27.6	5.3	2.0	315	115	34	37
17:30	18.1	27.0	6.3	0.7	225	155	38	26
18:30	19.3	26.8	6.5	1.2	215	80	23	29
19:30	20.6	26.5	6.8	0.9	240	80	18	37
21:30	7.5	25.9	7.2	0.7	370	115	16	35
23:30	4.4	24.9	6.8	0.3	195	155	34	43

Q	Quantity of Water Flow
WT	Water Temperature
DO	Dissolved Oxygen
BOD	Biochemical Oxygen Demand
COD-Mn	Chemical Oxygen Demand
SS	Suspended Solids
-	not measured

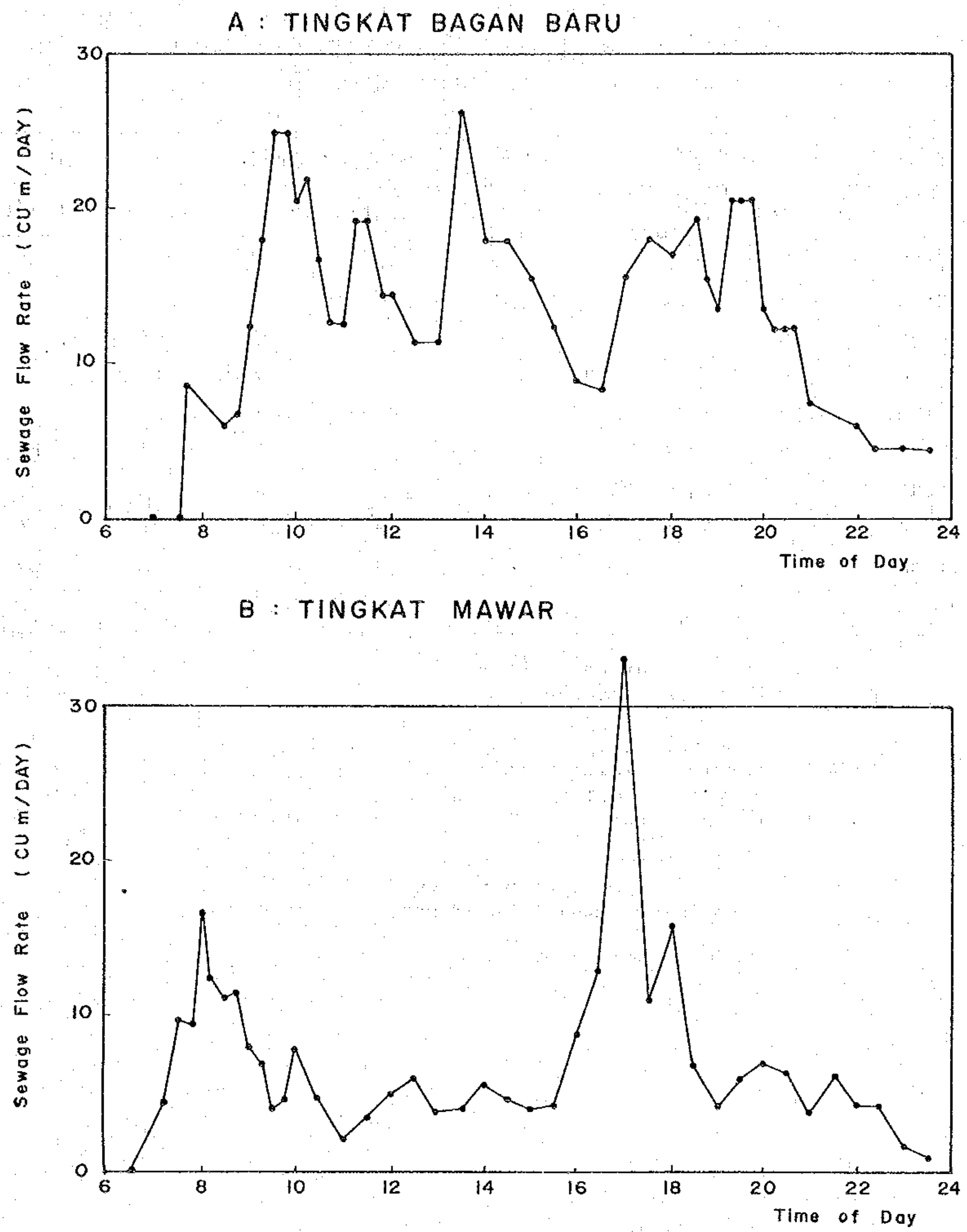
Note Date: 7 Dec., 1976
Location: Tingkat Bagan Baru Sepuluh, Butterworth

Table F-8 The Findings of Survey for Domestic Wastewater Quantity and Quality (Residential Area - 2)

Time	Q	WT	pH	DO	BOD	COD-Mn	SS	Cl^-
	(cu m/d)	(°C)		(mg/1)	(mg/1)	(mg/1)	(mg/1)	(mg/1)
7:30	9.8	26.8	7.2	4.5	105	135	25	34
8:30	10.8	27.0	6.8	4.3	275	145	14	51
9:30	4.0	27.1	6.7	3.9	110	190	35	27
11:30	3.4	28.9	6.2	3.5	125	135	42	70
13:30	4.0	30.6	6.2	4.1	195	180	21	37
15:30	4.2	29.6	7.0	1.5	320	495	86	61
17:30	10.8	28.6	6.7	4.8	255	145	26	43
19:30	6.2	28.4	-	-	320	360	94	64
21:30	6.2	-	-	-	265	450	85	102
23:30	0.9	-	-	-	140	180	31	60

Q. Quantity of Water Flow
WT Water Temperature
DO Dissolved Oxygen
BOD Biochemical Oxygen Demand
COD-Mn Chemical Oxygen Demand
SS Suspended Solids
\- not measures

Notes Date: 14 December 1976
Location: Tingkat Mawar Dua, Butterworth



SEWERAGE AND DRAINAGE SYSTEM PROJECT BUTTERWORTH / BUKIT MERTAJAM METROPOLITAN AREA, MALAYSIA	
DAILY VARIATION OF DOMESTIC WASTEWATER FLOW RATE	FIGURE F - 2

CHAPTER 2

QUANTITY AND QUALITY OF INDUSTRIAL WASTEWATER

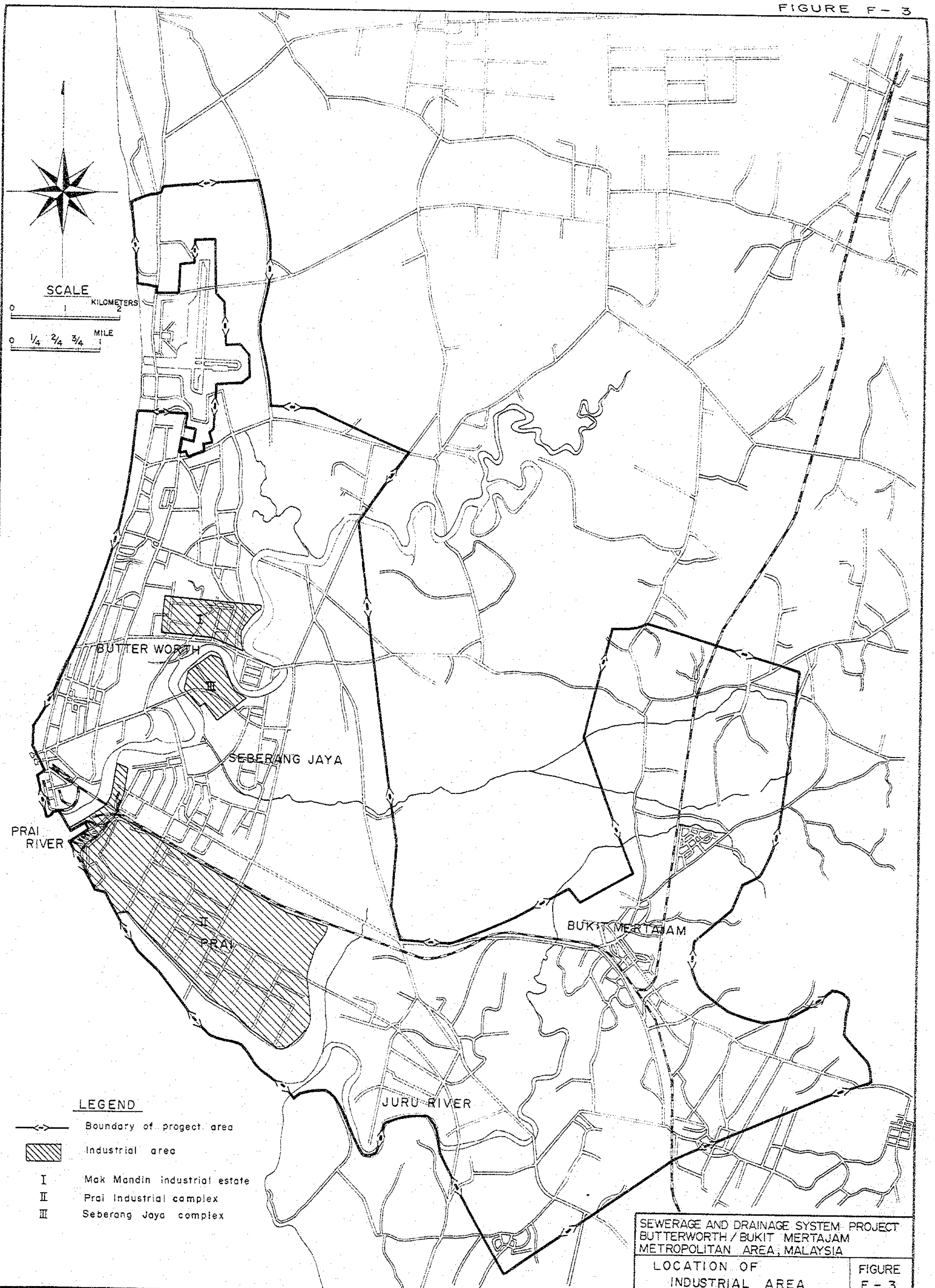
In this chapter, the present and future conditions of industry in the Project Area are described and studied on quantity and quality of industrial wastewater. All data and information used herein are offered by courtesy of Ministry of Environment, MPSP, Department of Chemistry of Penang, PDC, and PWA, together with field survey conducted by survey team through field visits and questionnaires directed to each establishments.

2.1 General

There are 8 industrial areas in Penang State. Five of them are located in the Butterworth/Bukit Mertajam Metropolitan Area (see Figure F-3).

Most of large and medium scale factories are concentrated in the industrial areas of Mak Mandin Industrial Estate, Prai Industrial Complex 1/ and Seberang Jaya Complex. Rest of the smaller scale factories are disbributed in the Bukit Mertajam District, and a few in the other area.

1/ Prai Industrial Complex is, in this report, defined as the whole area including Prai Industrial Estate, Prai Free Trade Zone, Prai Wharves Free Trade Zone, and other industrialized Prai area (see Figure F-3).


SCALE
KILOMETERS
0 1 2
MILE
0 1/4 2/4 3/4 1
BUTTER WORTH
SEBERANG JAYA
PRAI RIVER
PRAI
BUKIT MERTAJAM
JURU RIVER
LEGEND
Boundary of progect area
Industrial area
I Mak Mandin industrial estate
II Prai Industrial complex
III Seberang Jaya complex
SEWERAGE AND DRAINAGE SYSTEM PROJECT
BUTTERWORTH / BUKIT MERTAJAM
METROPOLITAN AREA, MALAYSIA
LOCATION OF INDUSTRIAL AREA
FIGURE F - 3

MAK MANDIN INDUSTRIAL ESTATE covers an area of about 110 ha within the Butterworth town area, and mainly food stuff manufacturing is operated with the rest of textile, light metal and plastic processing, and others. As of 1976, there are 32 detached-type factories in operation in the Estate, and another 13 allocated sites for future construction. In addition to the detached-type factories mentioned above, there are 48 middle scale factories, which are classified into terraced-type and/or semi-detached-type ones, in operation in MIEL units* constructed in the Estate. The Estate has already reached full utilization.

PRAI INDUSTRIAL COMPLEX covers an area of about 1,000 ha at the southeast part of the Project Area. It is the biggest industrial estate in Malaysia, in which food staff manufacturing will dominate and be followed by textile, light metal and plastic processing, and others. As of 1974, at least 27 factories employing about 6,000 workers were in operation in the area. According to PDC's information, as of end of 1976, there are 77 factories including MIEL units[1/] in operation, and 23 factories are additionally allocated in the area. This indicates that industrial development of this area is well in progress.

SEBERANG JAYA COMPLEX includes an industrial area covering about 50 ha. This complex is under construction for development and only 2 factories are in operation for production.

Number of factories in each industrial areas are shown in Table F-9.

1/ MIEL units are built by private industrial developers who put up ready-built, fully served standard factory buildings for industrialists who are unable to put up their own factories.

TABLE F-9 The Number of Factories by Industrial Classification and their Location (1976)

Class No. ***	Mak Mandin Industrial Estate		Prai Industrial Complex		Seberang Jaya
	Existing	Approved	Existing	Application	Existing
1	11	1	13	4	1
2	4	1	11	1	1
3	2	-	6	3	-
4	4	-	9	4	-
5	-	1	4	1	-
6	5	3	10	5	-
7	-	2	2	-	-
8	1	1	4	4	-
9	5	4	18	1	-
Total	32*	13	77**	23	2

* excluding MIEL units (48 factories)

** including MIEL units

*** Class No. : 1 Food
2 Textile
3 Chemicals
4 Rubber & Plastics
5 Stone & Clay Products
6 Metals
7 Electrics
8 Machinery & Equipments
9 Others

In the area outside of the three industrial areas, about 60 middle scale factories and about 700 small scale factories are scattered. Most of them are distributed among residential and/or commercial areas.

2.2 Industrial Wastewater Survey

2.2.1 Industrial Wastewater Surveys

The Sub-Committee on Pollution Control of Penang conducted an industrial wastewater survey in the Prai Industrial Complex in 1976 to study on the pollution of Kuala Juru with the assistance of MPSP and Department of Chemistry. Questionnaires were sent to 73 factories in Prai Industrial Estate to get information on the volume of water consumed and discharged. Answers to the questionnaires were received from 41 factories, and further, effluent samples from 22 factories suspected to be worst pollutants were taken and analyzed.

In addition to the survey, Ministry of Environment through its Anti-Pollution Committee in Malaysia surveyed the industrial wastewater quality of some factories in the Prai Industrial Complex and the Bukit Mertajam District. These surveys were too simple to estimate the total amount of wastes from the industrial area at present and/or in the future.

For the purpose of estimating present and future industrial wastewater production and its quality in the Project Area, survey team conducted its own industrial wastewater survey in November 1976. Using the list of factories from PDC, the factories were classified into 9 categories and 29 groups as shown in Table F-10. The questionnaires including items on (1) water consumption, (2) wastewater production and disposal, (3) treatment facilities, (4) effluent quality, (5) factory scale and expansion plan, (6) working hours, and (7) main process related to wastewater production, were sent to 47 factories altogether selected from all those classified and grouped accordingly. The staff members followed up with field visits after the questionnaires were sent, and discussed with the persons in charge. Eighty Five percent of the questionnaires sent were returned.